THE DESIGN OF A TEMPORARY REFUGE TO RESIST EXTERNAL BLAST

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AUTHOR BIOGRAPHICAL NOTES

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ABSTRACT

The accommodation module of an offshore platform will usually form part of the temporary refuge (TR) for the platform. It must therefore provide protection against blast, fire, smoke and other associated hazards for up to an hour after a major accidental event [1].

The accommodation module will ideally be positioned away from any explosion source and will hence, at worst, be subjected to blast loading from a distant explosion which will typically consist of a steep sided pressure pulse of short duration. This short duration gives rise to the possibility of exploiting the compliance of the module on its mounts to give favourable dynamic characteristics and reduced response.

This paper describes this approach in the context of a recent major project where conventional, linear, elastic structural design software was used to produce a design capable of withstanding peak explosion loads of up to five times the static capacity.

1. INTRODUCTION

The structural design of an accommodation module with non-linear supports has been achieved by using a combination of simple linear elastic dynamic and static analyses.

The motivation behind the approach was that during the blast response the module will attain a maximum deflection where all the energy absorbed by the module takes the form of potential or stress energy. At this point the inertia forces are zero and the peak displacement obtained by a full dynamic response analysis may be reproduced by a similar static analysis giving the same overall level of displacement.

For the case of a short duration dynamic impulse load, the equivalent static load will have a similar distribution but is likely to be of a lower magnitude than the peak dynamic load. The ratio of the equivalent static load to the peak dynamic load is referred to as the dynamic amplification factor (DAF). This DAF was found to be less than one in all cases.

The design process not only involves the calculation of response but also involves member code checks which were not possible to perform within the dynamic response software. The code and joint checks were performed using the results of the equivalent static analysis.

A design based on linear elastic response was possible for this structure because of the nature of the loading and the structure's dynamic characteristics. It was not necessary to allow for plastic deformation, strain rate or strain hardening effects in this case. Member by member checks were, however, performed as a post-processing activity with modified load, dynamic 'yield' stress and member shape factors to estimate the capacity of the module with these effects taken into account. This was done for the purposes of the assessment of the robustness of the design for ALARP purposes.

The non-linear behaviour of the supports arises from the non-linear stiffness characteristics of the anti-vibration mounts whose main design function is to isolate the module from transmitted machinery vibration. The dynamic reaction loads were calculated iteratively. The reaction loads were then modified by the application of a point load time history at the module supports calculated using the actual stiffness characteristics of the support. An equal and opposite load was applied at the AVM connections to the deck to preserve global equilibrium.

The design method is discussed after a brief description of the some of the special features of external blast loading and response.

2. EXTERNAL BLAST LOADS

A number of references from the Nuclear and Defence industries [2,3,4,5] deal with the calculation of far field blast effects from nuclear and high explosive detonations. These techniques are not directly applicable to the calculation of overpressure loads in the near field of a hydrocarbon explosion, as the load duration in this case is typically much longer.

The far field of a hydrocarbon explosion may, however, be similar to that of a nuclear or high explosive event. This disturbance may result from the venting of a confined explosion or directly from an external explosion. The pressure disturbance generated by a hydrocarbon explosion will evolve into a blast or shock wave due to convection and non-linear wave interaction effects. This shock wave typically has a shorter duration (30 milliseconds) and a steep fronted profile. As the far field shock is propagating through air and does not directly involve combustion it will have a negative or suction phase with a similar impulse to the positive phase which precedes it. The whole shock wave time history will then have zero net impulse.

This has important consequences when considering global structural response. These considerations have given rise to the TNT equivalent method for estimation of blast overpressures at a distance.

The extent of the loading may also be limited. For a 30 millisecond, positive phase, disturbance travelling at just over the speed of sound, the shock wave will be about 10m thick. If the target structure is larger than this, the loading will be seen by the structure as a travelling line load with differing characteristics depending on the orientation of the exposed surface considered. The loading will naturally occur at different times on the walls and roof of the structure. In design, the instants of peak overturning and base shear may be considered separately. It is particularly important for the primary structure design to have a full definition of the loading magnitudes and sequence of loading on all exposed surfaces.

