SIMPLIFIED APPROACHES TO FIRE AND EXPLOSION ENGINEERING

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AUTHOR BIOGRAPHICAL NOTES

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Vincent Tam graduated from Imperial College, University of London in 1973 and obtained his PhD in 1979. He has been working in safety technology and consequence modelling techniques since 1981. Over the past twelve years, he, in various capacities, has directed and carried out gas explosion research projects, and worked with BP design projects and existing assets on gas explosion issues, e.g. modelling, control and mitigation.

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ABSTRACT

Recent and proposed large developments in deep water and the use of hub platforms with high throughputs in the Gulf of Mexico have raised awareness of the profile of fire and explosion safety assessments for new and existing installations.

In response to these developments BP has developed new guidelines in Fire and Explosion Engineering described in this paper. The guidelines give useful, practical advice for the Structural and Process engineering disciplines at a level appropriate for direct use in a project environment.

In view of the maturity of the subject matter the focus is on code based and simpler methods of analysis. The approach is compatible with API RP2A and recommends two levels of explosion design overpressures by analogy with earthquake assessment representing ‘Design Level’ and ‘Ductility Level’ events. The ‘Design Level’ explosion tests the installation against more severe performance standards appropriate to the higher frequency events.

Three levels of analysis are defined for use in fire and explosion assessment. These are, 'Screening Analysis', linear elastic 'Design Level' analysis and non-linear 'Ultimate Strength' analysis. Acceptable inputs to these levels of analysis are defined.

The emphasis of this paper is on simple linear analysis methods for the evaluation of a structure during the first level of assessment.

The Guidance is to be used as a basis document for the new proposed API RP on Fire and Blast.

1. INTRODUCTION

The Guidance covers assessment, risk classification, fire loads and response, explosion loads and response, analysis methods and the interpretation of the results of fire and blast consequence analyses. Combined Fire and Explosion load cases are also considered.

The Installation risk classification method follows an approach based on API RP2A WSD 21st edition(1).

Assembly of the document involved no new research but involved the documentation of published methods and advances. It was found that there are a number of areas which were not covered in the Interim Guidance Notes(3), these have now been included in the Guidance. These include:

? Interpretation of code checks in a design level analysis to take account of reserves of strength.
? Interpretation of Ultimate strength analysis results
? Dynamic pressure – drag load estimation
? Confined and ventilation limited Fires
? Piping response to drag/dynamic pressure/blast wind
? Simple methods of the calculation of Blast wave loading on adjacent structures.
? Tension effects in panels and Biggs' method.
? The Dimensioning explosion – the use of elastic design analyses for extreme events as a conventional design load case.
? Good structural details for blast resistance.
? Floating structures issues

2. API INSTALLATION CLASSIFICATION FOR FIRE AND EXPLOSION ASSESSMENT

This Section is a brief overview of the assessment process described in detail in References 1 and 3. In the context of this Section 'Failure' may be identified with the release and ignition of hydrocarbon. The Category descriptions are simplified for brevity, full definitions for each category may be found in References 1 and 3.

The 'Risk Assessment' process is illustrated in the flowchart shown in Figure 1, which is a modified version of that given in API(1). This consists of a series of tasks to be performed to identify installations at significant risk from fire or explosion events. The large majority of existing small, normally unmanned installations are expected to be assigned Risk Level 3. Structures that are identified as falling into higher risk categories will need to be assessed on a case-by-case basis.

Platform exposure categories are assigned depending on the manning and possibilities of evacuation.

Life Safety Categories

Categories for life-safety are:

L – 1 Manned non-evacuated
These must have provision for Temporary Refuge (formerly API 'safe muster areas')

L – 2 Manned evacuated
This classification is not likely for an explosion event as no warning is given but could apply for a fire event – especially if a TR is provided

L – 3 Unmanned
A likely category, but visits to platform for maintenance/modification are the most likely times for Hydrocarbon releases.

Consequences of Failure

Categories for consequences of failure are:-

L-1 High consequence
Major installations and/or those which have the potential for well flow of either oil or gas in the event of failure.

L-2 Medium Consequence
Platforms where production would be shut in during the design event. All wells with fully functional SSSV's.

L-3 Low consequence
Platforms where production would be shut in during the design event. All wells with fully functional SSSV's. Platforms in this category are in depths less than 100 feet and have less than 5 well completions.

Consequences of failure are 'potential' and express the vulnerability of the installation to the whole range of possible release scenarios.

The Life Safety Categories are combined with the Consequence of Failure Categories to give an overall Exposure Category (L-1, L-2 or L-3) for the installation. The final Exposure Category is the more restrictive level derived for Life Safety and Consequences of failure.

At this stage, no scenario related consequences are included - only the probability of occurrence is estimated from indicators. A probability of occurrence of fire and explosion events is then assigned to the installation based on the probability of release, ignition and escalation potential. A number of indicators are used to assign High, Medium or Low values to this parameter.

