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VBLAST — A COMPUTER PROGRAM FOR THE DESIGN OF GRP BLAST WALL SYSTEMS

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ABSTRACT

This paper is a description of the development of the computer program VBLAST. The project included verification of the program against explosion test results obtained at FRS Cardington which are also described. Subsidiary theoretical studies involving non-linear finite element analyses were performed using the LUSAS structural analysis package.

VBLAST enables the design and analysis of blast wall systems composed of GRP (glass reinforced plastic) panels and a supporting steel frame. Dynamic deflection and reaction load time histories are calculated in response to an idealised triangular pressure loading time history defined by the user.

The panel analysis section of the program calculates the required stiffness/deflection characteristics of the GRP panels and calculates their dynamic response. Peak dynamic edge reactions are also calculated and compared with the shear resistance of the panel.

The wall design section of the software is written to choose suitable sections for the frame. These sections are checked against BS5950 requirements for buckling, shear and bending resistance with modified material factors. The mass and natural period of the whole wall is calculated and a dynamic response analysis performed to obtain deflection and reaction time histories. Peak dynamic reactions are checked against the shear capacities of the primary members. The wall panels are also checked for failure under imposed deformation of the frame.

The theoretical background to the analysis of orthotropic sandwich panels and the non-linear dynamic analysis of steel frames is described and the design philosophy underlying the choice of suitable steel sections is discussed.

INTRODUCTION

A typical GRP/steel blast wall geometry is shown in figure 1. The panels consisting of two layers of GRP separated by an insulating core are bolted to a steel frame. For the purposes of analysis or design, using the VBLAST software, the panels are assumed of equal size (equal to the maximum panel size for a wall with variable panel size).

Composite materials (such as GRP) have low density and high specific strength compared with steel. On an offshore platform, heavy blast and fire resistant structures not only entail more material and cost in their own construction and installation, but they also create a problem in terms of the support structure. It is therefore worth considering lightweight materials in the design and construction of new platforms as well as their use in upgrading existing platforms. Lightweight structures are also easier to handle during fabrication, transportation and installation.

There are also other factors to be considered for blast and fire protective structures such as fire performance, projectile resistance, durability and reliability. Often lightweight materials offer advantages over steel in one or more of these respects[1].

A GRP material is a composite material created by the synthetic assembly of reinforcing glass fibres and a compatible plastic matrix binder or resin. The resin binds the fibres together such that they act in unison. In the case of the panels manufactured for the blast test programme, the GRP skins were manufactured by a mechanised process (pultrusion) that gives reduced resin content when compared with hand lay-up. There are three main forms of re-inforcement:

1. CSM (Chopped Strand Mat): glass strands are chopped into short lengths (up to 50 mm) and distributed in a random pattern in the plane of the mat and held together by a small amount of resin binder;

2. WR (Woven Rovings): glass rovings are woven into a coarse fabric to form a bidirectional reinforcement;

3. UD (Unidirectional): untwisted long strands are aligned into unidirectional rovings, held together by a thin weft or by a light chemical binder.

GRP panels are usually constructed as woven rovings as they offer comparable strength in all directions. Strength properties of GRP are given in Table 1 in which a comparison with steel is made.

For a GRP panel the role of the insulating core in the structural deformation must be first examined in establishing the blast capacity. If an incompressible core is assumed rigidly bound to the face laminates, then all the components act in unison. In particular, the interface shear stresses need to be carefully examined as they govern delamination failure of the panel.

A GRP material fractures at strains around 2-3%, which is only a fraction of that for steels. There is hence little or no reserve strength. If at any moment the stress caused by a blast overpressure exceeds the ultimate stress of the material, a progressive fracture may occur. Therefore, a relative large safety factor should be used in the design.

Designing GRP plates with the same stiffness as steel plates results in significantly shorter natural periods. Coupled with the increased resistance, the shorter natural period generally results in lower dynamic than static response. This is shown in figure 2[2] where the relevant curves are those for utilisations less than 1.0 ($R_m/P_{max} > 1$) as the response needs to be elastic at all times.

The impact resistance of fibre reinforced composites is characterised by the presence of various energy absorbing mechanisms that are derived from their composite construction. Fibre pull out, delamination, and fibre fracture are important mechanisms in absorbing impact energy.

The relative magnitudes of the interlaminar strength of the composite and the tensile strength of the fibres determines the dominant failure mechanism. Where the interlaminar strength is high and the tensile strength of the fibres is low the failure mode is likely to be dominated by tensile failure on the non-impacting side. In composites reinforced by tough fibres such as glass and Kevlar fibre, failure is followed by debonding, fibre pull out, and delamination.

BLAST WALLS - FRAME DESIGN PHILOSOPHY

This section describes the design philosophy adopted in the design of the frame supporting the GRP panels. It is required that limited plastic deformation of the frame occurs at the design overpressure in order that an efficient design is arrived at with minimum weight.