3. RECEIVED LOADS

The received loading on a flat surface from a sharp fronted blast wave is not equal to the incident pressure field as the reflected wave interacts with the original shock. There is also a convected flow associated with the shock referred to as the blast wind or dynamic pressure. The properties of the flow field such as shock velocity, peak wind velocity, density and dynamic pressure may be readily calculated from equations given in Reference 4. The received load for surfaces normal to the direction of travel of the shock wave may also be simply calculated. The received load on a surface will depend on the angle of incidence.

For strong shocks the received load may be as high as **eight times** the incident peak pressure, although in most offshore situations weak (acoustic) shocks would be generated with a received peak load of approximately twice the incident peak.

Simple methods for the calculation of the blast loads on a box shaped structures are available from the Reference 4. Methods of calculation for the other faces and also for partially open enclosures, lattice frames and cylinders are also given.

4. RESPONSE

4.1 Direct loading and frame action

The stresses in the structure of the module are from three sources:-

1. Direct loads from contact with the overpressure loading.
2. Indirect, inertia loads at connection points from enclosed accelerating masses.
3. Indirect loads transmitted through the structure from other directly and indirectly loaded components through frame action.

The cladding or panels forming the external surfaces will experience the first kind of load. The floors and ceilings within the module will be loaded by the second form of loading.

The primary framing structure will be stressed mainly by the indirect loading transmitted from the cladding supports and through 'frame action'. The main floor elements may also act as part of the main framing of the module.

The dynamic response of the cladding and panels will be determined by the natural period of the cladding elements themselves. The dynamic response of the primary framing structure will be determined by the natural periods of the frame in the global modes of response. These natural periods will be relatively long and the associated DAF's will hence be relatively small for the short duration loads present in external blast loading.

If the module is supported on anti-vibration mounts, the natural periods of the module in shear and rocking will be further increased giving DAF's for the support loads as low as 0.2. This represents an 80% reduction in effective loads as compared with the static peak values.

If rigid body motion is allowed, by allowing movement of the module supports, the stresses in the primary structure may be reduced further. This is especially true for blast loading from a distant explosion as the load time history is likely to have zero net impulse with the positive phase loading being counteracted by the suction, or negative phase of the blast wave.

4.2 Balanced and Out-of-balance Loads

The loading on the module may be separated into balanced and out-of-balance loads. The difference between the effect of these two types of loading on the primary structure of the module may be appreciated by consideration of Figures 1a and 1b.

Figure 1a illustrates the scenario of an out of balanced load. The car on the right is stationary and is impacted by the other car coming from the left. The effect of this impact is likely to be local damage to the rear of the target car, with the target car proceeding subsequently as a rigid body to the right. The impact force is limited by the friction applied by the brakes and the inertia of the target car. In the same way a module acted on by an out of balance load is likely to experience local damage on the directly loaded part of the structure. If it is allowed to move the stresses in the primary structure are limited by the support restraints and its own inertia.

As the global natural periods of the structure are large compared with the loading duration, the maximum deflection resulting from the positive phase of the loading will occur long after this phase of the loading has ceased. In the case where there is also a suction phase present, the suction will tend to reduce the maximum deflection as it will act before maximum deflections (and stresses) are reached.

The DAF's associated with out-of-balance loading are likely to be small being associated with global module movement.

Figure 1b illustrates the scenario of a balanced load. The target car is struck simultaneously by two cars travelling in opposite directions. The target car is likely to suffer extensive, local and global damage in this situation. The impact force is now dependent on the momentum or energy of the bodies involved. In the same way a module loaded in this way by a balanced load is likely to experience large stresses throughout the primary structure. As it is not possible to mobilise the inertia of the module as motion is limited the DAF's associated with this type of loading are likely to be near to unity and could reach 1.8 for very stiff structures.

It is clear that the main columns and supports for the module will be designed mainly by the out of balanced loads and the horizontal bracings and floors will be designed mainly by balanced components of the loading.