Indicators given in Reference 3 include:-

? Inventory
? Equipment type, complexity, piping and valves. The material integrity and the margins against pressure variations are also considered.
? Risers and wells

- Ignition sources
- Type of operations
- Production operations
- Deck type
- Location
- Whether Inherently safer design principles have been applied
- Age, re-supply frequency, maintenance and safety management

Low, Medium and High values are associated with the values of these indicators. The probability of occurrence is then combined with the Exposure Category to assign a risk level to the installation following the Risk Level Table below which is a modified version of that given in Reference 1 for Metocean and Seismic assessment.

Table 1 Risk Matrix for Fire and Explosion Events

Probability of failure (hydrocarbon release and ignition)	**High (design level)**	Risk Level 1	Risk Level 1	Risk Level 2
	Medium	Risk Level 1	Risk Level 2	Risk Level 3
	Low (ductility level)	Risk Level 1	Risk Level 3	Risk Level 3
		L – 1	**L – 2**	**L - 3**
		Installation Exposure Category		

The entry for the low probability event and exposure category L-1 (the highest level) has been changed to Risk Level 1 to reflect the severity of explosion events at this level of probability.

This matrix is appropriate for fire events but may need modification to reflect the fact that full evacuation of all personnel may not be possible during and after severe explosion events.

For those installations considered to be at high risk for a fire or explosion events (Risk Levels 1 or 2), it is necessary to complete the detailed structural integrity assessment for fire and explosion events described in detail in the Guidance document(2).

3. STRUCTURAL ASSESSMENT - LEVELS OF ANALYSIS

A Structural Assessment may be performed at three levels of increasing complexity starting with a 'Screening Analysis' then a 'Design level analysis' possibly followed by an 'Ultimate Strength Analysis'. If the 'Ultimate Strength Analysis' indicates failure then further mitigation measures are required. An Ultimate Strength analysis will often be required to examine blast and fire barriers and their connections.

'Failure' in the context of fire and explosion structural assessment can be interpreted to mean failure to satisfy the minimum performance standards for the installation. Example minimum performance standards are given below.

Performance Standards

- In the case of a fire event, it is required that at least one escape route must remain available during and after the event.
- In the case of an explosion event at least one escape route must be available after the event for all survivors. For a manned platform a Temporary Refuge (TR) or Safe Mustering Area(1) must be

available to protect those not in the immediate vicinity of an explosion and to survive the event without injury.

The transfer of conclusions and load characteristics from the analysis of a similar platform is acceptable for all levels of analysis. The nomination of a typical installation to represent a fleet of platforms is acceptable.

3.1 Screening Analysis

Screening analysis for an existing installation consists of condition assessment which may involve a survey(1) followed by design basis checks.

Design basis checks consist of determining whether the methods used for the design are acceptable in the context of the fire and explosion events considered.

3.2 Design level analysis

Design level analyses are conventional linear elastic analyses as used in design. The loads used in such an analysis are in a similar form to conventional environmental, live and dead loads. This gives the advantages of ease of use and interpretation. Such analyses fit well within a project environment and give outputs which can be used directly in conventional code checks.

This level of analysis is scenario based with mitigation already introduced and examines the residual events which cannot be eliminated or prevented.

Alternatively a ‘Dimensioning Explosion’ overpressure may be derived from the Ductility Level explosion to represent these effects with the normal limits on acceptable utilization factors for the members. These methods are discussed in detail in Section 4.4.

3.3 Ultimate Strength analysis

An ultimate strength analysis may be required for Risk level 1 or 2 installations. This method of analysis can take into account the load re-distribution, the sequence and timing of member failures.

Explosion load cases and appropriate analysis levels are given in Tables 2 and 3 below and illustrated in Figure 2.

Table 2 Appropriate Method of analysis – explosions

Risk Level	Analysis method	Load Calculation	Response Calculation
Risk level 1 Or Risk level 2	1. Screening analysis	Design level and Dimensioning explosion from nominal overpressures	Design basis checks
	2. Design level analysis	Design level and Dimensioning explosion	Design level analysis
	3. Ultimate strength analysis	Design level and Ductility level explosions from CFD simulation	Ultimate strength analysis
Risk level 3	None required	None required	None required

Table 3 Appropriate Method of analysis – fires

Risk Level	Analysis method	Load Calculation	Response Calculation
Risk level 1 **Or** **Risk level 2**	**1. Screening analysis**	**Allowable temperature (yield strength reduction of 60%)**	**Design basis checks**
	2. Design level analysis	**Calculate peak temperature member by member.**	**Design level analysis**
	3. Ultimate strength analysis	**Calculate temperature - time history of primary members**	**Ultimate strength analysis**
Risk level 3	**None required**	**None required**	**None required**

4. LOAD CASES FOR EXPLOSION ASSESSMENT

For explosion structural assessment, the risk matrix described in Section 2 should be constructed to identify the risk level for at least 2 levels of explosion severity.

