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(54) **Fire walls incorporating blast proofing.**

(57) Safety on for example an offshore gas or oil platform is enhanced by constructing it with walls (138) formed from fireproof sheets having predetermined resistance to plastic deformation caused by shock waves generated by blasts due to explosive ignition of for example escapes of gas. This resistance to plastic deformation will exceed a predetermined value for the fireblast wall (138) concerned which will itself be determined by inputting into a database of elastic behaviour data for structural beams (A, B, C, D) which are to extend lengthwise and transversely in a supporting framework of the fireblast walls. A computer program is then run to yield instructions for provision of an array of structural beams for providing generally at minimum cost, a minimal deflection of a shock wave incident normally on the wall sufficient to achieve the predetermined resistance to plastic deformation.

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This invention relates to a fire wall and, more particularly, a fire wall incorporating blast proofing, and to the production of such form of fire wall.

A typical fire wall is built up from Durasteel (Registered Trade Mark) sheets which have been commercially available for over 50 years and which, in their basic form, are a non-combustible, asbestos-free composite of fibre-cement between mechanically bonded metal facing sheets normally of galvanised steel, but also possibly in austenitic stainless steel or other materials when performance or environmental requirements demand. The wall may also have a sandwich construction made of a pair of such sheets with a flame retardant, for example, mineral wool filling therebetween. Coupling of sheets or sandwich panels made up therefrom at their junctions is conventionally achieved by means of L-section members extending along the edges of sheets or panels and bolted to each other at their upstanding flanges and also to the sheets or panels. If panels are employed, then at the edge regions there may be an internal reinforcement of U-section through which pass bolts connecting the panels to flanges of L-section members. Although there have been various developments in fire wall technology over the years, they are normally produced from an array of standard sheets or sandwich panels, which array is sized to the overall wall which is to be installed, with there being an appropriate array of horizontal and vertical connections to adjacent sheets or panels. As the sandwich panel construction is the more common, reference will generally be made thereto in the description which follows except where otherwise indicated.

In recent years there have been a number of situations in which fire walls of conventional type have proved to be unsatisfactory in operation, not so much because of inadequacies in their fire limiting effect but because the fires are preceded by a blast which causes deformation of the fire wall to such an extent that opposite sides of the fire wall are communicated with one another so that flame can pass from one side of the wall to the other. This has proved to be a particular problem on offshore oil and gas platforms where, inter alia, living environments have to be safely separated from working environments in the event of fire breaking out at the well head after a gas explosion has occurred.

The deformation which conventional fire walls undergo in the event of a blast is entirely plastic deformation. For a fire wall to offer resistance to a blast, it must possess overall a capacity for elastic deformation, a resilience which will counter the effect of a shock wave from a blast which is incident thereon, before onset of plastic deformation. Such behaviour when the wall is subject to the action of an intense shock wave from a blast has hitherto been generally lacking when the wall is of conventional design.

It is an object of the present invention to provide a fireproof and blastproof wall (hereinafter termed a fireblast wall) which will provide the required extent of protection of living and other environments on, for example, an offshore gas or oil platform.

According to one aspect of the present invention, there is provided a building construction comprising at least one fireblast wall located at a predetermined position in the building construction and which has a design such that it is formed from an array of fireproof sheets formed of a non-combustible, asbestos-free composite of fibre-cement between mechanically bonded metal facing sheets; or from an array of panels each formed from a spaced apart opposed pair of said fireproof sheets, optionally with flame retardant material disposed therebetween, the fireproof sheets in said array being secured to a supporting framework of structural beams extending lengthwise and transversely thereof, characterised in that beams extending in at least one direction impart to the wall a resistance to plastic deformation caused by a blast shockwave of predetermined intensity which is incident thereon, which resistance is in excess of a minimum value predetermined for a wall to be located at said position. The predetermined value will generally be calculated for a wall at a predetermined location in a building construction and for an expected maximum intensity of blast shock wave acting normally thereon. This latter parameter can be determined by a man skilled in the art who has knowledge of for example potential gas pressure build-ups on offshore oil and gas platforms.

It is preferred that positions of structural beams extending in at least one direction coincide with abutting edge regions of fireproof sheets, with the fireproof sheets then being connected to the structural beams as a means of achieving the structural integrity of the array of sheets or panels. Additional structural beams may be disposed at positions intermediate such abutting margins. The term "wall" is used herein to denote both vertical walls and roof and/or floor partitions. In the former case, at least the structural beams extending vertically will provide the resistance to plastic deformation.

The design of such a wall is not simple to achieve because of the very many considerations involved. The production of the wall is a costly procedure and the weight of reinforcement as provided by the structural beams should therefore be minimised but without prejudice to the effectiveness of the fire wall acting as a blast partition. In a single constructional project, many different sizes of fire wall may well be required and each will be in a location where it can be potentially subject to a different blast force. It would be desirable for each individual fire wall to be designed as appropriate for its location. In view of the large number of fire walls to be incorporated on an offshore platform and because of the large number of factors which has to be taken into consideration in designing such wall, the design of compartmentation for equipment protection and for quarters to be occupied on an offshore oil or gas platform with walls being blastproof would be extremely time consuming and unreliable.

Thus, according to a second aspect of the invention, there is provided a method of constructing a fireblast wall according to the first aspect of the invention which comprises the steps of:

a) establishing for the wall the value of said predetermined resistance to plastic deformation;

b) inputting into a database elastic behaviour data for structural beams to extend in at least one wall direction and which are of different profile and size and establishing in the database a file correlating this data to fireblast walls of a said design having a range of resistance to plastic deformation within which lies said predetermined resistance to plastic deformation;

c) running a computer program which yields instructions for provision of an array as aforesaid of such structural beams of identified character in panels of the wall for providing at least a minimal deflection of a shock wave incident normally on the wall sufficient to achieve said predetermined resistance to plastic deformation and;

d) constructing a said fireblast wall including such an array of structural beams as part of a said building.

Although in principle the method may be employed to produce any array of reinforcing members which will enable the wall to have the desired deformation characteristics, as there may be a variety of alternative arrays which fulfils this function, the computer is preferably programmed to select for use those reinforcing members which yield a wall at minimum cost.

Alternatively, if only one type of structural beam is available, but in different sizes, the smallest sectioned such beam type which provides the desired reinforcing effect will be indicated. In addition, different sizes of fireproof sheet or panel are commercially available and it may be desirable to use a smaller number of large sheets or panels with relatively large reinforcing members to reduce labour costs. A further approach will be for the user to have the freedom to select which beam member type he wishes to use and then he can accept the design of wall produced for such reinforcement.

The present invention thus provides a method which is simple to carry out by unskilled workers since a procedure using dynamic/plastic procedures to determine the deflection of a given structural beam for a given load condition and checking this against wall thickness and deflection criteria to see whether the structural beam is acceptable can be repeated for a wide range of different beam types and beam section sizes with all necessary details being contained in a database file. In this way, fire walls not previously contemplated can be readily produced.

If there is a door (or hatch) through the wall, the structural beams adjacent to the door will have different weights per unit length to those remote from an opening. As a door is a source of weakness in a partition, the computer will be programmed to select members for such a wall which will generally all have the same depth. Additional parameters which may be incorporated in the program will enable the handling of complex geometries or highly irregularly shaped environments with suitably sized structural beams then being proposed.

Hitherto herein, there have been no observations on the securing of the fireblast wall of the invention to side walls. The strength of the securing as such will be reflected in the maximum blast force which it is required that the wall withstand.

For a better understanding of the invention and to show how the same can be carried into effect, reference will now be made by way of example only to the accompanying drawings wherein:

Figures 1A an B are sections through conventional fire walls at panel connection positions;

Figure 2 is a section through a fireblast wall produced according to the invention;

Figure 3 shows the build up of an offshore platform and the required locations of fireblast walls;

Figure 4 is a flowchart showing the overall structure of a program whose running is embodied in the method of this invention;

Figure 5 is a computer screen design of a fireblast wall produced according to this invention; and

Figure 6 is a pressure-time curve of a blast acting on a wall structure.