It is important that the reaction loads transmitted to the supporting structure do not lead to failure of the primary steelwork of the module to which the wall is attached. In particular blast tension and direct loads transmitted to the vertical columns of the module should be limited as they are often under large axial, compression loads arising from their support function. As a result the major primary members of the wall are assumed to span vertically, with the horizontal secondary members present to support the panels which bear on the secondary member flanges and to restrain the front flanges of the primary members to prevent lateral torsional buckling.

In order to ensure plastic deformation of the primary members only 'plastic' sections are selected. The plastic bending capacity is chosen to be 90% of the plastic capacity required to resist the total peak load on the complete span of the member, from floor to ceiling, with a width equal to one panel width. Checks to BS5950 are performed at a peak pressure reduced by the same 90% factor. A buckling check at the actual design peak pressure is then performed. If the primaries pass these checks then the peak dynamic deflection in response to the design pressure time history is calculated and compared with the allowable value. The dynamic, shear reactions are calculated and compared with the shear capacity of the member at the connection points. If these displacements and shears are acceptable then the software proceeds to the design of the secondary members.

The secondary members are designed to resist the actual peak pressure load across the span between verticals elastically. BS5950 buckling and shear checks taking into account the required restraint axial forces from the primary member flanges are then performed. Secondary members are considered to be continuous members across the wall width. If the members pass on this basis they are accepted.

The above design philosophy ensures plastic deformation at the design pressure without buckling. The primary members are assumed identical and two way spanning action is excluded to limit load transfer into the support columns at the wall edges. Repeated runs of the software are possible to determine optimum sizing of the primary members given differing panel sizes at different locations along the wall. however uncertainty in the load distribution across the wall may make such optimisation unnecessary.

VBLAST - THEORETICAL BACKGROUND

This section is a brief discussion of the main aspects of the design of blast walling systems in GRP and steel.

Dynamic and Static Response

It is essential to consider both large deflections and dynamic effects in the design of efficient blast walls. The dynamic effects may be particularly important in determining the reaction loads on the supporting structure and the peak deflections, including rebound effects.

SLP Engineering favour the Bigg's method[2] for the incorporation of dynamic effects into the analysis. This method is an efficient way of representing the dynamics of the situation by modelling the wall and wall panels as equivalent one degree of freedom spring/mass models.

The first stage in the calculation of dynamic response is the determination of the natural period of the wall. The effective mass of the primary members and panels is calculated assuming that a panel mass equal to one panel width is associated with each primary. The effective mass of each secondary is included as a point mass at each secondary/primary intersection with a weighting derived from the deformation shape of each primary. The natural period of the wall is then calculated from the effective stiffness derived earlier.

The dynamic response of the wall is calculated from the response of the equivalent one degree of freedom spring/mass system. The loading is idealised as a triangular time history with rise time, duration and peak value input by the user. The equation of motion of the system is solved using a time-stepping technique with the appropriate stiffness at each time step being used for the elastic and plastic parts of the response.

Strain rate effects may be included as an option. The Cowper-Simmonds formulation is used[8]. The velocity of the midpoint of the wall is calculated at each time step and the yield stress is modified to reflect this through an expression for averaged strain rate related to this velocity. Strain rate effects are assumed to be in effect only during the initiation of the plastic phase of response because it is in this phase when localised strains are high as opposed to the elastic phase when strain is distributed throughout the member length.

In this way the problem is reduced to that of obtaining realistic large deformation resistance/displacement curves for the possible modes of failure of the wall or panel. These curves may be obtained from static non-linear analysis software packages such as LUSAS, or in simple geometries from analytical, closed form solutions available in the literature[3,4].

Resistance Determination

It is assumed that a means of maintaining a thermal seal between the panels and the steel frame is available, even if plastic deformation of the frame occurs.

It is necessary to determine the resistance displacement functions for a single panel and the whole wall to determine which mode of failure is associated with the minimum resistance.

The single panel resistance function may be obtained from available analytical solutions including membrane and tension effects. For FRP panels however these analytical solutions are not, in general, available as the material properties vary with direction.

The resistance functions were determined for a number of representative configurations shown in Figure 3, by use of the non-linear finite element package LUSAS. The resulting resistance/displacement curves were used to calibrate the one way span approach and be incorporated into the VBLAST software in a way which was transparent to the user.

The panels are composed of brittle materials which are assumed to behave elastically up to failure with a constant stiffness depending on the fixity or boundary conditions. The panel skin materials are assumed to be orthotropic with differing stiffnesses in the in-plane directions. The strong direction was taken to be in the longitudinal direction.

Three possible fixity conditions were allowed by the software:-

A one way span with the panel supported on its short horizontal edges. The longitudinal edges are free.

A one way span supported on its short vertical edges. The longitudinal edges are free, the strong direction is horizontal.

A panel simply supported on all edges. The strong direction is vertical.

The first two cases are essentially identical with the panel dimensions interchanged. The flexural rigidity 'DY' of the panel is calculated in the longitudinal direction from the sandwich 'wide beam' formula for faces of differing thickness[5]. The material properties for the two faces are assumed different but the tensile and compressive properties are averaged. The stiffness for the panel mid-point associated with out of plane deflections is calculated from DY.