The return period for an explosion event may be calculated directly from the probability of occurrence of the event or from the installation's PLL (probability of loss of life per annum) and the associated fatality levels.

4.1 The Design Level explosion

The Design Level explosion is a relatively high probability low consequence case resulting in elastic response of the primary structure, with the essential safety systems remaining functional. This case provides a measure of asset protection.

A number of cumulative overpressure probability distributions have been published for the expected peak overpressure in particular situations giving the probability of occurrence of a particular level of peak overpressure(7). These probability distributions represent conditional probabilities of overpressures given an explosion occurs. Return periods based on these distributions are hence relative. The probability distributions have been found to fit the exponential distribution which has a mean value equal to its standard deviation. This illustrates the magnitude of the uncertainty in the calculation of peak pressures.

An example of the cumulative probability distribution function for peak overpressure is shown in Figure 3 for a release at a particular location. This cumulative distribution function is fitted to the data given in Reference 7 and represents a mean peak pressure of about 0.7 bar.

The choice of a 'Design Overpressure' peak pressure is complicated by the fact that any explosion is an accidental event with a probability of occurrence depending on module geometry, inventory, safety systems and whether Inherently Safer Design principles have been applied.

It has been proposed(6) that a Design Overpressure is chosen using the most extreme credible event overpressure. It is likely that 'bounding cases' will all lie close to this value with respect to probabilities and lie to the right of Figure 3. Allowance must be made for the method used to calculate this extreme value. Most recent practice involves the use of a dispersion analysis to model a realistic release and ignition probability. It was suggested that Design events may then be derived by considering the overpressure corresponding to overpressures with return periods one or two orders of magnitude lower than the extreme event or probabilities one or two orders of magnitude higher.

Examination of the data given here would suggest that an event one order of magnitude more frequent than the extreme event would give a reasonable value for the Design level explosion overpressure for design using elastic methods.

It is envisaged that the ‘Dimensioning Explosion’ described in Section 4.2 and the ‘Design level’ explosion will be of similar magnitude in most situations.

For small installations at low risk it may be suitable to assign a generic minimum value for design of the order of 1 bar overpressure. Most structures will resist this loading with minimal modification of connection details. This topic is still under investigation.

4.2 The Ductility Level explosion

The Ductility Level explosion is a low probability, high consequence case retaining TR integrity and with escape possible during and after the event.

Ultimate Strength analysis for the Ductility Level Explosion may be replaced by a Design Level Analysis so long as:

? A Design Level overpressure (or Dimensioning Explosion) is derived from the Ductility Level Explosion overpressure by rigorous means (Safety factor removal and reserve strength inclusion) an dynamic effects are included in some way.

OR

? Code checks may be re-interpreted to take account of the inherent reserves of strength, available from both material effects and plastic deformation.

Fire and Explosion barriers and their connections must be checked using the full overpressure for the Ductility Level explosion.

5. EXPLOSION RESPONSE OF PRIMARY STEELWORK

The assessment of primary steelwork explosion capacity differs in four major respects from the blast assessment of free-standing blast walls:

? Loads are transferred between walls, decks and equipment. Membrane and tension loads may be applied from loaded, connected surfaces.

? Out-of-plane static loads will be induced in decks due to the presence of equipment and piping.

? Mainly drag (dynamic pressure) loads act on isolated columns and beams.

? The spacial variation of load is not negligible giving a ‘scale factor’ on the applied peak overpressure to be applied for design.

A dynamic frame structural model will represent these effects.

5.1 Modified member code checks

Member code checks may be re-interpreted to take account of the following inherent reserves of strength.

1. The explosion event is an accidental event and hence the stress may be allowed to approach yield. A factor of 1/0.66666 (1.5) is then appropriate on the allowable utilization.
2. The material strain rate effect will generally give an increase of yield stress of the order of 20%, this value is used in the nuclear industry$^{(5)}$.
3. Strain hardening will occur around regions of local plasticity. Where it is appropriate (for tension members, plastic sections in compression and/or bending) allowance may be made for this by taking the design yield strength as the ultimate tensile strength divided by 1.25 (Reference 3 Section 3.5.8).
4. The occurrence of plastic hinging may be taken into account by factoring the acceptable utilization factor by the ratio of the plastic 'Zx' to elastic section modulus 'Sx'. The member must be able to sustain the formation of a plastic hinge before buckling.

Shear checks should also be made using the correct dynamic reaction loads with strains being limited to elastic limits.

Taking into account all the factors above gives a possible acceptable utilization factor of 1.5 x 1.20 x 1.25 x 1.1 (a factor of 2.5) for a tension member. A factor of 2.0 is acceptable for Primary members so long as the member does not buckle where local buckling is not acceptable. Usually the benefit of both strain hardening and strain rate yield strength enhancement is not taken into account.