Referring to Figure 1A of the drawings, a single skin construction of conventional fire wall has two panels 100 and 101 each formed of a non-combustible, asbestos-free composite of fibre-cement 102 between metal facing sheets 103 and 104 which are mechanically bonded together (not shown) and which are hereinafter referred to as "Durasteel" panels. The end edges of the sheets abut at 105 and connection therebetween is achieved by L-shaped members 106 which are connected together by means of bolts 107 and to the panels 100 and 101 by means of bolts 107a.

Figure 1B shows a sandwich construction between two wall sections 110 and 111 each of which is formed of a pair of Durasteel panels 100 and 101 respectively with a mineral wool or other flame retardant filling 112 therebetween. The respective pairs of panels 100 and 101 are separated by means of U-sectioned members 113. L-shaped members 106 are again employed on one face of panels in connecting together of the panels. However, hexagonal head self-tapping screws 108 are employed for connecting the L-shaped members to the panels. A facing strip 114 of Durasteel lies over the opposite face of the panels at the joint and the screws 108 for connection of the L-shaped members to the panels extend right through the panels and through Durasteel strips 114 to maintain structural integrity and sealing at the connection position between the respective panels.

The structure shown in Figure 2 is a modification of that appearing in Figure 1B with like reference numerals denoting like parts in Figure 1B, in that in place of the L-shaped members essentially external of the sandwich panel structure, there is positioned within the panel structure and abutting the internal faces of panels 110 and 111 an I-sectioned joist 120 whose shape and location obviates the need for the U-shaped member 113 of Figure 2. A strip 114 of Durasteel lies at both faces of panels 110 and 111 over the junction therebetween. Because of the depth of the panel structure, it is often not necessary for a flame retardant filling to be present between panels 110 and 111 or, if present, for it to occupy the entire space between the panels. The quantity of filling is determined by the fire rating required and, as shown in Figure 2 at 116 it is more of a lining for those panels 110 and 111 on which flame is likely to be incident, the flame approaching in the direction shown by arrow 119. The joist 120 is clad with like filling, usually of mineral wool, as shown at 117. The filling is attached to joist 120 and panels 110 and 111 by means of electric discharge stud fixings 118 as shown or mineral wool adhesive. Sealing is completed by silicone mastic 115 applied over both faces of panels 100 and 111 where they are to be contacted by strip 114 or joist 120.

The use of a variety of different structural beams together with, or as alternatives to, the joist shown in Figure 2 can take place in building up a wall construction or floor construction which is to have the fireproof and blastproof characteristics. Once the array of structural members has been predetermined, then conventional building techniques as used in offshore platform construction may be employed to assemble preformed walls from wall sections constructed either on or off site or from an array of panels and with structural beams possibly present at positions intermediate margins of the wall sections and certainly present at the margins of the walls sections and able, additionally, to take part then in linking of walls of different chambers either in line with each other or at an angle. The constructional techniques employed do not themselves form part of the present invention and do not require elaboration here.

Figure 3 shows through four views A, B, C and D the build up of an offshore platform. The walls which need to have at least fireproof and generally also blastproof characteristics are indicated by reference numerals 131 to 141 and appear shaded in the respective view indicated in the stage at which the wall concerned is installed. Thus, in Figure 3, the reference numerals indicate the following:

131) Escape tunnel
132) Blow out preventer walls
133) Machinery housing
134) Generator casings
135) Accommodation and service area compartment
136) Turbine house walls
137) Combustible material storage
138) Fire walls to risk elevations on manifold compression platforms
139) Protection below heli deck
140) Shield below flare stack of maintenance platform
141) Fuel storage protection

In general, of these featured walls, only shield 140 need not have blast protection. Other parts of an offshore platform which need to have both blast and fire protection will be escape routes, the water ring main, gas cables, etc.

These various walls are of different size and shape and potentially subject to different intensities of shock wave according to blasts to which they may be subject. Each requires its own careful design parameters which should enable as much onshore construction as possible to take place. Such construction preferably takes place according to the program of the flowchart of Figure 4 of the accompanying drawings. This flowchart is driven from a main menu 1 which is a first instruction screen reached on entering the program. From here it is possible to select one of ten main sub-screens 10 to 19 having the following features:-

Retrieve model 10 enables data and results to be retrieved from a previous run. The program checks at 21 to see whether any retrieved file exists or whether it does not exist and to prevent a premature exit from the program. If, at 20, the retrieve option is not used, then the program starts with a default set of values.

Geometry module 11 will be discussed in greater detail hereinafter. If changes are made to the wall geometry, then a graphics screen which will illustrate the recommended partition structure will be redrawn on exiting from the geometry module.

Loading module 12, materials module 13 and options module 14 will be described in greater detail hereinafter. Data from each of modules 11 to 14 is edited at 22 to 25 respectively.

Run module 15 causes a program to enter a design routine. The data is first checked at 26. If errors are detected, then at 33 the user is returned to the main menu. If no errors are detected, the program proceeds at

34 with the design process. Further details are set out hereinafter.

Results module 16 calls up the results sub-menu. 27 indicates the absence of availability of results with the user being returned to the main menu. If a results file is found at 28, the user may proceed to view the results. Further comments on the results follow hereinafter.

Print module 17 enables the user to print results from within the program. If no results are currently available, it is necessary at 30 for the user to call up an existing print file 36. If results are available from a recent run as indicated at 29, then it is necessary to write a print file for that run 35.

Save module 18 enables the user to save the current data 31. This is written to file and may be retrieved at a later time using the retrieve module.

Finally, exit module 19 enables the user to exit from the program in an orderly fashion with all files being closed ready for ending the program at 32. Data may be saved through this routine.

Some of the aforementioned modules will now be described in greater detail as follows.

The geometry module 11 allows the user to specify the overall dimensions and spacings for the blast wall. There is a list of parameters to set out:

a) Wall height, i.e. deck to deck height which will correspond to the vertical beam length;

b) Spacing between verticals, i.e. spacing between main load carrying members. This is preferably 1 m or 3 m corresponding to the widths of available Durasteel sheets.

c) Spacing between horizontals. Horizontal members need not always have a reinforcing function and may be of prior art type. They may span between large uprights, typically at a spacing of 1.5 m or less.

d) Door widths clear openings. This is not an essential parameter if the door width is less than the spacing between verticals. Otherwise in its calculation, account should be taken of an arbitrary allowance for flange widths of the upright and any door framing.

e) Door height clear opening, i.e. deck to horizontal support member for any uprights above door opening and to define the height above the door. If this height is less than 0.15 m, then there is no need for any reinforcing member above the door. If there is no door, this dimension is set to zero.

f) Maximum static wall thickness: this defines the maximum thickness of wall when installed. Thickness is defined as the depth of the beam plus 25 mm free to clad face. For a typical wall clad on both sides this equates to the beam depth plus 50 mm. This parameter is used to reject members that have to large a depth.

g) Deflection allowance: this is the distance beyond the static wall thickness that a wall is allowed to deflect. If the actual wall thickness is less than the maximum static wall thickness, then the difference is added to the deflection allowance. For all widths of wall, this dimension is used purely to work out the total weight of the wall.

The various supporting members will be referred to hereinafter as beams. Materials menu 13 is used to select the type and section of beam. The use of three types of material has been found to be satisfactory, these having BS4360 grades 43, 50 and 55. All are assumed to have a Young's Modulus of 210 x 10^3 N/mm^2 and yield stresses of 275, 355 and 420 N/mm^2 respectively. The program may have a facility for the user to define his own yield stress and Young's Modulus. The beam sections which may be employed include universal beam (UB), joist (JST), channel (CHNL), universal column (UC) and bearing pile (BP). Because of the wide variety of approaches which may be selected according to profile and material, which would result in a considerable computational overhead, it is preferred to preselect the section type to be used, thereby, in addition, achieving a control over the final design.

For achieving optimal design of fireblast walls a number of analysis options are available which can be used at will. These include the following:

1) Strain raise effects.