The failure pressure is calculated from that pressure which gives yield stress in compression at the extreme fibre of face 1 or the pressure at which the extreme fibre of face 2 gives a yield stress in tension. The mid span deflection at yield taking into account shear and bending deformation is then calculated from the failure pressure.

The effective stiffness of the panel is calculated by applying a transformation factor of 0.64 corresponding to the pinned condition[2] to the stiffness derived above.

The all round supported panel case is dealt with following the method described in[3], pages 372 and 373. The flexural rigidities of the panel in the two principal directions (longitudinal and transverse) are calculated. The aspect ratio of an equivalent isotropic panel is calculated and the peak bending moments derived together with the mid span deflection at yield. Averaged tensile and compressive Young's moduli are used in this part of the calculation. The spring stiffness for out of plane displacements is calculated and the effective stiffness value is calculated from[2] using the transformation factors given in Table 5.4 of that reference.

In all boundary condition cases, the curvature of the panel at yield is calculated from the maximum bending moment and flexural rigidity. This is used later in checking for panel failure under the imposed peak deflection of the frame.

There is no comprehensive theory which rigorously defines the deflection of an orthotropic sandwich panel. A comparison of orthotropic plate theory using the equivalent thickness of the sandwich and sandwich beam theory give deflections which differ by up to 30%. For the typical panels used to test the software, however, the panel deflections are small for pressures below 1 bar and so this uncertainty is relatively unimportant in most practical cases.

Dynamic Response and Reaction Checks

The panel dynamic response is calculated using the natural period calculated in the software. The mass of the panel per unit area is calculated and the effective mass of the panel is derived using mass transformation factors depending on the panel boundary conditions. The natural angular frequency is given by the square root of the effective stiffness divided by the effective panel mass.

The equation of motion is solved for elastic deformations by a time stepping technique. At deflections above yield the panel is assumed to have failed with zero stiffness and resistance. The peak reactions on the supported edge (and at the centre of the longitudinal edge in boundary condition case 3) is calculated from expressions given in(2) Tables 5.1 and 5.4.

The dynamic shear forces at the panel edge are compared with the core shear capacity and the capacity of the panel skin/core interface.

Wall Resistance

Because of the design philosophy adopted to limit reaction loads on vertical supports, the resistance of the wall is taken to be equal to the resistance of a typical primary (vertical) member. The resistance displacement curve is built up from the elastic stiffness of a typical primary in bending expressed in terms of the out of plane displacement. The ultimate capacity of the wall is calculated from the plastic modulus and yield stress of the material. The effective yield displacement is calculated from the intercept of the elastic stiffness part of the curve with the ultimate resistance value. The LUSAS calibration runs indicated that composite action between the panel and the frame is negligible. The effective stiffness and resistance are then calculated assuming pinned ends for the primary members and using transformation factors.

Unless the 'analysis' option is used, the software will choose the steel sections automatically.

The sections for the wall frame will generally be chosen on the basis of minimal weight from the program section properties database file. The user may, however, specify maximum allowed dynamic deflection or web depth values as further restrictions on the frame design. Choice of these options may make a solution within these constraints impossible and it is recommended that the first iteration of a design is performed without these options.

Further checks on peak dynamic displacement and dynamic shear reactions are performed after the dynamic response has been calculated.

Reactions

The shear reactions at each primary member connection are calculated from expressions in reference 2, Table 5.1 at each time step. These expressions vary between the elastic and plastic phases of the response. In the plastic phase the reactions are limited by the plastic capacity of the wall. This plastic capacity will be increased by the consideration of strain rate effects and hence these effects should be included for design of connections to the supporting structure.

The peak dynamic reaction forces will in general be less than the peak applied load due to the inertia of the wall and phase differences between peak response and load. The dynamic shear forces at each primary member support are checked against the member shear capacity.

Panel Curvature Checks

The plastic deformation of the frame may induce large stresses in the panels. The maximum allowable curvature of the panels is calculated for the longitudinal and transverse directions. This maximum curvature is compared with the imposed curvature given the bolt separation.

LUSAS ANALYSES

Three different finite element analyses were carried out using the non-linear F.E. package LUSAS (see Figure 3). The analyses were performed to clarify the following;

1. The linear elastic behaviour of an orthotropic sandwich panel with various support conditions.

2. The extent of membrane action in the sandwich panel.

3. The development of plasticity in a typical steel member supporting the panels and the interaction between the panel and frame.

4. The non-linear response of the proposed test piece to a pressure of 1.0 bar static peak pressure.

The material properties used for the skins (series 500/525) and the core are listed in Table 2.

Case 1 - Single Panel A 2.4 m x 1.2 m sandwich panel was considered with the following features modelled; sandwich construction, orthotropic material properties in the skins, edges pinned in position with large deformations. Pressure loads were incremented by 0.2 bar to a maximum of 2.0 bar.

A load-deflection curve was plotted for the centre of the panel. This is compared with the load-deflection curve given by a linear analysis in Figure 4. It is clear that no significant departure from linear behaviour is experienced below 1.0 bar pressure. For pressures above 1.0 bar the deviation from linearity is around 12.5%.