4.3 TR Module design philosophy

It is desirable that during the module dynamic response, the reaction/support loads should be as small and smoothly varying as possible and that the inertia forces from the movement of the module mass should be mobilised to resist the applied blast load directly at each level. These inertia forces only come into play when the module is accelerating away from the applied load (during the positive phase of the overpressure). Allowing larger rigid body movement limits the stresses in the primary steelwork and results ultimately in a more blast resistant structure with more reserves of strength.

The high level 'Performance Standards' ascribed to the TR module design were as follows:-

The guidance in the Safety Case Regulations[1] prescribes that an endurance time for the TR of at least one hour is required. Endurance includes consideration of the ingress of heat, fumes or gas, lack of oxygen, toxic fumes generated internally and the effects of internal fires.

The external module cladding is rated to meet well defined jet fire conditions. Where these walls are subject to blast, they are required to meet these requirements after blast. If this is not practicable for all scenarios, then an upper bound impairment frequency of 10^{-3} per annum should be adopted.

The primary steelwork is allowed to deform plastically so long as the integrity of the enclosure is not compromised with respect to subsequent fire impingement.

The cladding should be able to prevent the ingress of toxic gases or smoke after the explosion.

The personnel within the TR should be able to gain access to the primary means of escape.

4.4 Choice of Analysis Approach

Two approaches are available for the calculation of the global response and for design of the primary and support steelwork. They are, the Biggs' one degree of freedom method [6] and the 'Direct Approach' using a time-stepping, dynamic structural analysis.

The direct approach was used in this case because:-

1. The load cases are very severe and so any unnecessary conservatisms should be avoided.

2. Combination load cases exist with simultaneous loads on two faces. No single mode has been identified which dominates.

3. At levels of overpressure in the region of two bar, response of single walls out of plane is significant and this support steelwork needs to be checked separately.

4. Full advantage should be taken of the suction phase of the loading in reducing the peak response, this is more easily done using the Direct Approach.

5. Some of the load cases are represented by localised, non-uniform loading on some of the walls, this requires use of the Direct Approach..

5. DYNAMIC AMPLIFICATION FACTORS

A number of dynamic simulations were performed to determine the DAF's to be applied for the components of the blast load broken down by loads on each wall and separated into balanced and out of balance loads.

Three sets of DAF's were derived to characterise the response of the cabins module.

The first set associated with the AVM's, reaction forces on the deck and connecting structure close to the AVM supports. The characteristic displacements for these regions are the AVM extensions or relative displacements between the top and bottom points of the AVM's. These relative displacements give a measure of the level of load present in the AVM's and local structure indicated as X^d_a in Figure 2(a) with the dynamic load time history L_d peaking at Pmax. For simplicity the part of the model representing the deck and the displacements of the lower point of the AVM's are not shown. The corresponding displacement from the static model X_a^s is shown in Figure 2(b) with a static load of L_s bar.

The ratio X^d_a/X^s_a is a measure of the reduction in extension of the AVM's due to dynamic effects and defines the equivalent static load L_e^s which will give the same general level of stresses as the dynamic load time history L_d for this part of the structure.

Hence a DAF may be defined by:-

$$DAF = X^d_a/X^s_a \quad \text{and} \quad L_e^s = DAF \times L_s$$

This equivalent static load L_e^s is then applied to the static model in order to perform the code checks on the lower part of the structure. The DAF for the AVM's and local structure was found to be less than 0.2 which was confirmed by hand calculations.

For consistency between the dynamic and static analyses, all displacements quoted from the equivalent static analysis are for the blast load case only and do not include self weight.

The second set of DAF's is associated with the global flexing of the structure in a mode similar to a cantilever or shear beam mode of response.

The displacements of the top corners of the module X^d_{total} on the North and South faces are decomposed into two components as shown in Figure 3 for the dynamic analysis. The rigid body component X^d_a and the elastic component X^d_{e}, where $X^d_{total} = X^d_r + X^d_e$.

The rigid body displacement of the module X^d_r does not contribute appreciably to the local stresses in the module primary structure except at the supports. The inertia forces from internal masses are however taken into acount in the dynamic elastic deflection component.