5.2 Global Loading scale effects

It is quite clear that the Spadeadam test rig which has been subjected to local overpressures over 10 bar, was not designed for, but has withstood these pressures with minimal damage.

The simplest factor which affects global primary structure response is the scale factor or coherence factor. The following cases are given only as an illustration of the scale effect. The precise geometry of the target structure and the loading pattern will change these conclusions and a detailed assessment of these factors should be made depending on the scenario considered.

The pressure load duration 't_d' for a typical hydrocarbon explosion is of the order of 50 milliseconds (0.05s). Longer durations are common in the early stages but generally a shorter duration is associated with a higher overpressure pulse later in the evolution of the explosion.

Considering the situation shown in Figure 4 for a pressure disturbance travelling along a wall or for a ridge of pressure travelling across a deck. The pressure disturbance is considered to be a pulse travelling with a velocity of about 340 m/s (the speed of sound C_0 in ambient conditions, neglecting convection with the gas flow) giving a length scale of the disturbance '$l_d = t_d \times C_0$ of about 17m. A typical bracing member or panel of length 'l' equal to 8m will hence see a peak averaged pressure of 0.75 of the peak. A module wall of length 'L' equal to 35m will see a peak averaged pressure of 0.25 of the peak value.

This indicates a scale factor of 1/3 on the global load/member load for this situation. In situations where a large deck is considered these conclusions may only apply locally as the pressure loading pattern may be circular or irregular in shape.

Figure 5 shows the variation of this scale parameter with variations of the ratio l/l_d. Only grazing incidence is considered above for application in deck load calculations (angle of incidence = 90 ?). The other curves shown in the figure which lie above this are calculated for angles of incidence down to zero (normal incidence). Reflection and possible ‘pressure doubling’ are not taken into account in these curves.

5.3 Global Received loads

Global loads on primary members are also dependent on the capacities and dynamic properties of the connected panels. Often the loads transmitted into the primary framing will be reduced by panel capacities and the delay in load transmission due to delayed panel response. The panel peak response may well occur long after the load has subsided and if a suction phase is present then the panel response itself may be reduced.

Often the direction of the loading on the primary framing may be changed locally due to panel membrane effects. These membrane loads may be balanced globally if panels of similar dimensions are attached to either side of the columns.

5.4 Plasticity and dynamic effects

The Biggs chart shown in Figure 6 may be used to estimate the benefit to member capacity resulting from plastic deformation and dynamic effects (within the constraints that the chart is applicable to the structure and loading considered).

This Biggs chart reflects the response of a structure idealised as a one degree of freedom system with natural period T under a triangular load with peak ‘F_1’ and duration ‘t_d’. The horizontal axis is the ratio of load duration to natural period and the vertical axis is the ductility which simplistically is the peak deflection divided by the deflection at effective first yield(2,3) . Each curve is for a different ratio of (plastic) resistance R_m to peak load F_1.

If a member has a t_d/T ratio of 0.2 for example, and a ductility of unity is required (elastic response) then the resistance to load ratio is about 0.6 and the member will resist (dynamically) a load of 1/0.6 = 1.6666 times the member resistance. For elastic response the dynamic amplification factor for this case is 0.6.

If a ductility of 10 is allowed then the resistance to load ratio is about 0.15 and the member will resist a load of 6.66 times the member resistance. Hence the ratio of the capacity allowing a ductility of 10 to the elastic capacity is 6.66/1.66 or a ratio of 4 for structures with a load to natural period ratio of 0.2.

This capacity ratio curves for allowed ductility levels of 5 and 10 are shown in Figure 7. Note the logarithmic scale. Values above 4 are reached for structures with long natural periods (large frame structures or compliant structures). The ripples on the curve are a direct result of the variations present in the original Biggs curves.

The allowed ductility will depend on the applicable performance standards. A ‘Ductility Level’ explosion event performance standard could well allow a ductility level of 10 for decks and beams. The ‘Design level’ event performance standard will only allow a ductility level of 1 with elastic response required.

6. DERIVATION OF THE DIMENSIONING EXPLOSION OVERPRESSURE

In this Section methods are discussed which enable Ultimate Strength Analyses to be performed using conventional linear elastic methods.

Dimensioning explosion loads are of such a magnitude that when they are applied to a simple elastic analysis model with conventional code checks for an accidental load case, result in members dimensioned to resist the worst case credible event or 'Ductility Level' Explosion.

The definition of Dimensioning-explosion overpressure Q_{dim} is based on a simulated overpressure for the Ductility Level explosion Q_{duct}. This overpressure should represent those values generally indicated by simulation to be applied to a substantial proportion of the structure.