Case 2 - Panels with Vertical Posts Under pressure loads the panels experience two-dimensional bending whereby they distribute the load to the primary steel members and the supports. The stronger direction in the panel skins was assumed parallel to the steel members. The steel members span across 2.4 m (the length of one panel) and no secondary steel members were assumed present.

The objectives of the analysis were to establish whether any composite action between the panel and the steel member takes place and if composite action is proved, establish its influence on the stiffness and the ultimate strength characteristics of the steel member.

To this effect, the primary steel beams were designed to carry a pressure of 1.0 bar (100 kN/m^2). Because of symmetry and the repetitive nature of the structure, only one quarter of a bay was modelled.

A bilinear stress-strain approximation was used for steel with a yield value of 355 N/mm^2.

The interface between the panel and the steel was modelled by 3-D joint elements. Elements corresponding to bolt positions were given the stiffness properties of the bolts. The only shear connectivity between the panel and the steel member was that due to the shear stiffness of the bolts.

The direct bending stresses in the steel beam computed in the FE analysis are plotted in Figure 5 for a section at midspan and six other sections at 0.15 m intervals. A shift in the neutral axis is evident with a resulting unequal distribution of stress between the two extreme fibres. The tensile stresses in the bottom fibre (corresponding to the back face of the wall) are higher than the compressive forces in the top fibre by about 100 N/mm^2. The shift in the neutral axis is approximately (17 mm) which corresponds to (8 %) of the depth of the beam.

At 1.05 bar, the pressure corresponding to the onset of yield in steel is higher than the predicted value of .92 bar: the latter being based on the elastic section modulus.

The theoretical moment capacity of a steel panel arrangement can be calculated for a limiting stress in the panel structure provided strain compatibility between steel and the panel is assumed: ie plane sections remain plane. There is an increase in the moment capacity of the composite section over that of steel alone depending on the levels of stress in the extreme fibre of the panel and the effective width. In order to obtain an increase of 8% in moment capacity at a limiting stress of 20 N/mm^2, an effective width in excess of 160 mm is needed.

Results suggest that if an effective width were to be defined it will be rather small and will be unable to explain fully the composite action.

It is, therefore, suggested that composite action may be ignored in design.

Case 3 - Test Blast Wall Model A finite element model was set up for a blast wall designed tested at the Fire Research Station. It was envisaged that the FE analysis would enable predictions to be made of the behaviour of the test wall and to verify some of the assumptions employed in the software.

The wall was supported by two primary steel members spanning 6 m and six secondary steel members at 1.2 m spacing. Edge angles on the blast face that run parallel to the primary members were also modelled.

The member sizes were as follows:

primary steel UB 406x140x46 kg/m.
secondary steel UB 127x76x13 kg/m.
edge Angle UA 125x75x12.2 kg/m.

A quarter symmetry model was set up with a regular mesh of 10 x 4 elements. In the beam elements it was possible to build up an I-section by defining three quadratic elements sharing the same nodes. This avoids the need to define the elastic and plastic properties of the section. It also means that a continuum yield criterion can be applied.

In general stress levels in the panels were found to be relatively low. The highest stresses are tensile and occur in the centre of the back face.

Figure 6 shows the load-displacement curve for the central wall for displacements between 0.6 bar and 0.9 bar. The slope of the curve suggests that the resistance of the wall is actually higher than the hinge load of 0.9 bar as the panel continues to support load after the hinge development.

Considering the load at the onset of yield and the plastic hinge load, it is clear that composite behaviour improved the load carrying capacity of the wall structure. The analysis suggests that the onset of yield increases by 12.5% and the hinge load by 32% over the values based on the relevant section moduli of the steel member.

VBLAST - PROGRAM DESCRIPTION

This section is a brief description of the computer program VBLAST.

VBLAST enables the design and analysis of blast wall systems composed of GRP (glass reinforced plastic) panels and a supporting steel frame. The primary (vertical) and secondary (horizontal) members may be selected by the software using the 'automatic' option. The selected members are checked using BS5950 for strength and buckling. Dynamic deflection and reaction load time histories are calculated in response to a triangular pressure loading time history defined by the user.

A 'hierarchy diagram' for the VBLAST program shown in Figure 7. The program is fully interactive and is composed of the following major parts:-

1. The 'Input' module deals interactively with the geometric, material and other required parameters for the problem and structure definition. The global geometry is defined in terms of panel dimensions and numbers of panels in the vertical and horizontal directions. Section types for the frame may be chosen from a data backing file available to the program. Material properties for the panels and frame may also be defined.

2. The 'Design' module is composed of two main parts, the panel analysis section and the wall design section. The panel analysis section calculates the various required stiffness/deflection characteristics of the GRP panels and calculates the dynamic response of the panel. Peak dynamic edge reactions are also calculated and compared with the shear resistance of the panel. The wall design section of the code is written to choose suitable sections for the frame. These sections are checked against BS5950 requirements for buckling, shear and bending resistance. Once the section types have been chosen the mass and natural period of the whole wall is calculated and a dynamic response analysis performed to obtain deflection and reaction time histories. Peak dynamic reactions are checked against the shear capacities of the primary members. The wall panels are checked for failure under imposed deformation of the frame.