The effective static load L_e^s, and the DAF for the upper part of the structure is then defined by:-

$$L_e^s = X^d_e/X^s_e \text{ x } Ls = X^d_e/X^s_e \text{ x } Pmax$$

Hence a DAF for the upper part of the structure may be defined by:-

$$DAF = X^d_e/X^s_e \quad \text{and} \qquad L_e = DAF \text{ x } L_s$$

This second set of DAF's were found to be larger than the first set because of the larger stiffnesses associated with elastic deformation. Typical values for the second component of the blast response were less than 0.4.

As there was a possibility that the shape of deformation under dynamic loading differed from that under the same static loading, a third, intermediate set of DAF's was also developed to investigate the local deflection behaviour of the columns between intermediate levels.

However consideration of the more realistic DAF's calculated using realistic non-linear stiffness data for the AVM's were found to be smaller than those used for design. This topic is discussed in the next Section.

The balanced component of loading gave rise to a further DAF associated with the balanced loading. A DAF of one was conservatively used for this part of the loading. For the same reasons vertical loads were applied at their full peak values.

These DAF's were then used to factor the component static loads which make up the final load combinations.

5. TREATMENT OF NON-LINEAR RESTRAINTS

The relative deflections of the AVM upper and lower reference points from the linear dynamic analyses were compared with the ability of the AVM's to sustain these displacements and the applicability of the assumed AVM stiffnesses was assessed for the displacements calculated.

As the stiffness/deflection characteristics were found to be non-linear in shear for the calculated displacements, a number of supplementary analyses were found to be necessary

The load deflection characteristics of the AVM's in the vertical and horizontal directions were obtained. The restoring forces at the deflections during the response were calculated from these curves. The time history of the difference between the calculated reaction loads and the reaction loads assuming constant stiffness was then determined. This reaction load difference was then applied as a force at the top point of each AVM in the computer model. An equal and opposite force was applied at the bottom point of each AVM to maintain global equilibrium.

The dynamic response model was then re-run to check consistency of the resulting deflections and the process repeated if necessary.

6 ALARP INVESTIGATION

As part of the Health and Safety Executive's requirement for the demonstration of ALARP (probability of loss of life As Low As Reasonably Practicable) we were required to perform a number of analyses for scenarios outside the design envelope to demonstrate robustness and 'defence in depth' against failure.

The analyses took account of reserves of strength available from the following:-

- The consideration of plastic hinge capacity as opposed to first yield elastic bending moment capacity member by member (without consideration of load re-distribution between members). This gave a bending moment capacity increased by the shape factor of the member considered.

- Strain rate yield enhancement effects. An increase of 20% on the code based yield strength may usually be attributed to strain rate effects.

- Strain hardening enhancement effects. For large deflections of ductile components, a further enhancement factor of 20% may be applied to the code based material yield strength.

Together these last two factors indicate an increase in the capacity to sustain blast loads by a factor of 1.2 x 1.2 = 1.44.

The joints may not perform so well, being relatively brittle details. If the joints are not loaded to their full capacity this would not be a problem.

Other factors could also lead to additional reserve capacity in the structure, however taking these into account would have involved substantial extra effort. These included:-

- Full plastic hinge analysis with load re-distribution.

- Tension restraint from the cladding

- Internal restraint from the architectural contents of the module.

- Relaxed performance standards for more extreme less likely events.

6. CONCLUSIONS

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7. REFERENCES

1. 'A guide to the Offshore Installations (Safety Case) Regulations 1992 - Guidance on Regulations L30', Health and Safety Executive, HMSO 1992.

2. 'Design of Structures to Resist the Effects of Atomic Weapons', U.S. Army Corps of Engineers Manual, EM 1110-345-415., ASCE 42, 1957.

3. 'The safe use of explosives underwater', MTD, 1997.

4. JD. McCaughey, ‘Damage from explosions - Real or imaginary’, The Structural Engineer, Vol 62A, No. 5, May 1984, pp151 -158.