2) Composite action, i.e. adding in to the elasticity calculations those which may be attributed to the wall in its unreinforced state.

3) Cladding: This may be dispensed with on one side of the wall where an HO fire rating rather than H120 rating is acceptable. With single sided cladding, i.e. single Durasteel rather than sandwich structure of Figure 2, then the use of composite action in the computation should be prevented since single sided applications would not contribute to the elastic behaviour of the overall wall.

4) Material factor: This represents small variations in material properties such as for the natural scatter in yield strength associated with a particular grade of steel.

5) Load factor: A safety margin to be applied to the analysis.

Reverting to Figure 4 of the drawings, the results menu 16 provides five graphics screens. The first of these is a wall sketch 50 which is reproduced herein as Figure 5 of the accompanying drawings. This screen displays the geometry of the wall associated with the set of results active at the time. If results are called up from an existing file, then the screen that is displayed is appropriate to the results. Main beams are indicated at B. Determination of the appropriate uniform load for these beams is straightforward. For beams A, C and D which are associated with door openings, it is necessary to make assumptions with regard to the area that is loading the beam concerned. If the facility is being used for only a 3 or 4 point curve (see hereinafter), then the remaining pressures up to 6 should be set to zero. Horizontal beams which are here non-load bearing are not captioned but are referred to hereinafter as beams E. Certain typical dimensions which are shown are expressed in metres.

51 denotes a results summary which comprises a summary of the main input parameters plus details of the recommended beam sizes. If there is no door, results are only presented for the type B beams. Results are provided as follows:-

1) Shear at end 61, (SHEAR 1) is the reaction force at one end of the beam. It is calculated from the peak in the formally distributed load multiplied by the static load factor. It thus accounts for dynamic amplification effects. For number types B, C and D the reaction force will be the same at each end.

2) Shear at end 62, (SHEAR 2) is the reaction force at the other end of the beam and will differ from end 61 only for beam type A.

3) Permanent deflection (PRMD) is an indication of the degree of plastic response.

4) Maximum deflection (MAXMD) is the maximum deflection calculated during the dynamics routine for the size of beam specified. It includes elastic as well as plastic deflections. The maximum deflection should always be less than the allowable deflection.

5) The <u>PERIOD</u> of the wall system is determined by the dynamics routine by calculations based on classic theory and the assumption that the beam remains elastic. In practice, the beam may become plastic but not to an extent sufficient to cause its deformation to take the wall precision out of its installed position.

6) Static Load Factor (SLF): The dynamics routine solves the equation of motion on a time stepping basis. At each step it is possible to evaluate the reaction as a proportion to the reaction were the beam to be loaded statically with peak pressure. The program checks the SLF at each stage and stores the highest value. The static load factor is a good indication of whether the system being proposed is efficient. Design should aim for SLF's approximately equal to or less than unity. SLF's greater than about 1.25 indicate that the beam is responding primarily in the elastic region.

Examples of results 1) to 6) appear in Example 1 which follows together with additional information which is optionally provided for information, i.e. information in respect of wall weight, nominal wall weight for main steel comprising entirely type B beams, local to door weight, being the weight of the main steel associated with an opening and excluding the area weight there. (It comprises two type A beams and all beams lying therebetween), total weight exclusive of any doors etc. Wall displacements of interest to the engineer constructing a fireblast wall are allowable displacements which comprises the static wall thickness plus the deflection allowance less the actual wall thickness, and the second displacement which is the maximum wall displacement predicted by the program. Deflections for type A, C and D beams are additive. The maximum wall deflection is thus the greatest of the following:

1) Mid-point deflection of beam type B.

2) Mid-point deflection of beam type A.

3) Mid-point deflection of beam type C plus the deflection of beam type A at the position of intersection with beam C. The latter portion is determined on the assumption that the type A beam has a deflection varying with the square of the distance along the beam.

4) Half the mid-point deflection of beam type C plus the mid-point deflection of beam type D.

A view detail screen 52 indicates the results for those parameters which have been specifically inputted as a matter of choice by the engineer. The view detail screen allows the user to look at the results of other section sizes as screen 55. Only those section sizes that passed the preliminary selection will be available for viewing. General details will only be presented for a type B beam loading. The view detail screen will only be added to the print file at 56 if specifically requested by the user.

Finally, the program will develop a series of notes at 53 appropriate to specific sensible data which has ben inputted to provide final guidance to the designer. When the results sequence has been followed, this program section can be exited to the main menu at 54.

The design of a fireblast wall in accordance with the invention is carried out automatically by computer program once all load factors, materials, geometry and options have been correctly entered. Routines 37, 38 and 39 in the flow scheme are followed in a first step to establish the beam sizes. This is coupled with a dynamics routine 41 which however takes a relatively long time to run. Hence it is unacceptable to pass all beams to be solved. Beam sections are therefore screened at the start of the design phase. Since screening involves eliminating beams that are too large, the screening procedure is carried out, in essence, for the type A beams. However when there is no door in the fireblast wall, screening is carried out for type B beams. Screening step 38 involves the following:

1) Checking that the beam is of a type selected in the materials module.

2) Checking that the beam will not result in a wall thicker than the static wall thickness.

3) Checking that the beam elastic modulus is less than 1.25 times the required elastic modulus to resist the peak load. The required elastic modulus is determined from simple beam theory using yield of the limiting stress. This limits vastly oversized sections.

Once a beam has passed preliminary screening, the data for passing to the dynamics routine can be assembled. If the beam does not pass preliminary screening, then the next section in the properties file is to be read and screened.

At 40, the load is determined for each beam in accordance with assembled data. All of the data is available from the input routines except for the beam properties with include depth of beam and beam weight and which are required to calculate the beam mass. The beam properties filed must be read before all data can be assembled for handing to the dynamics routine. The mass handed to the dynamics routine incorporates a factor of 0.5 which is the mass transformation factor derived from classical theory.

The dynamics routine which will be described in greater detail hereinafter consists of dynamic load factor, natural period, permanent set and maximum deflection. These values are stored along with the beam properties in a dynamic array within the program for later processing.

In the beam properties file, the various beams are divided into different depth groups. All beams within that group have a nominal depth ±10 mm approximately.

Once the type A beams have been analysed, a routine is used at 43 to check whether the maximum deflection exceeds that allowable. Those depth groups that have one or more type A beams with an acceptable deflection have identification therefore stored.

For a wall with a door, it is necessary for all beams to have a similar depth. There is no purpose served in analysing a section for beams B, C or D loading unless there exists a suitable type A beam. Once a suitable type A beam has been found, the program proceeds to assemble the loads and masses. The dynamics routine is then called up and the process is repeated at 44 for all the selected sections within acceptable depth groups and for each of beams B, C and D. The results obtained are then sorted at 45 to select the beams which have been analysed to give the lightest weight wall. Sorting proceeds on a depth group by depth group basis since all beams selected for a given wall must have a similar depth. Sorting starts by identifying the lightest weight type A beam. To this is passed the deflection of the lightest weight type C beam and the lightest weight type D beam. If the deflection calculated by this procedure accepts the allowable deflection, then the next heaviest D beam is used. If none of the D beams within a given depth group is suitable then the next heaviest C beam is used and the procedure is repeated until all possible combinations have been analysed or the lightest weight wall for that particular depth group identified. A standard for the lightest weight wall may be defined as the weight of the door framing plus the weight of four type B beams. When a suitable combination of beams has been found, the section types are stored at 46.

The above procedure is repeated for each depth group. When all depth groups have been checked, the program selects the depth group that has the lightest weight wall of all. If none of the depth groups has passed all the criteria of maximum deflections exceeding allowable deflections, then the program reports at 47 that no solution has been found. To find a solution will require a change of input criteria.

If a solution has been found, the beams are identified for use in the results. The program also then defines at 48 a type E beam which is the lighter in terms of mass per metre of the following:-

1) Type B beam.

2) Beam comprising 80 x 8 mm flanges and a 5 mm thick web. The beam depth is equal to that of the type B beam.

At this stage the program returns to the main menu 1.