6.1 The LRFD approach

Following the LRFD approach discussed in Reference 7, the balance of load and resistance for a component in an Ultimate Strength analysis may be represented by the equation:

$$X_s X_m ? ?_m R_{el} = X_q ? ?_e Q_{duct}$$

Where:

X_s	is a factor accounting for system strength and redundancy – this cannot be assumed to be the same for elastic and Ultimate strength analyses as elastic response utilises all members whereas Ultimate Strength analysis may result in local failures. If a frame analysis is performed the effect of this factor will be included by default and the value may be set to unity.
X_m	is the modeling uncertainty parameter – this parameter is expected to increase with the general level of overpressure.
$?_{m,}$ $?_e$	are the partial resistance and load factors corresponding to the component under consideration.
?	indicates a product of the partial safety factors
X_q	is the bias on loading.

And

R_{el} is the component design strength as given by standard formulae in the codes.

The factors will be scenario dependent as they reflect the location, extent and severity of the scenario considered.

A conventional Design level analysis limit state may be represented by the equation:

$$R_{el} = Q_{dim}$$

(Some formulations have further load and resistance partial factors associated with accidental load case elastic response which are omitted here for clarity).

The R_{el} in both equations may be equated for the Dimensioning explosion overpressure, hence formally:

$$Q_{dim} = X_q ? ?_e Q_{duct} / (X_s X_m ? ?_m)$$

This gives the design overpressure for the Dimensioning explosion in terms of Q_{duct} the overpressure level derived by simulation for the Ductility Level explosion. Values for the factors referred to above are given in Reference 7.

6.2 Assessment of reserves of strength

As a specific case, consider the limit state equation for a Ductility level event with no safety factors included.

$R_{duct} = Q_{duct}$

From Section 4.3

R_{duct} = Strain rate factor x Shape factor x Scale factor x R_{el}

Or alternatively including dynamics with a, well defined, generally acceptable allowed ductility:-

R_{duct} = Ductility factor x Strain rate factor x Scale factor x R_{el}

Appropriate values for the Strain rate factor are 1.1 to 1.27 from Reference 7.

For the elastic limit state under the dimensioning explosion overpressure.

$R_{el} = Q_{dim}$

Hence

$Q_{dim} = Q_{duct}$/(Ductility factor x Strain rate factor x Scale factor)

The yield stress used for the calculation of R_{el} should be the recommended value used in Design which is somewhat higher than the mean value. The stress should be allowed to approach this yield stress locally without the application of any factors, as befits an accidental load case.

The Ductility factor may be read from Figure 7 and the Scale factor from Figure 5.

The Ductility factor used here is strictly only applicable to members and structures which can be represented as one degree of freedom systems. This restriction may be relaxed. The inherent redundancy in the structure is not represented but would be partially represented in the elastic frame analysis used to determine response. Factors representing uncertainty are not included but may be applied to Q_{dim} directly as in the LRFD approach.

Data already exists to ‘benchmark’ the above approach by direct comparison of the results of a Design Level analysis using the Dimensioning Explosion with an Ultimate Strength analysis using the Ductility Level explosion overpressure for a real topside design. This may put into context some of the theoretical objections which could be raised to discourage the application of the techniques described in this paper. If the methods gain acceptance then a consensus for the factors to be used will make the benefits available throughout the industry.

7. CONCLUSIONS

? BP has developed new guidance on Fire and Explosion engineering giving useful advice for project use.

? The API risk classification method has been applied to Fire and Explosion engineering.

? Two levels of explosion loading are suggested for explosion assessment by analogy with the earthquake assessment.

? Factors are identified representing the reserves of strength mobilized in dynamic plastic response. This enables a method for determining representative explosion overpressures which can be applied to a conventional elastic structural model and result in members sized to resist the Ductility level explosion.

? A ‘Scale Factor’ is defined which takes account of the finite extent of explosion loads.

8. REFERENCES

1. ‘Recommended Practice for the planning, Designing and Construction Fixed Offshore Platforms – Working Stress Design’, API RP 2A-WSD, 21st Edition. API (American Petroleum Institute).

2. ‘Guidance for the protection of offshore structures against fires and explosions’, Ref. CH152R002 API Draft 0, November 2001.

3. ‘Interim Guidance Notes for the Design and Protection of Topside Structures Against Explosion and Fire’, Various Authors, Joint Industry Project on Blast and Fire Engineering for Topside Structures, November 1992.

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

5. ‘Structural Analysis and Design of Nuclear Plant Facilities’, ASCE 58, 1980.

6. Corr R.B., Tam V.H.Y., Snell R.O. and Frieze P.A., ‘Development of the limit state approach for design of offshore platforms’, Era Conference ‘Safety on offshore installations’, 99-00808, 1999.

7. R.L. Bruce, 'Blast overpressure prediction - modeling the uncertainties', ERA Conference, ‘Offshore Structural Design – Hazards, Safety and Engineering’, November 1994, ISBN 0 7008 0587 7.