5. S. Glasstone and PJ. Dolan, ‘The effects of nuclear Weapons’, 3rd Edn., Castle House Publications Ltd., 1980.

6. J.M. Biggs, 'Introduction to Structural Dynamics', 1964, McGraw Hill.

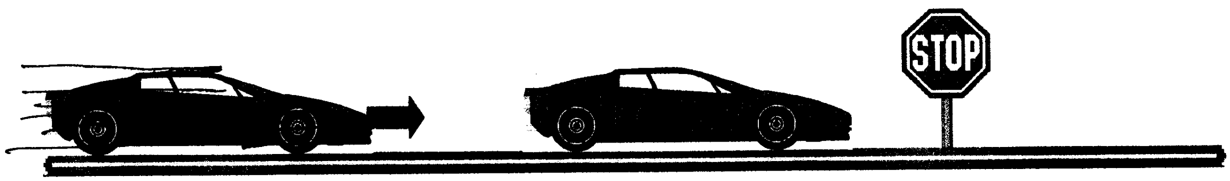


Figure 1a Scenario 1: Out-of-balance loads

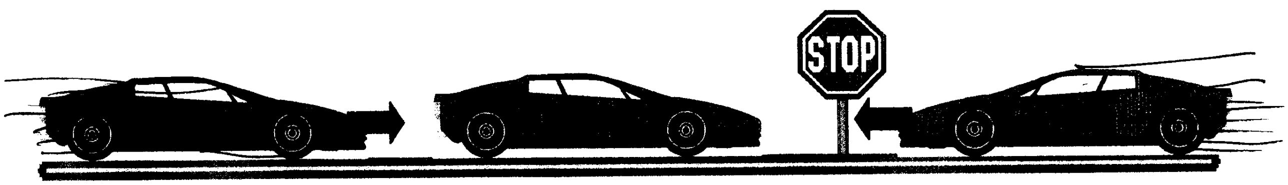