The flow scheme of Figure 4 will also operate with alternative routes through the program. Specifically two other routes may be identified:

a) When there is no door in the wall it is unnecessary to analyse type A, C and D beams. The program may thus be set up so that if the door width is less than the spacing between the verticals, then the door is not considered to be present. The path through the program is similar except that type B loading is used for any type A beams. The results and graphic screens are modified accordingly.

b) A user may want to specify his own section properties. These may be entered and a solution immediately found. Loads, basic geometry, options and materials are taken from the main body of the program and automatically used. Results can be sent to the print file if required.

Dynamics routine 41 forms the core of the present invention. The program uses the technique of idealising the simple beam system of a lumped mass, single degree of freedom (SDOF) model. This means that the beam can be regarded as behaving as if it were a mass oscillating on a spring. In this analogy, the spring equates to the stiffness of a beam, the mass to the beam mass and the force in the spring to the moment in the beam.

Loads can be applied to the system to force a response. A limitation of the idealisation is that, although the spring stiffness can be chosen to give a correct load deflection behaviour under dynamic loads, the response of the system is not correctly modeled. This results from inertia effects which are a function of mass, velocity and acceleration. These vary along the beam length. However if the deflected shape of a beam is known, it is possible to derive transformation factors which can be applied to the load, mass and resistance of the actual system in order for it to be correctly modeled by the SDOF idealisation.

The resistance factor is applied to the stiffness of the beam. In order for Hooke's law to hold both for the actual and equivalent system, it can be shown that the resistance factor must be equal to the load factor. The load and mass factors are derived on the assumption of a deformed shape of beam under loading. The deformed shape must obviously be a comparatively forward equation. Although this is a simplification of the analysis, it can be established that the "shape functions" used give a comparatively accurate idealisation.

In the practice of this invention, the beam will transfer from an elastic regime to a plastic regime by the formation of a hinge at mid-span. A major change in the deformed shape of beam occurs with this. It is thus necessary to use a different set of transformation factors once yielding has occurred. The transformation factors applicable to a pin-ended beam with uniformly distributed mass unload are given in the following Table 1.

Table 1

STRAIN RANGE	LOAD FACTOR	MASS FACTOR
ELASTIC	0.64	0.50
PLASTIC	0.50	0.33

The program changes from elastic values to plastic values when the yield point is reached. Since in practice this does not occur at a point, and to help reduce potential numerical instabilities, the procedure used is to linearly interpolate the load and mass factors between 0.9 of yield and yield.

In the idealised SDOF model, the force in the spring has been equated to the moment in the beam. From this it is possible to calculate the reaction forced by considering the equilibrium of the beam. However the idealisation does not have the full load applied. It is thus necessarily assumed in the idealisation to be transferred direct to the support points by known mathematical techniques. The reaction forces for the elastic and plastic cases are as follows:-

Elastic: Dynamic reaction = 0.39R + 0.11F

Plastic: Dynamic reaction = 0.38R + 0.12F

where R = instantaneous beam resistance

and F = instantaneous total load

In the application of the theory to the program, the same equation may be used for both elastic and dynamic ranges:

$$\text{Dynamic reaction} = 0.385R + 0.115F$$

The program utilised in the practice of this invention is also based on the basic equation of motion as defined below:

$$F(t) - ky - M\ddot{y} = 0$$

wherein F(t) is the load which can vary against time, k is the spring stiffness, M is the mass, y is the displacement and $\ddot{y}$ is the acceleration. Note that in the application of this equation to find the motions of a beam it is necessary to use the transformation factors mentioned above.

The above equation can be solved by numerical integration if the conditions are known at a given moment in time. This will always be the case at time T = 0 when the displacement, velocity and force are all zero. Otherwise, the numerical integration procedure is a standard procedure and numerical solutions to the equations of motion can be found in various references including "Introduction to Structural Dynamics", John M. Biggs, McGraw Hill Book Company 1964.

The basic equation of motion can be modified to include the effects of damping as follows:

$$F(t) - ky - c\dot{y} - M\ddot{y} = 0$$

wherein c denotes damping coefficient and $\dot{y}$ is velocity. The damping coefficient acts to restrict movement causing oscillations to "fade away". Most structural systems exhibit some level of damping and although difficult to define precisely, values of c of 0.08 have been found to be suitable.

For a fireblast wall embodying the invention, higher levels of damping are to be expected. Despite large initial displacements, there is almost no evidence of . induced harmonic motion. The harmonic response which is evident has a longer natural period than that of the beam and is believed to represent a warping mode. For this reason c has been increased to 0.1. A further increase in c would have overestimation consequences leading to a serious underestimate of deflection.

The basic equations of motion assume that the system remains in the elastic regime. For fireblast walls embodying the invention, it is a specific requirement that the beams be allowed to deform plastically, thereby absorbing energy and attenuating the load. The basic equation of motion can thus be modified further to reflect the fact that the resistance function of the beam is no longer following Hooke's law.

$$F(t) - R_m - c\dot{y} - M\ddot{y} = 0$$

wherein R_m is the plastic resistance of the beam.

The program first solves the aforementioned equations in the elastic region and determines the deflection. From this deflection is subtracted any plastic deflection that has occurred in previous time steps. The resulting deflection can be used with the beam stiffness to determine whether the beam resistance R_m has been exceeded. If it has, then the equation is used to determine the correct deflection.

In the above procedure it is necessary to distinguish between elastic deflection and plastic deformation since the latter remains locked in and is not recoverable. The value of R_m is not a constant and should be adjusted at each step to vary both with the values of the transformation factor and the strain rate.

One advantage of a time stepping procedure is the opportunity to make adjustments to parameters of each time step. This enables strain rate effects to be realistically included. The procedure is to work for a deflection strain for a given load step from the deflection increment according to the equation:

$$\text{STRAIN} = \varepsilon = 4d\frac{\delta}{L^2}$$

where

d = length of beam

δ = displacement increment

L = beam length

This strain is turned into a rate of strain by dividing by the time increment.

Once the rate of strain is known, it is converted to a factor which can be applied to the yield stress. This is done using the Cowper-Symonds equation:

$$\text{Yield factor} = 1 + \left(\frac{\text{rate of strain}}{D}\right)^{\frac{1}{p}}$$

where, for structural steels, the recommended values are D = 40.4 and p = 5. This factor is not applied to yield stress in the program but to the plastic resistance limit R_m.

The output of the dynamics routine is a dynamic load factor (DLF) which is the ratio of the maximum dynamic reaction force to the statically derived reaction for consuming peak load. The DLF is calculated at each time step and checked against the previous highest value. At the end of the dynamics routine, the maximum DLF is returned to the main program.

As indicated before, the load curve provided to the diameter routine is generally a 6 point curve as shown in Figure 6 in which pressure is plotted against time from explosion. Pressure build-up begins after under 1 sec. and peaks and declines again within 100 to 200 ms before subsidiary peaks occur and can also be monitored. However, load data may only be available for the first two sections of the curve for build-up to the main pressure peak and decline therefrom, i.e. a triangular distribution. The dynamics routine checks for this condition by monitoring for two successive zero load states.

In order to calculate the DLF's, the dynamics routine needs to check the load data to determine the maximum load. This is carried out at the start of the routine. The time step used in the analysis is determined by first calculating a variable TFAC which indicates the rate of change of load relative to the natural period:

$$\text{TFAC} = \frac{(\text{beam natural period})}{(\text{time interval})} \cdot \frac{(\text{change of load})}{(\text{maximum load})}$$

where time interval = period of time between start and end of the sector of curve

change of load = load change above the time interval defined above

The time step is them selected by the following criteria:

* if TFAC < 0.3 then timestep = (beam natural period)/24

* if TFAC < 0.1 then timestep = (beam natural period)/12

* if TFAC < 0.01 then timestep = time interval
* if TFAC >= 0.3 then timestep = (beam natural period)/36

The timestep determined above is converted after calculation to be an integer division of the time interval. The values against which TFAC are checked and the right hand denominators have been selected by careful sensitivity analysis and found to give solutions with accuracy better than 2% for a wide range of curve shapes. Because each part of the curve will generate a different time interval, it is necessary to analyse each part of the curve separately. This does not affect the numerical integration procedure in that there is always a starting set of conditions.