3. The 'Output' module enables plots of the response of the panel and wall to be presented on the screen or printed to the output device. Other outputs include a global geometry plot showing the wall major dimensions. Text output in the form of a detailed or summary file is also available for later reference. Input to the VBLAST program is through the menu system.

BLAST TESTS AND RESULTS

Explosion tests were carried out on a prefabricated wall consisting of a steel frame clad on one side by FRP panels.

The main purpose of the testing was to obtain data on the performance of FRP in a "real blast" situation, in particular the behaviour of the fastening system and panels and to validate VBLAST which may be used to design other blast walls

The test piece was representative of a section of a blast wall of height 6.1 metres with simple supports at top and bottom. This fixity condition were chosen as the most unfavourable from the point of view of deflections. The two longitudinal edges of the test piece were unsupported to represent the one way spanning action of the wall. The primary members of the wall, which in the real situation would be vertical, were arranged horizontally in the FRS blast box and arranged to bear on the box support pads.

The wall was designed to resist a pressure of at least .63 bar elastically and for hinges to form at about .72 bar. The expected deflections at 1 bar overpressure were calculated and found to be acceptable. 50D steel was used for the steel frame with a minimum assumed yield stress of $350N/mm^2$. This figure for yield stress was found to be conservative as was shown by the test results.

There were no bolt penetrations on the primary member flanges and the panels of 1.2m by 1.95m spanned transversely and were bolted to the secondary steel members and angles at 30cm bolt separation. The primary members had continuous flanges with the secondary member flanges butt welded to the flanges of the primary members.

FRS Test Specification

This specification identifies the tests performed and parameters measured as part of the Vosper Thornycroft Blast Wall tests at the FRS on the 4th of September 1991.

FRS responsibilities included the provision of personnel to set up the tests, instrumentation, signal logging, operation of the blast chamber, videos and keeping a diary of events.

Having set the uprights and bearing plates into the correct position, the wall panel was lowered vertically into position. It was necessary to provide packing blocks under the wall in order that it sat at the correct vertical elevation.

Once the wall was located, sealing tape was applied around the four edges.

Instrumentation

Four parameters were measured:

- Pressure against time over wall area
- Displacement of wall against time
- Strains against time
- Reaction loads against time

Pressure transducers were connected to a signal recording system. The system had an effective cycle time of less than 1 mS. All signals (both pressure and deflection) were recorded digitally against a common time base. These were in a form suitable for subsequent manipulation by PC.

Pressure transducers were accurate to within ± 10 mbar.

There were two beams which formed the basis of the wall construction. The deformed profile of these beams was measured in detail. Panel midpoint deflections were also required for three panels.

Measurements were from outside the blast chamber on the rear of the blast wall.

Displacement transducers were accurate to within ± 1% of stroke when operating at high speed. They were suitable for digital logging at a cycle time of less than 1ms.

Prior to the tests the displacement transducers were manually displaced a known amount in order to check their correct operation.

In order to monitor the growth of plastic hinges and strain rate variations on the primary members it was suggested that strain gauges be used.

Reaction Loads at the support points were also measured.

Programme of Tests

5 blast tests were carried out on the wall section. Initial tests started at 0.3 bar. It was shown to be possible to raise the pressure to about 0.7 bar without permanently deforming the panel steelwork. At this point the pressure was raised to 1.05 bar to see how the wall resisted a significant over pressure.

VBLAST Calibration

This section is a description of the work required to calibrate the VBLAST software against the test results. The raw data was supplied to SLP on floppy discs in the form of voltage values sampled at 500μs intervals. The calibration factors were also supplied by FRS.

The offsets to the data were calculated from the average of the first 50 values obtained before each blast test. Attention was focused on Tests 3, 4 and 5 which corresponded to the highest pressures and hence were the most relevant from the point of view of wall capacity, peak dynamic response and permanent plastic deformation values.

The displacements for the twelve displacement transducers and the output from the five pressure transducers were processed and compared with the predictions from the VBLAST software. Figure 8 shows the positions of the displacement transducers on the test piece.

A VBLAST data file was constructed to represent the test wall and panels. The yield strength of the steel was set to the value from the mill certificate for the primaries. A factor of 1.1 was applied for Test 4 to take account of strain rate and strain hardening effects. A factor of 1.2 was applied for the extreme case of Test 5 for the same reason. The strain rate formulation in the software following the Cowper-Simmonds rule gives reasonable displacements without these factors but the response time histories are distorted by the relatively sudden changes of yield stress at the onset of plasticity. The theory has, however, been implemented correctly.

In the computer runs the panels were assumed simply-supported on all four sides.

The computer simulation overpressure time histories were represented as triangular pulses with the actual rise times, durations and peak pressures.