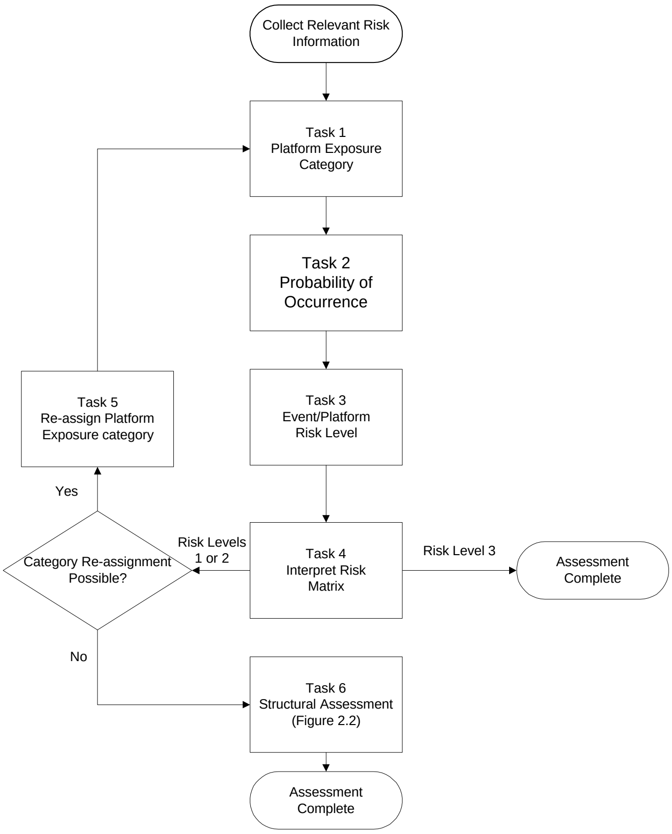
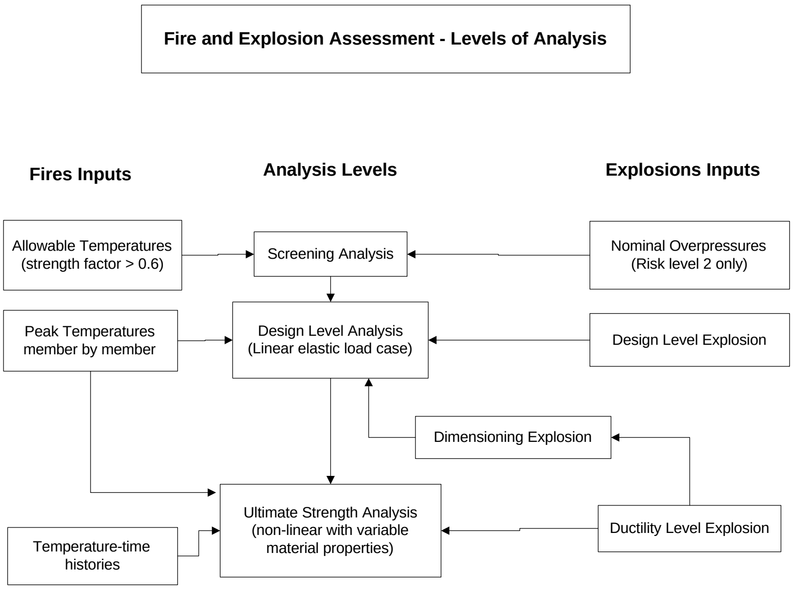


Figure 2.1 Overview of the Assessment Process

Figure 1 Overview of the Assessment Process



n.b. Fire and Explosion barriers and their connections must be checked using the full overpressure for the Ductility Level explosion