The following example illustrates the invention.

Example 1

Utilising a computer program according to the flow scheme of Figure 4 a fireblast wall was designed having the following characteristics:

GEOMETRY:

Height of wall	6.00 m	Overall width of wall	9.00 m
Spacing between verticals	1.00 m	Max spacing between horiz.	1.00 m
Door width clear opening	1.60 m	Door height clear opening	2.00 m
Max. static wall thickness	0.50 m	Deflection allowance	0.10 m

LOADING:

Default profile:			
Peak pressure	1.00 Bar	Time to peak	30.00 ms
Total load duration	120.00 ms		

MATERIALS:

Material type: GRADE 50		Young's Modulus	205 kN/mm^2
Yield Stress	355 N/mm^2	Section properties based on	BS4

OPTIONS:

Strain rate effects included
Composite action: Yes
Wall clad: Both sides

Material strength factor	1.00	Load profile factor	1.00
Sections included in program selection:		UB	

The aforementioned data was inputted into the computer and the data set out in Table 2 below outputted as a specification for a wall of lightest weight having the desired dynamic properties:

Table 2

REF	TYPE	SIZES mmxmm	WEIGHT kg	SHEAR1 kN	SHEAR2 kN	PERMD mm	MAXMD mm	PERIOD ms	SLF
A	UB	356x171	57	305.50	349.15	81	128	35	0.87
B	UB	356x127	39	267.43	267.43	74	122	37	0.89
C	UB	356x127	33	154.77	154.77	0	2	8	1.03
D	UB	356x127	33	72.68	72.68	0	10	18	1.09

Beam type E: fabricated I section: 80 mm x 8 mm thick flanges, 5 mm thick web.

The wall which was thus designed had a maximum allowable displacement of 350 mm for a predicted displacement of 68 mm, a nominal weight of 103 kg/m, a local to door weight of 123 kg/m and a total weight of 5.5 T.

Example 2

The procedure of Example 1 was repeated for like loading characteristics but with a predetermined UB section 305 x 127 mm and a weight of 48 kg. The following data were determined for a wall having such data inputted:

Table 3

BEAM TYPE	A	B	C	D
PERMANENT SET (mm)	478	81	0	0
MAX. DEFLECTION (mm)	533	134	2	10
NATURAL PERIOD (ms)	43	39	7	17
REACTION LOAD FACTOR	0.67	0.91	1.03	1.08
REACTION END 61 (kN)	233	272	154	72
REACTION END 62 (kN)	266	272	154	72

Once again it is assumed that the wall was clad on both sides. The wall weight was based upon the 'B' beams having a weight of 110 kg/m^2 with a nominal wall thickness of 360 mm. If all beams of the wall were of this type, then the maximum deflection of the wall was to be 237 mm and the total weight of the wall to be 5.7 T.

Comparative Example

The modified procedure of Example 2 was repeated using UB beams 127 x 76 mm with a weight of 13 kg. The results obtained are set out in the following Table 4:

Table 4

BEAM TYPE	A	B	C	D
PERMANENT SET (mm)	6986	5361	143	629
MAX. DEFLECTION (mm)	7156	5526	158	689
NATURAL PERIOD (ms)	129	114	22	51
REACTION LOAD FACTOR	0.13	0.19	0.73	0.60
REACTION END 61 (kN)	47	57	110	40
REACTION END 62 (kN)	53	57	110	40

Again it was assumed that the wall was clad on both sides. The wall weight based on this lighter beam was 73 kg/m^2 with there being a nominal wall thickness of 177 mm. If all the beams of the wall would have been this type, then the maximum deflection of the wall would have been 3181 mm and the total weight of the wall would have been 3.7 T. This maximum deflection is in excess of that allowable and therefore the aforementioned beam could not be utilised in practice.

In the foregoing examples the following factors were taken into account:

1) The analysis assumed that all beams were pin ended. If the beam ends have a degree of fixing, this will reduce maximum deflections but will increase reaction loads.

2) The end reactions must be resisted by supporting structures. These are significant forces which generally require local stiffening of the supporting structure.

3) The analysis assumes lateral restraint is provided to the beam flanges by the basic fireproof wall structure. This requires the structure to be attached rigidly to the beams.

4) Composite action has been assumed. This requires that the panels be attached to the steel work in the manufacturer's recommended manner.

5) The horizontal beams have been selected as I sections in order to minimise weight. If weight is not critical, a type B beam can be used for the horizontals.

6) Material stream rate effects are included. If low ductility steel is to be used and/or the wall is expected to experience very low temperature, the strain rate effects are to be removed from the computation. Alternatively, further advice is to be obtained from a metallurgist.

7) The program selects equal depth beams for the wall assuming that there is cladding on both sides. If the wall is only to be clad on one side, it may be possible to make a lighter weight construction by using different depth beams. A view detail option should then be used to obtain details of suitable beams. However, the detailing then becomes much more complex if different depth beams are used.

8) The wall is drawn on a screen as shown in the accompanying Fig. 2. This is for the purpose of identifying

beams but does not represent the full width of the wall as input on the geometry menu.

Claims

1. A building construction comprising at least one fireblast wall (138) located at a predetermined position in the building construction and which has a design such that it is formed from an array of fireproof sheets (100, 101, 110; 112) formed of a non-combustible, asbestos-free composite of fibre-cement (102) between mechanically bonded metal facing sheets (103, 104); or from an array of panels each formed from a spaced apart opposed pair of said fireproof sheets (103), optionally with flame retardant material (112) disposed therebetween, the fireproof sheets in said array being secured to a supporting framework of structural beams (A, B, C, D) extending lengthwise and transversely thereof, characterised in that beams extending in at least one direction impart to the wall (138) a resistance to plastic deformation caused by a blast shock-wave of predetermined intensity which is incident thereon, which resistance is in excess of a minimum value predetermined for a wall to be located at said position.

2. A building construction according to Claim 1, wherein structural beams (B) extending in at least one direction coincide with abutting edge regions of fireproof sheets.

3. A method of constructing a fireblast wall (138) according to Claim 1 or 2, which comprises the steps of:
   a) establishing for the wall the value of said predetermined resistance to plastic deformation;
   b) inputting into a database elastic behaviour data for structural beams (A, B, C, D) to extend in at least one wall direction and which are of different profile and size and establishing in the database a file correlating this data to fireblast walls of a said design and having a range of resistances to plastic deformation within which lies said predetermined resistance to plastic deformation;
   c) running a computer program which yields instructions for provision of an array as aforesaid of such structural beams of identified character in panels of the wall for providing at least a minimal deflection of a shock wave incident normally on the wall sufficient to achieve said predetermined resistance to plastic deformation; and
   d) constructing a said fireblast wall (138) including such an array of structural beams as part of a said building.

4. A method according to Claim 3, wherein the database contains structural beam cost data and the computer program yields said instructions for providing a wall having said minimal deflection at minimal constructional cost.

5. A method according to Claim 3, wherein the computer program yields said instructions for providing a wall having minimal weight for said minimal deflection.

6. A method according to Claim 3 or 5, wherein the computer program is able to yield said instructions for one or more predetermined structural beam types and/or sizes only.

7. A method according to any one of Claims 3 to 6, wherein said wall comprises one or more openings and their weakening of the wall is compensated for in said instructions.

8. A method according to any one of Claims 3 to 7, wherein only structural beams having a static level factor equal to or less than unity are inclined in said array.

9. A method according to any of Claims 3 to 8, wherein the structural beams all have an elasticity modulus less than a multiple by 1.25 of the elasticity modulus required to provide resistance to plastic deformation equal to said predetermined value.

10. A method according to any of Claims 3 to 9, wherein all structural beams (A, B, C, D) in the panels of the wall have the same depth.

