Proceedings of OMAE '02:
21st International Conference on Offshore Mechanics and Arctic Engineering
June 23-28, 2002, Oslo, Norway

OMAE02-28623

NEW GUIDANCE ON FIRE AND EXPLOSION ENGINEERING

Steve Walker
MSL Engineering Limited, Platinum Blue House, 1st Floor, 18 The Avenue, Egham, Surrey TW20 9AB, UK
Tel: +44 (0) 1784 439194, Fax: +44 (0) 1784 439198, E-mail: swalker@mslengineering.com

Brian Corr, Vincent Tam
Pat O'Connor
BP, 501 Westlake Park Boulevard, Houston, TX 77079-2696, USA
Tel: 001 281 366 2985, Fax: 001 281 366 7983, Email: oconnpe@bp.com

Justin Bucknell
MSL Services Corporation, 11111 Katy Freeway, Suite 620, Houston, Texas 77079, USA
Tel: 001 713 463 6180, Fax: 001 713 463 6557, Email: Jbucknell@mslengineering.com

ABSTRACT

This Paper describes a new approach to explosion loading and response developed by MSL as part of a project to develop Guidance for the Design and Assessment of Offshore Installations against Fire and Explosion accidental events.

The Guidance document developed for BP is being considered as one of the source documents of the new Guidance Notes for the UK. This project is in progress and has been financed by UKOOA and the HSE.

The Guidance document is based on the risk matrix approach described in API RP 2A (21st edition) and is being considered as one of the source documents for the development of the new API Recommended Practice on Fire and Explosion Engineering.

This paper outlines the new thinking which is being incorporated into the new Guidance Notes and the new API standard. The paper concentrates on the areas of explosion loading and response and in particular the use of simplified methods in structural explosion assessment and design.

INTRODUCTION

It is now twelve years since the results of Phase 1 of the Joint Industry Project on 'Blast and Fire Engineering for Topside Structures' gave rise to the Interim Guidance Notes[1]. There have been a number of significant developments since that time some of which have been published in Technical Notes by the Fire and Blast Information Group (FABIG) [2,3,4,5,6,7].

Other valuable work, mostly executed in Norway and following the probabilistic approach, has resulted in the new NORSOK guidance documents [8,9] which will also be source documents for proposed future ISO Standards and the new Fire and Explosion Engineering Guidance Notes. The results of these and other major investigations are summarised in the new Engineering Handbook published by Corrocean [10].

In view of the maturity of the subject area and the thinking behind the UKOOA decision framework the focus is turning to more code based and simpler methods of analysis and assessment, particularly in the early stages of design or assessment projects.

The paper begins with a brief review of the API [11] approach and defines two levels of explosion overpressure representing the 'Design Level' and 'Ductility level' events by analogy with API's approach to Earthquake assessment. This approach uses, as its starting point, the cumulative probability curve of overpressure and associated return periods as recommended by the NORSOK Protocol [8].

Three levels of analysis/assessment are defined following API these are:

? Screening Analysis – For an existing installation consists of condition assessment followed by Design Basis checks which are checks to determine whether the methods used for design are appropriate for the fire and explosion scenarios considered.

? Design Level Analysis – These are conventional linear elastic load cases similar to those used routinely for design with the normal code based elastic failure criteria applied.

This level of analysis is scenario based and examines the residual events which remain after mitigation measures have been introduced. The Design level and Dimensioning explosion cases are examined using these simple analysis methods.

? Ultimate Strength Analysis - This is a non-linear analysis suitable for the most extreme ('Ductility Level') credible events which could occur on large offshore installations. This level of analysis is recommended for checking barriers and their connections.

The 'Ductility Level' explosion overpressure may be replaced during the early stages of a project by an equivalent linear elastic analysis utilising the concept of a 'Dimensioning Explosion' overpressure. This Dimensioning Explosion Overpressure is calculated from the Ductility Level Overpressure by taking into account the reserves of strength in the structure and reducing the applied overpressure accordingly.

(n.b. The Dimensioning Explosion overpressure as defined in NORSOK is defined (loosely) as that explosion overpressure which 'designs' the structure and differs from the definition given above which is a modified overpressure for use in elastic design.).

The focus of the remainder of the paper is on 'Design Level' structural explosion response analyses which are linear elastic analyses and which may be routinely examined in a project environment as a conventional load case.

The mechanism of loading of Primary structure in an explosion event and the relaxed performance standards associated with an extreme event give rise to a much reduced equivalent elastic load. An example of an appropriate performance standard is given in Section 3. This is examined in detail in the latter part of this paper.

INSTALLATION CLASSIFICATION FOR EXPLOSION ASSESSMENT

Ref. [12] gives more details of the process outlined below for both fire and explosion assessments. The risk assessment process which follows the API approach is shown in Figure 1.