Figure 2 Levels of Analysis and Inputs

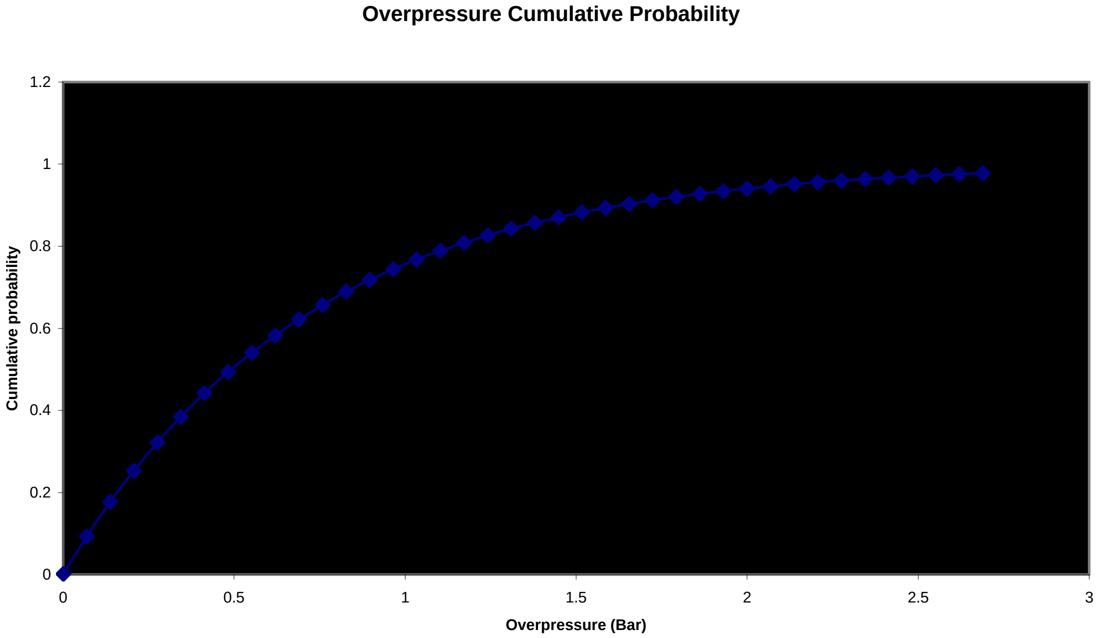


Figure 3 Example Overpressure Cumulative Probability (from Reference7)