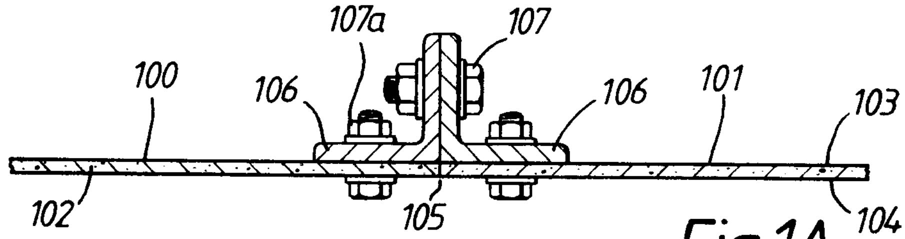


Fig.1A.

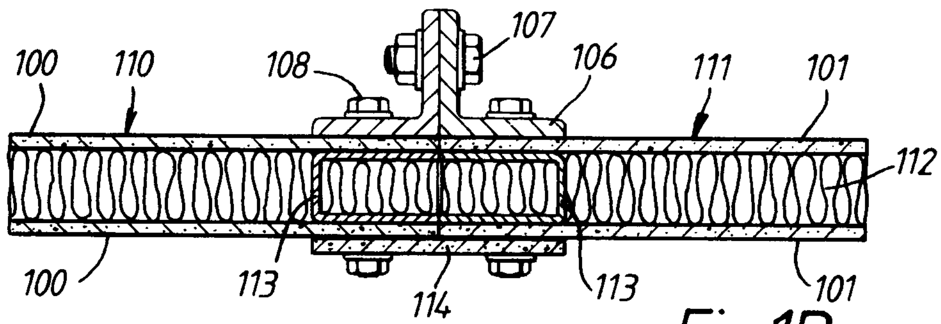


Fig.1B.

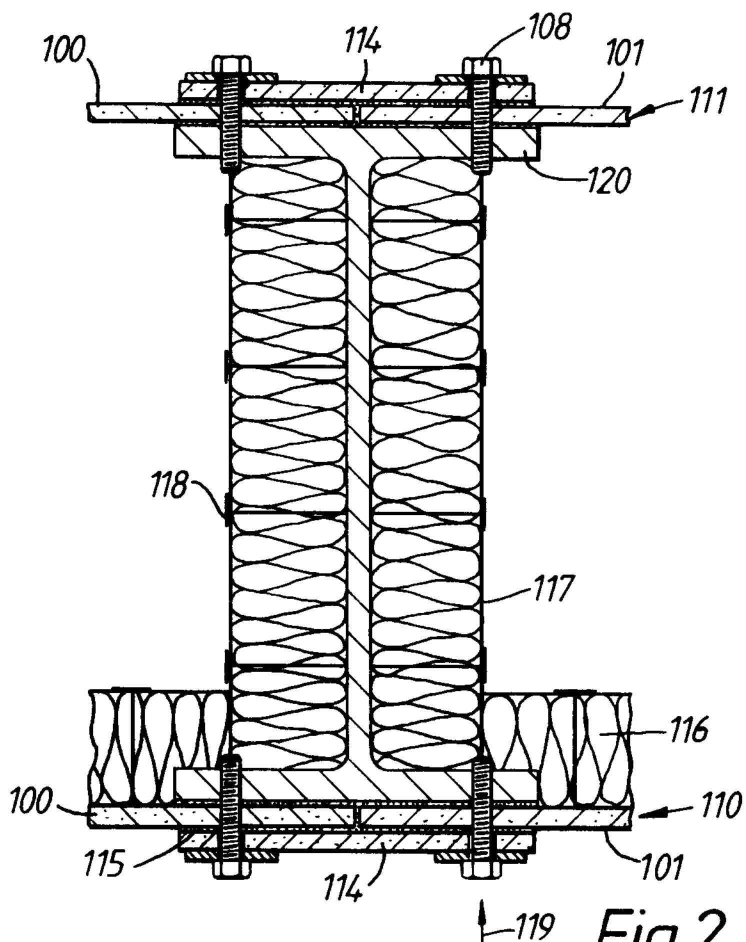


Fig.2.

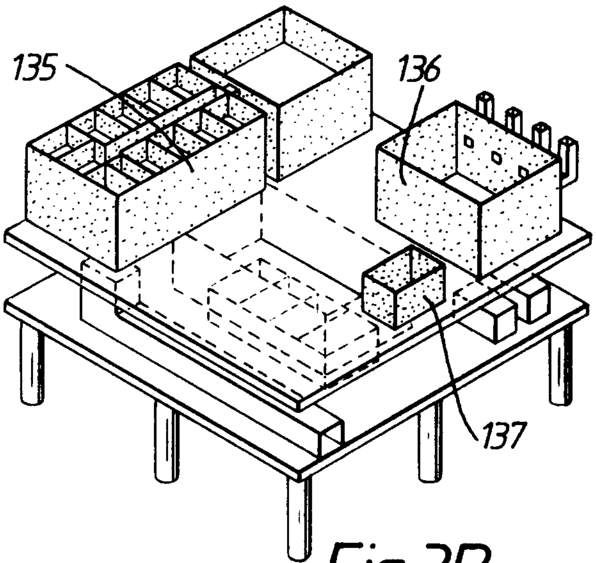


Fig.3B.

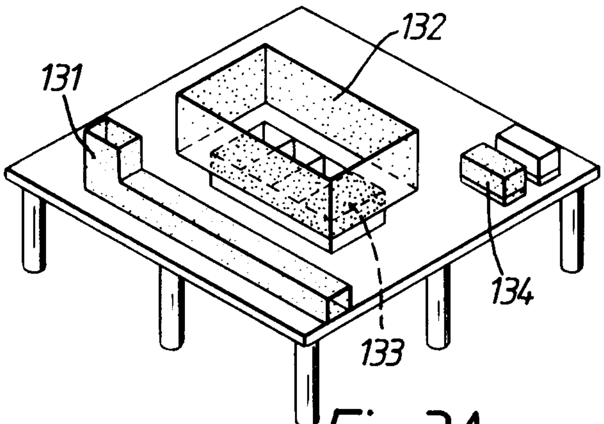


Fig.3A.

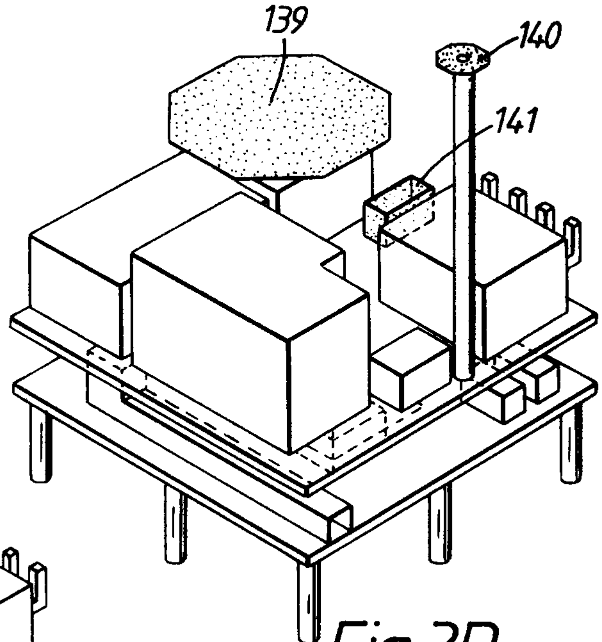


Fig.3D.