The first task is to assign an exposure category to the installation. This is a measure of the manning levels on the installation and the facilities available for evacuation or taking refuge on the installation during and after ignited hydrocarbon releases. This is referred to as the Life-Safety measure for the installation. Consequences of a release are represented in general terms by looking at the potential vulnerability of the installation to a large range of possible release scenarios taking into account measures such as inventory, water depth and the degree of mitigation/prevention in place.

These two measures are combined to give an exposure category L-1(high), L-2 or L-3(low). The exposure category is the more restrictive level derived for Life Safety and Consequences.

The probability of an ignited release is then estimated from indicators such as equipment type, confinement and other indicators relating to the operation of the installation.

The consequences and probability of an ignited release are then combined to give a risk level associated with the installation as given in Table 1.

This Table is a modification of that given in API as the entry for low probability events on installations with a high exposure category (L-1) has been elevated to Risk level 1 to reflect the severity of rare explosion events. At the time of writing there are still a number of aspects of this approach which are being debated by the API committee responsible for the proposed 'API RP on Fire and Blast'.

Installations found to be at high risk (Risk Levels 1 or 2) will need to be subjected to a Structural Assessment, which at the highest level will involve the examination of the consequences of specific, representative explosion scenarios. This is shown as Task 6 of Figure 1.

The large majority of existing small, normally unmanned installations will be assigned Risk Level 3. It is recommended that installations in this category may be eliminated from further consideration.

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.

EXPLOSION LOAD CASES FOR STRUCTURAL 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.

For the 'Ductility Level' explosion a performance standard such as the one given below is typical.

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[12] must be available to protect those not in the immediate vicinity of an explosion and to survive the event without injury.

Other requirements may be specified to limit or prevent escalation resulting from additional releases.

For the 'Design Level' explosion it is required that the Primary Structure remains elastic, with the Essential Safety Systems remaining functional. This load case is a relatively high probability low consequence case which provides a measure of asset protection. It should be possible to re-start the installation shortly after this explosion event. Assessment of the installation against this loading will often identify additional improvements/modifications to the installation.

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 associated with the event) and the corresponding fatality levels.

The starting point for the derivation of the 'Design Level' explosion overpressure is the cumulative overpressure distribution for the installation. An example cumulative distribution is shown in Figure 2 from Reference 13. This shows the probability that a given overpressure will not be exceeded. This distribution is based on the conditional probability of a peak overpressure given that an explosion occurs and corresponds to a mean peak pressure of about 0.7 bar.

Published probability distributions have been found to fit the exponential distribution which has a mean value ‘?’ equal to its standard deviation. The exponential distribution is chosen as this gives a linear plot of probability of exceedance equal to exp(-?x) on logarithmic paper. This is true for most published overpressure exceedance curves. This tends to break down at the lower end and the upper end. This latter is not surprising as the cumulative distribution is asymptotic at the upper end. The fact that the means and standard deviations are roughly equal is a consequence of the uncertainty inherent in the calculation of peak overpressures.

It has been proposed [14] that a Design Level Overpressure is chosen based on the Ductility Level overpressure. It is likely that 'bounding cases' will all lie close to this value with respect to probabilities and lie on the right hand side of Figure 2. 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 is recommended that Design Level Overpressures may then be derived by considering the overpressure corresponding to return periods one or two orders of magnitude lower than the extreme event or probabilities one or two orders of magnitude higher.

Rnom, the return period for a ‘detectable’ overpressure (release and ignition), is given by:

$$R_{nom} = 1/p_{nom} \quad (1)$$

Where pnom is the nominal annual probability of a ‘detectable’ overpressure occurring. The precice value of the detectable overpressure is not important here as the ratio of return periods defines the Design Level overpressure.

The return period ‘R(Pmax)’ associated with a given overpressure ‘Pmax’ may be calculated from the associated Cumulative Probability ‘Pcumul(Pmax)’ from Figure 1 using:-

$$R(P_{max}) = 1/(1 - P_{cumul}) * R_{nom} \quad (2)$$

Using the data of Figure 2 and assuming a nominal return period of 100 years for a detectable overpressure, gives a return period of about 27,000 years for the occurrence of an overpressure over 2.5 bar. The Design Level Overpressure will have a return period of about 2,700 years, which from Figure 3 is an overpressure of 0.8 bar. A value of Rnom does not need to be derived or assumed for the calculation of the Design Level overpressure.

For small, low risk installations 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.

It is envisaged that the ‘Dimensioning Explosion’ described later in this paper and the ‘Design Level’ explosion will be of similar magnitude in most situations.

EXPLOSION RESPONSE OF PRIMARY STEELWORK

Explosion overpressure loads will initially act on all connected surfaces such as barriers, vessels, cladding and deck plate giving indirect loading on the primary frame. The magnitude of these loads will depend on the capacity of the surface and the time of maximum load application will depend on the dynamic characteristics of the connected surfaces.