The simulation method in VBLAST does not contain any damping terms. This will not affect the peak deflections appreciably but will indicate a residual oscillation of the wall after the blast at the wall natural period. The permanent deformation is mid-way between the peaks of this residual oscillation.

Test 3 consisted of a blast with a peak overpressure of about 0.56 bar of 107 millisecond duration with a rise time of 56 milliseconds. The pressure across the wall was very nearly constant with negligible overpressures on the wall back face.

The panel deflection observed (the difference between transducers 9 and 4) is of the order of 3mm. This deflection corresponded closely to the calculated value from VBLAST although the accuracy of measurement is poor in this case.

Test 4 consisted of a blast with a peak of 0.8 bar of duration 93 milliseconds and rise time of 37 milliseconds.

A permanent deformation of the order of 9mm was observed which was also shown in the simulation. The natural period of the wall predicted by the software is also confirmed by the tests.

Test 5 was a blast of 1.05 bar with a duration of 110 milliseconds and rise time of 52 milliseconds. The pressures shown in Figure 8a indicate a pressure difference across the width of the wall of only 0.08 bar. The measured displacements shown in Figure 8b correspond closely with the simulation results shown in Figure 8c. The strain rate factor of 1.2 was applied in the simulation giving a smaller response than observed (a ductility of 1.2 as opposed to the observed value of 1.4).

A permanent plastic deformation of about 10mm is indicated by the measurements which is slightly larger than the predicted values. This small discrepancy is to be expected as the wall had already been deformed plastically in test 4. At the conclusion of the test programme, no degradation of the panels or in the integrity of the joints between them could be observed.

Fire and Impact Tests

Subsequent to the blast tests, the wall was subjected to impacts of up to 8 kJ applied by dropping a 250 Kg mass vertically onto the face of the panel which was supported at the edges in a horizontal position. After this the panels were removed from the steel framework and subjected to fire testing. The fire load followed the NPD hydrocarbon curve and was sustained for two hours. The fire tests showed that the effects of the blast and impact had not degraded the fire performance of the panel.

CONCLUSIONS

The VBLAST software predicts with excellent accuracy the peak and permanent displacements of the wall and panel under the loadings experienced in the tests.

Strain rate effects may be reliably taken into account by an increase in yield stress factor of between 1.1 and 1.2. The strain rate formulation incorporated into the software gives a reasonable estimate of peak displacements but introduces spurious distortions of the time history.

It is essential to use the true yield stress in the calculations.

Acknowledgement

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	GRP (WR)	Aluminium (6082-T6)	Steel (50D)
Density Tonnes/m^3	1.6 to 2.2	2.70	7.85
Tensile Young's Modulus (kN/mm^2)	15	70	210
Tensile Strength	200	255	355
Young's Modulus/Density	7.5	26	27
Strength/Density	100	94	45

TABLE 1. Comparison of GRP, Aluminium and Steel

Sandwich Component →	Blast Face (10 mm)	Core (50 mm)	Back Face (6 mm)
Elastic Modulus (N/mm^2) Lengthwise	12414	310	12414
Elastic Modulus (N/mm^2) Cross wise	6896	310	6207
Shear Modulus (N/mm^2)	3600	120	3600
Poisson's Ratio	0.29	0.22*	0.29

* Assumed property.