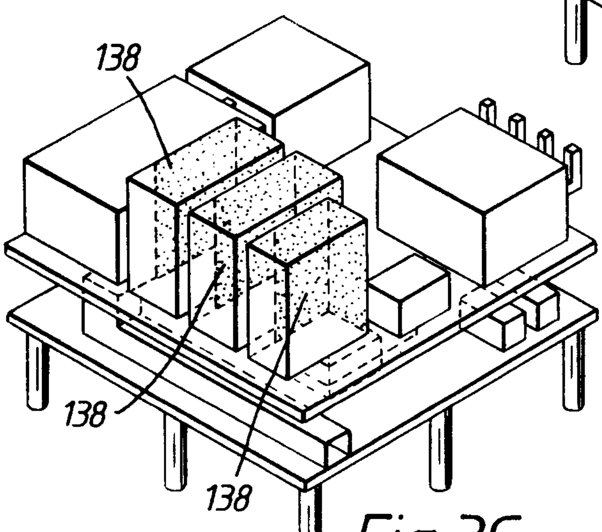
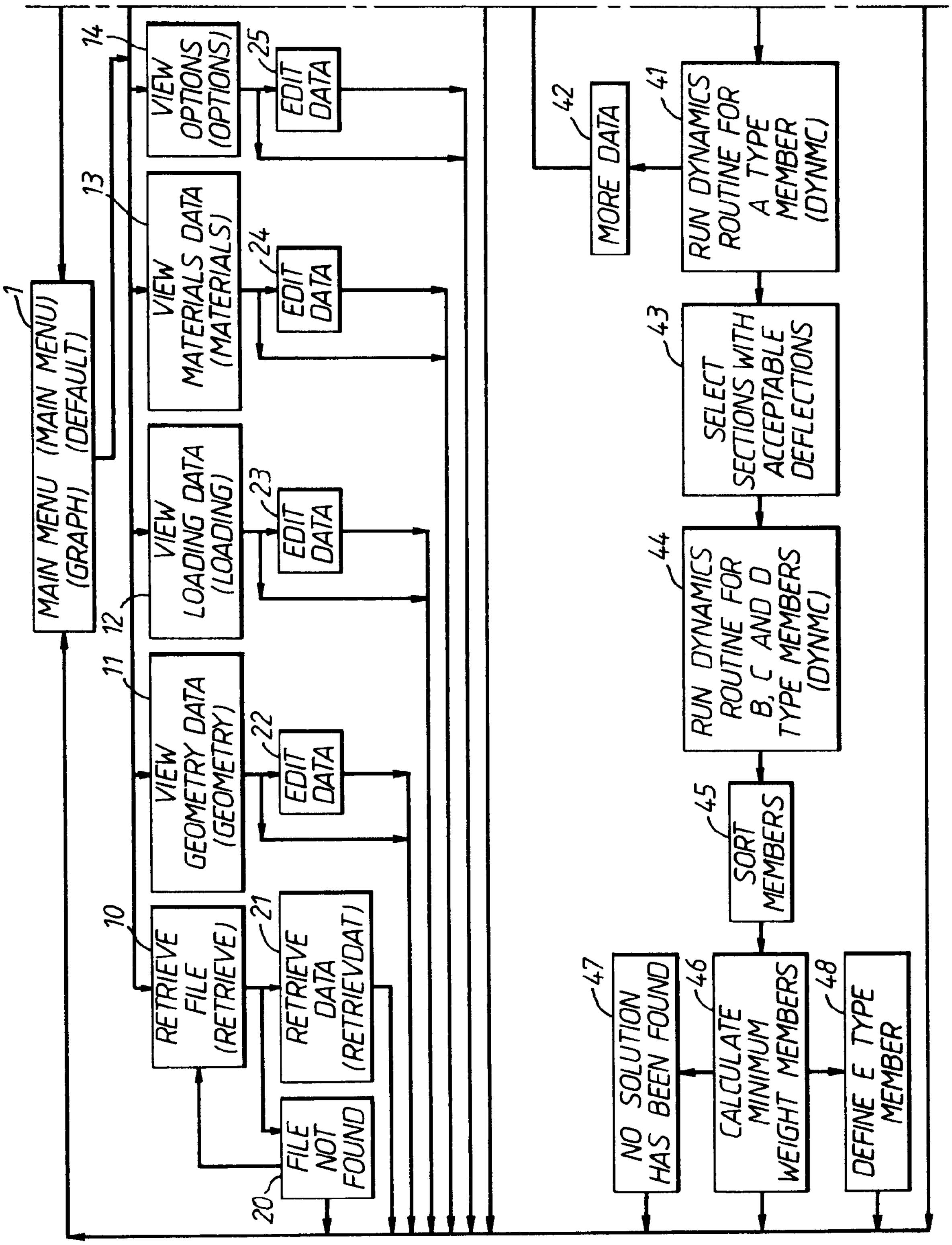


Fig.3C.

Fig.4.


MAIN MENU (MAIN MENU) (GRAPH) (DEFAULT)
1
RETRIEVE FILE (RETRIEVE)
10
VIEW GEOMETRY DATA (GEOMETRY)
11
VIEW LOADING DATA (LOADING)
12
VIEW MATERIALS DATA (MATERIALS)
13
VIEW OPTIONS (OPTIONS)
14
FILE NOT FOUND
20
RETRIEVE DATA (RETRIEVDAT)
21
EDIT DATA
22
EDIT DATA
23
EDIT DATA
24
EDIT DATA
25
RUN DYNAMICS ROUTINE FOR A TYPE MEMBER (DYNMC)
41
MORE DATA
42
SELECT SECTIONS WITH ACCEPTABLE DEFLECTIONS
43
RUN DYNAMICS ROUTINE FOR B, C AND D TYPE MEMBERS (DYNMC)
44
SORT MEMBERS
45
CALCULATE MINIMUM WEIGHT MEMBERS
46
NO SOLUTION HAS BEEN FOUND
47
DEFINE E TYPE MEMBER
48