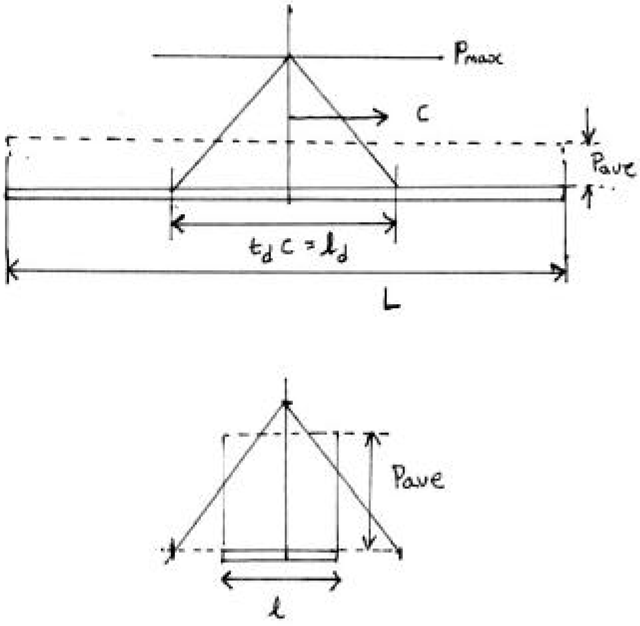


Figure 4 Explosion overpressure load as a travelling pulse

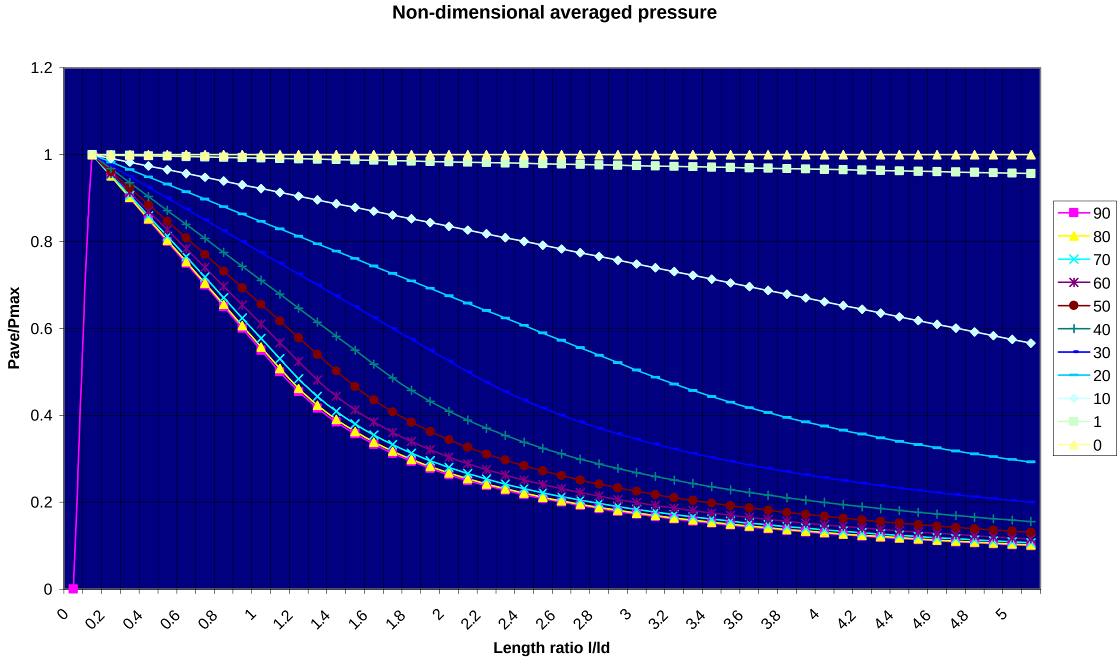


Figure 5 Variation of Scale Factor with load to target length ratio

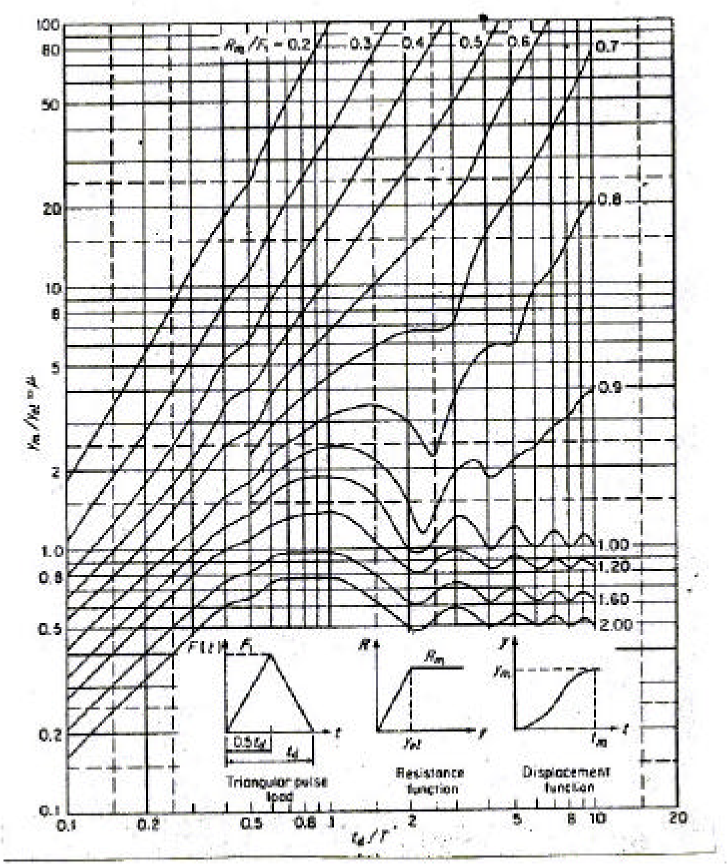


Figure 6 Biggs' Member response chart from Reference 4

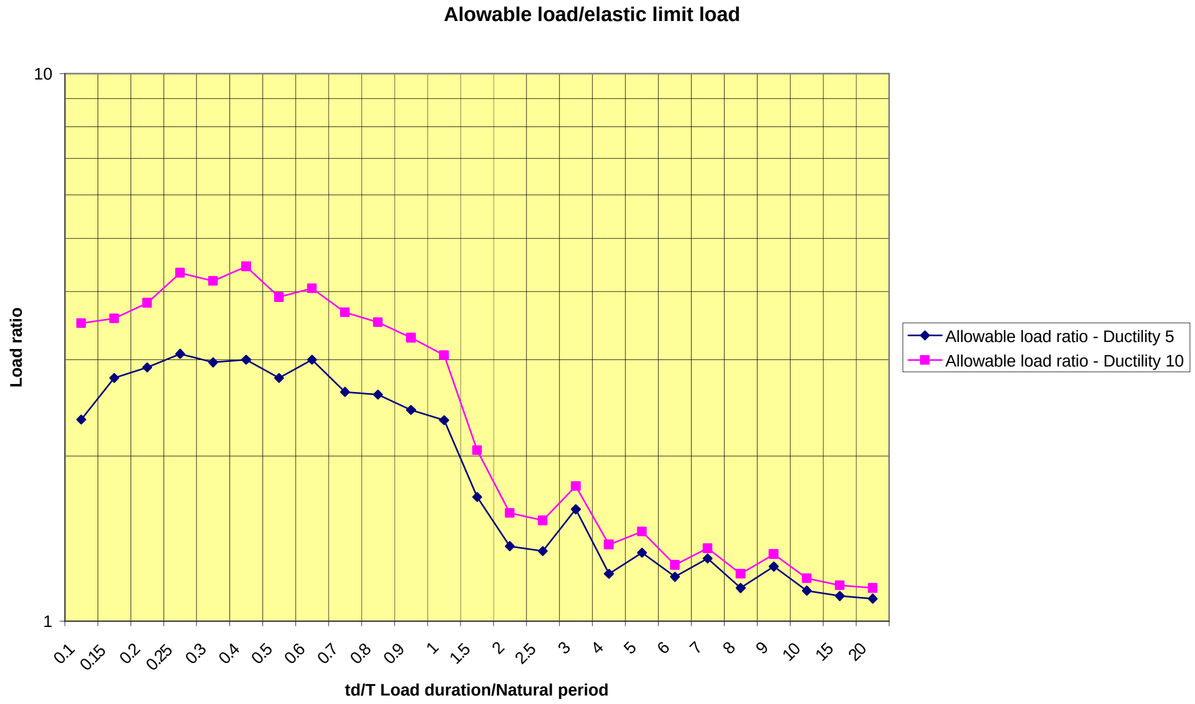


Figure 7 Capacity Ratio Curves (Ductility Factors)