TABLE 2

SANDWICH MATERIAL PROPERTIES USED IN FINITE ELEMENT ANALYSIS

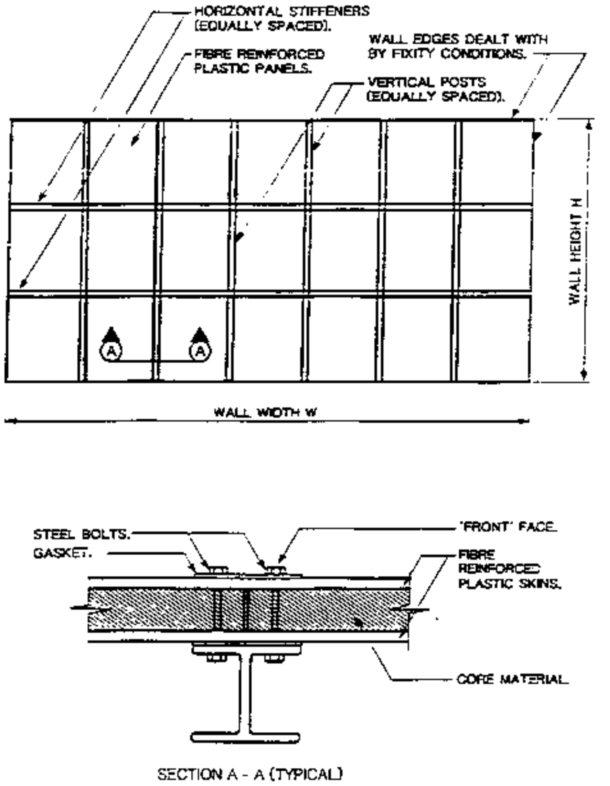

Figure 1 Typical Blast Wall Geometry

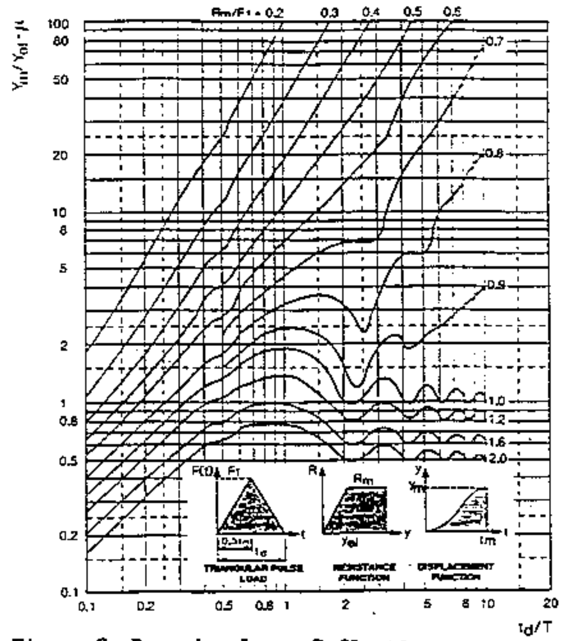

Figure 2 Dynamic, Large Deflection Response (Reference 2)