Figure 1b Scenario 2: Balanced loads

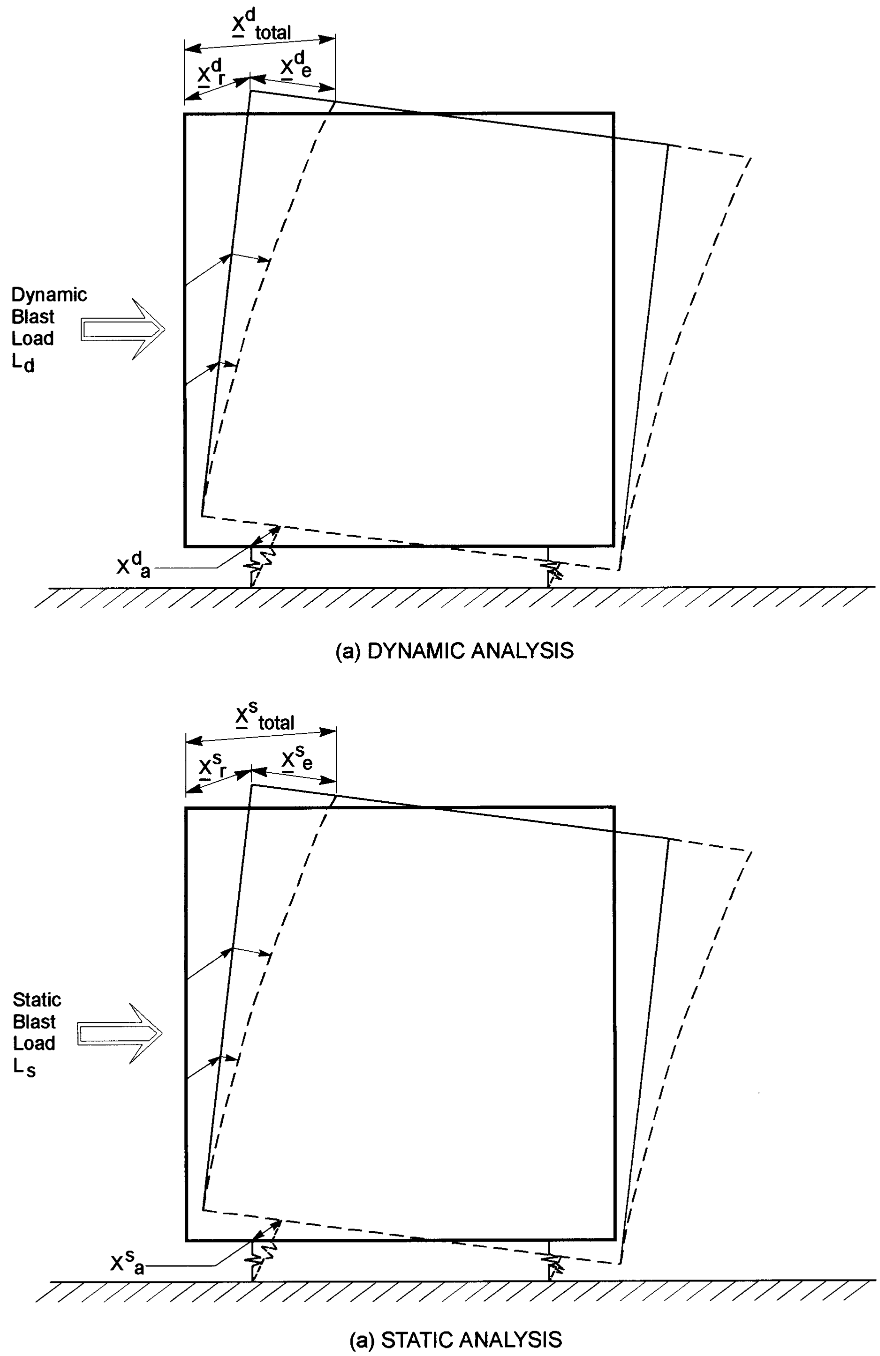
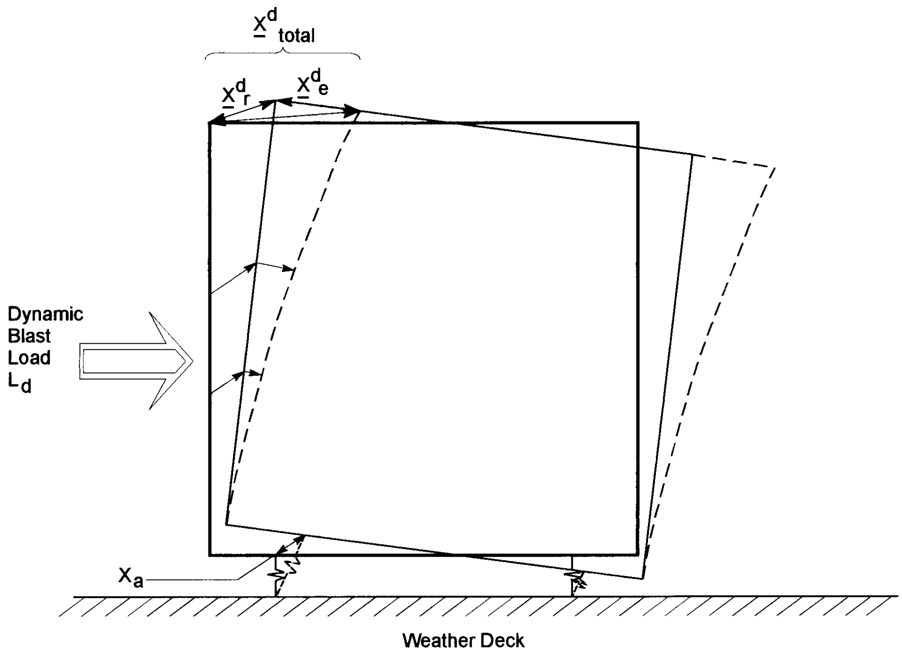


Figure 2 Definition of equivalent static analysis



$\underline{x}^d_{total} = \underline{x}^d_r + \underline{x}^d_e$

Total = rigid body + elastic

Figure 3 Decomposition of dynamic response displacements