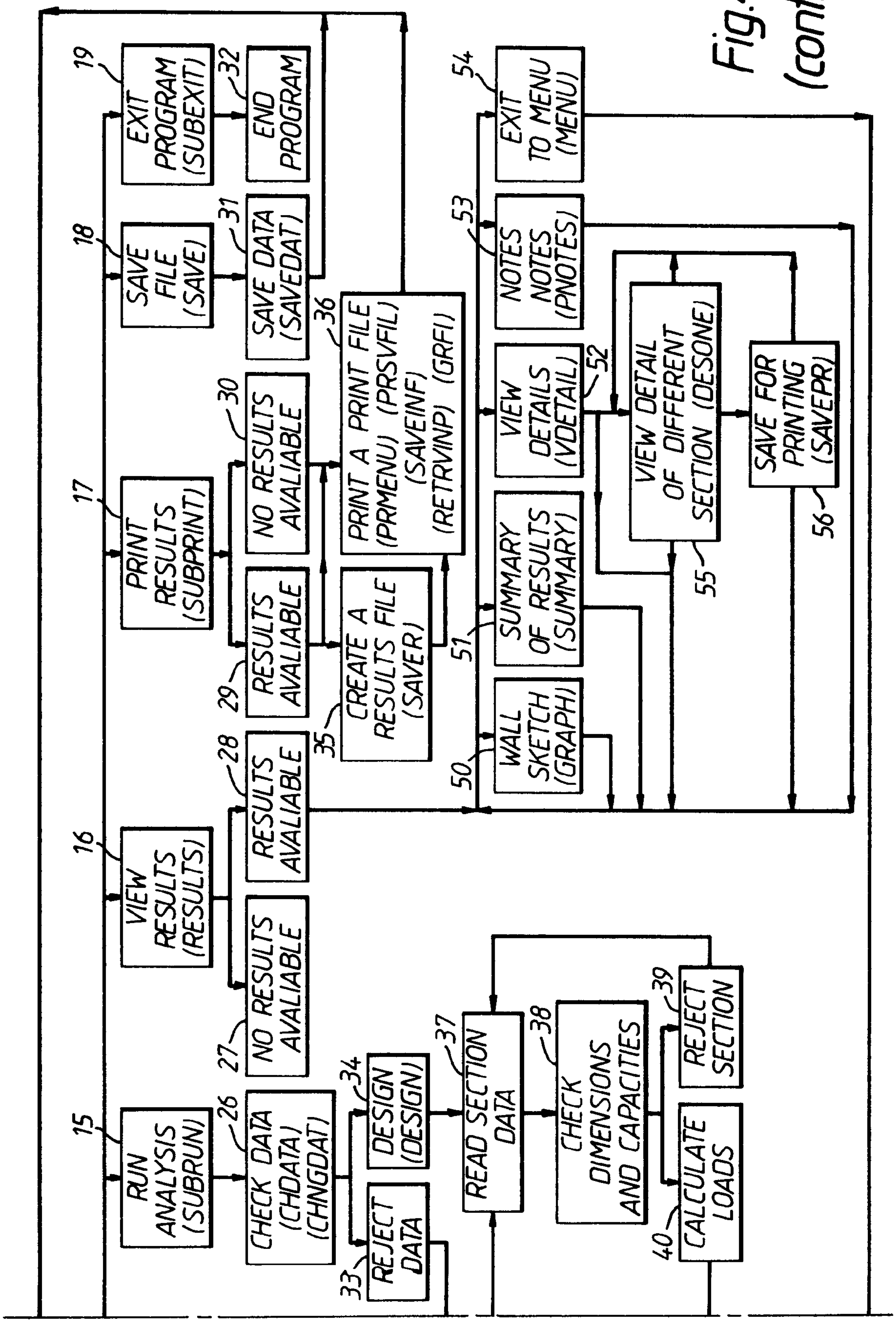
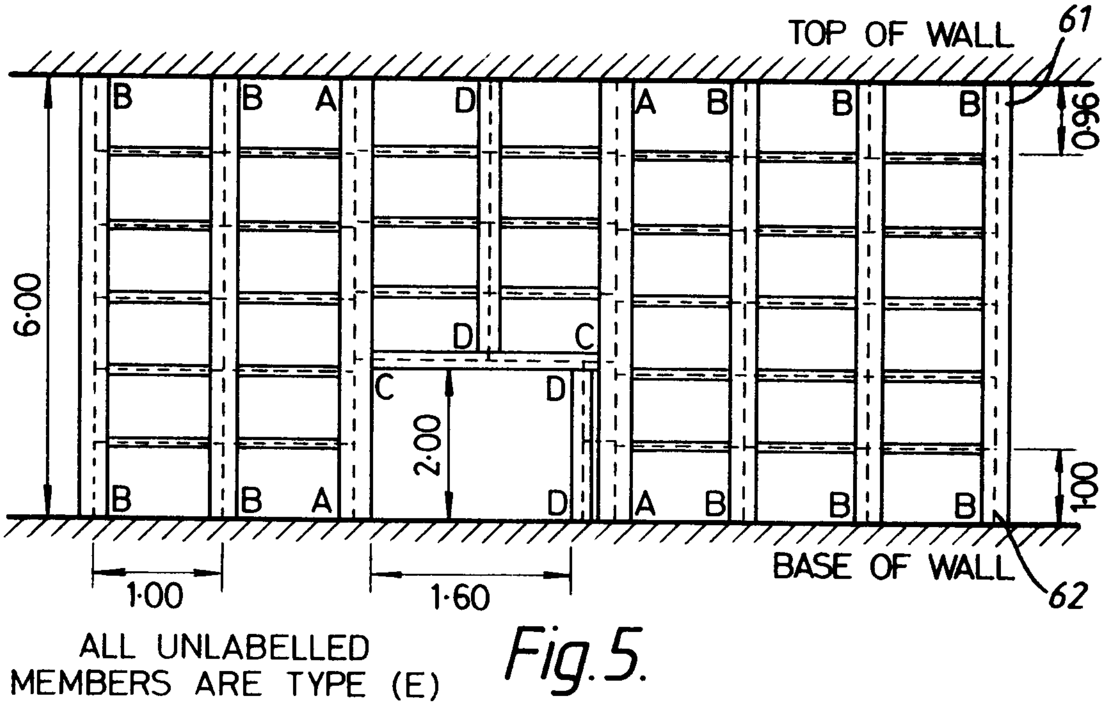
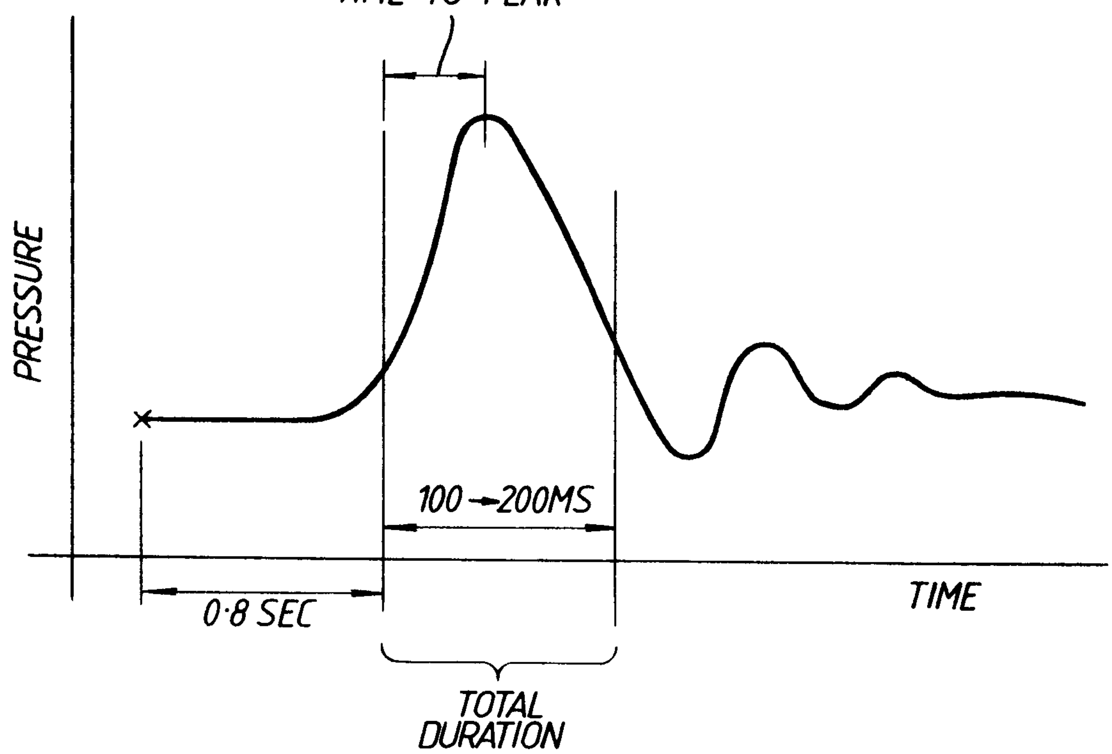

15 RUN ANALYSIS (SUBRUN)
16 VIEW RESULTS (RESULTS)
17 PRINT RESULTS (SUBPRINT)
18 SAVE FILE (SAVE)
19 EXIT PROGRAM (SUBEXIT)
26 CHECK DATA (CHDATA) (CHNGDAT)
27 NO RESULTS AVALIABLE
28 RESULTS AVALIABLE
29 RESULTS AVALIABLE
30 NO RESULTS AVALIABLE
31 SAVE DATA (SAVEDAT)
32 END PROGRAM
33 REJECT DATA
34 DESIGN (DESIGN)
35 CREATE A RESULTS FILE (SAVER)
36 PRINT A PRINT FILE (PRMENU) (PRSVFIL) (SAVEINF) (RETRVINP) (GRFI)
37 READ SECTION DATA
38 CHECK DIMENSIONS AND CAPACITIES
39 REJECT SECTION
40 CALCULATE LOADS
50 WALL SKETCH (GRAPH)
51 SUMMARY OF RESULTS (SUMMARY)
52 VIEW DETAILS (VDETAIL)
53 NOTES NOTES (PNOTES)
54 EXIT TO MENU (MENU)
55 VIEW DETAIL OF DIFFERENT SECTION (DESONE)
56 SAVE FOR PRINTING (SAVEPR)

Fig.4. (cont.)



TOP OF WALL
61
B
A
D
C
0·96
6·00
2·00
1·00
BASE OF WALL
1·00
1·60
62
ALL UNLABELLED
MEMBERS ARE TYPE (E)

Fig.5.



TIME TO PEAK
PRESSURE
100→200MS
0·8 SEC
TIME
TOTAL
DURATION

Fig.6.