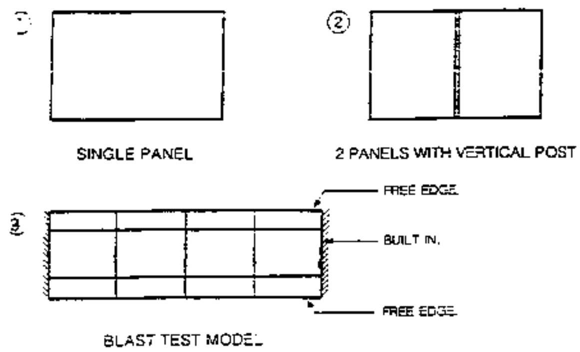

Figure 3 LUSAS Calibration Test Cases

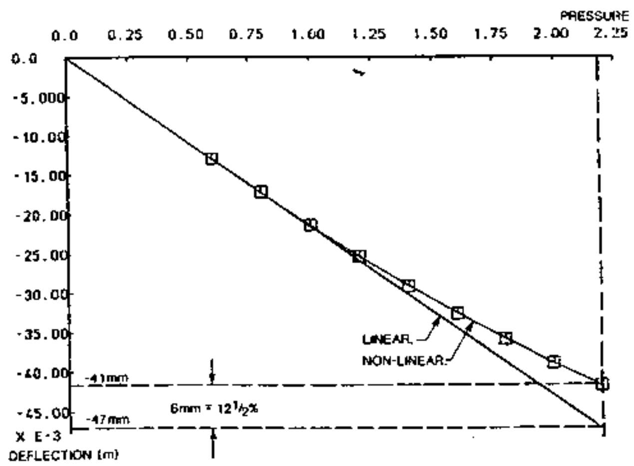

Figure 4 Midspan Deflection Vs. Pressure

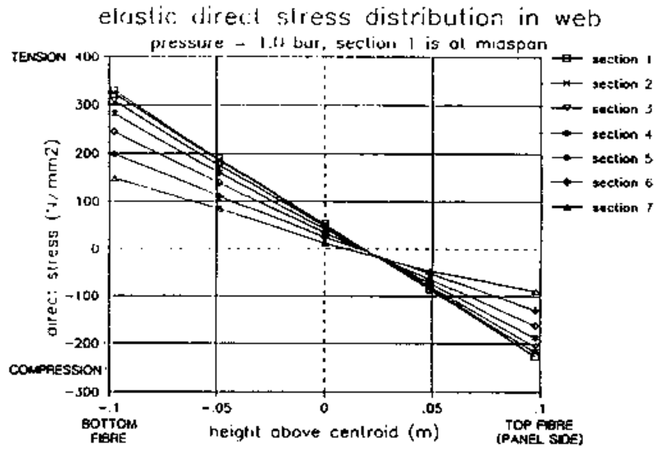

Figure 5 Elastic Stress Distribution in Web

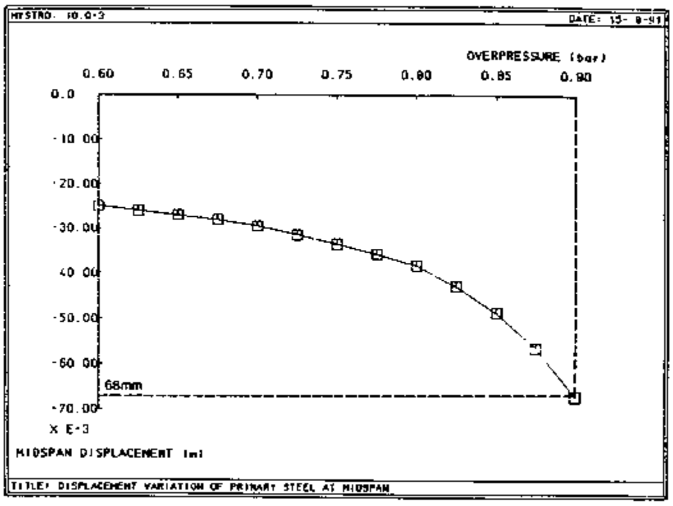

Figure 6 Load - Deflection Curve for Wall

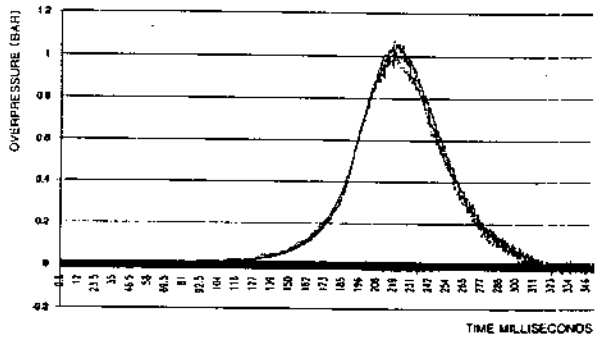

Figure 8a Test 5 Pressures

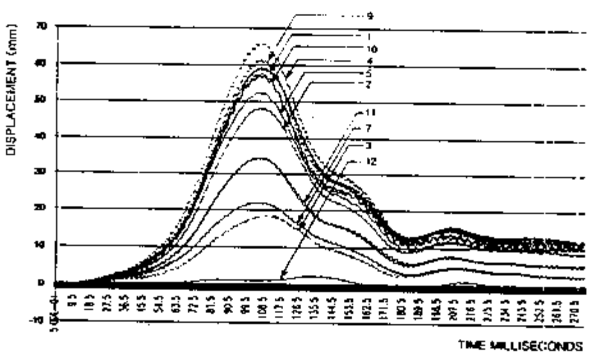

Figure 8b Test 5 Displacements

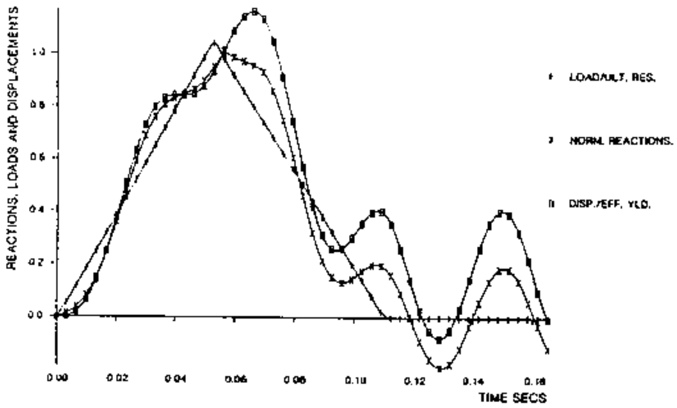

Figure 8c Test 5 Displacements (Simulation)

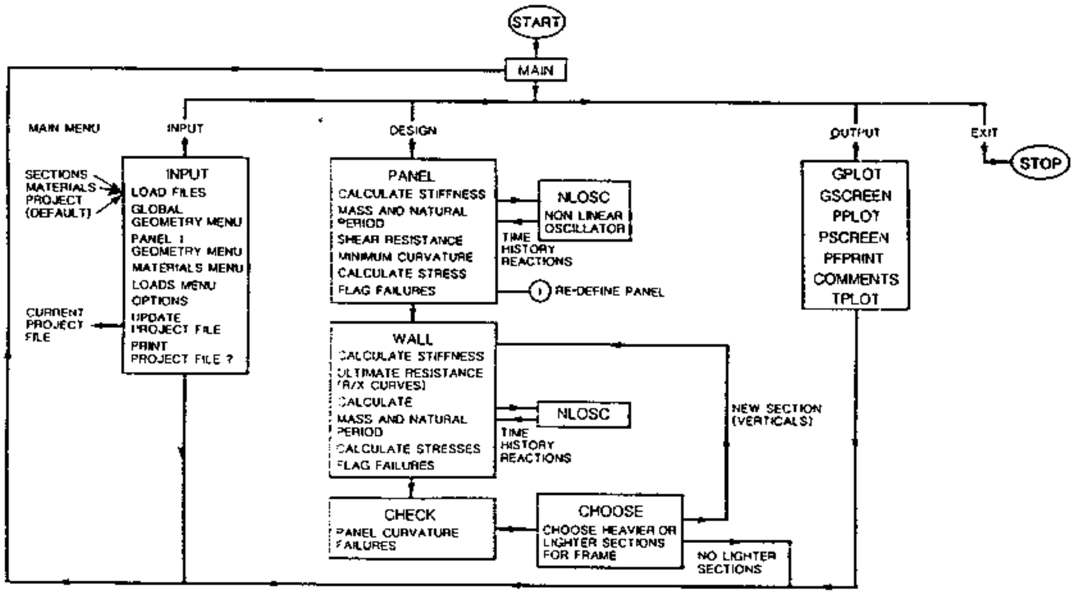

Figure 7 VBLAST Hierarchy Diagram