The Performance standards for the Ductility Level event will usually allow local damage and permanent deformations of secondary structure and barriers. It may even be desirable for such secondary and tertiary structures to deform plastically enabling them to transmit much reduced loads to the connections with the primary structure. The large deflections of the secondary structure may, however, result in large membrane forces acting perpendicular to the axes of the primary columns or deck beams. Often these forces are balanced globally by opposing forces from the opposite side.

Blast and fire walls are often arranged to span between decks with the main deck beams taking most of the load. The columns between decks are then subsequently loaded but because of the pressure loading pattern these may be put into tension and column buckling may not then be a problem.

Direct overpressure loading on isolated columns and beams are relatively small due to the small area presented to the explosion. In these circumstances it is necessary to consider dynamic pressure or drag loads on such members. Some guidance is given in References 10 and 15, where limits on the diameter of obstacles and their location with respect to the vents determines the relative magnitudes of the various components of dynamic pressure loads.

In summary, the response of primary structure to explosion loads differs in a number of respects from the response of free standing blast walls.

? Loads transmitted from secondary and tertiary structural elements may be quite different in magnitude, distribution and form from the overpressures seen by the secondary structure.

? Static and indirect loads transmitted through the frame modify the loading and hence the response.

? The spacial variation of load on a large frame will reduce the peak overpressure suitable for design of the frame. It may be appropriate to apply a scale factor to the local peak overpressure.

This Section attempts to quantify the various factors which alleviate the effect of overpressure loads on Primary structure. Section 5 then discusses how the reserves of strength and the load alleviation enables a reduced overpressure to be determined which is representative of the Ductility Level explosion but suitable for application to a linear elastic structural frame model.

Received loads on Primary Steelwork – Secondary Steelwork response

Reference 16 quantifies some of the effects related to secondary and primary structure ductility discussed above.

The conventional definition of the ductility factor ‘?’ is the ratio of the maximum displacement to the displacement ‘at yield’.

For a bi-linear resistance displacement (RX) function say for a simply supported carbon steel beam loaded statically, this ratio is easy to define. For beams of different materials such as aluminium or stainless steel, for two way spanning panels or beams with moment restraint at the ends, the displacement at first yield may not be easily identifiable or representative. In these cases it is customary to idealise the RX function as a bi-linear curve with the same area following the equal energy absorption assumption. The energy absorbed is equal to the area under the curve. The plastic section of the RX curve need not be horizontal as membrane or tension effects may come into play. This idealisation is usually possible for secondary structural components.

For a pressure pulse with maximum P_{max} and a duration of t_d acting on a member with natural period T, the static capacity R? required to resist this load with a response ductility of ? is given by:

$$P_{max}/R? = T/(?t_d) \, . \, ?(2?-1) + (1-1/(2?))/(1+2T/?t_d) \quad (3)$$

This equation is quoted in Reference 16 and was first derived by Newmark [17]. The error in this equation is less than 5% for all load duration to natural period ratios and for the practical range of ? which is considered to be less than 12. Note that the load applied P_{max} is a static load equal to the peak overpressure.

For elastic response the allowable ductility ? is one. This gives the maximum required elastic resistance R_e as:-

$$P_{max}/R_e = T/(?t_d) + 1/(2+4T/?t_d) \quad (4)$$

Equations 3 and 4 enable the reduction in resistance (or load) which is acceptable for varying allowable ductilities and various load duration to natural period ratios.

If R? is taken as a measure of the maximum load which can be transmitted to the primary structure then the reciprocal of equation 3 gives the effective reduction in primary structure load from the allowed ductility of the secondary structure and its dynamic properties relative to the loading duration.

Figure 4 shows the variation of transmitted load from secondary structure with ductility and load duration. As expected if the secondary structure is allowed to deform plastically with a high allowed ductility then the transmitted load is a smaller proportion of the applied overpressure. Loads of small duration relative to the natural period are alleviated by the dynamics of the system. Similar curves may be constructed to represent the reaction forces calculated using the method given in Biggs' [17].

Figure 4 indicates that the design pressure may be reduced by the factor above if the reduced load from surfaces are distributed to the primary steel work using the 'tributary area method'. In this method the area of the loaded panel or barrier adjacent to the primary member is applied to the member as a line load. The stiffness of panels and surfaces need not be represented in the structural model although their masses should be included if a dynamic analysis is to be performed. Figure 4 indicates that an allowable ductility of the order of ten for a panel would give an effective load (Dimensioning Overpressure) one third of the applied peak load as most panels of this type would also have the appropriate dynamic characteristics to place them on the left side of figure 4.

Effects of Plasticity and Dynamics – Biggs' approach

The load transmitted to the Primary structure may then cause some allowable plastic deformation of this part of the structure. If this is the case then some further reduction in the effectiveness of the load or conversely it may be possible to further reduce the dimensioning load to represent the event in a linear elastic analysis.

This section examines the reserves of strength in that part of an installation which can be idealised as a one degree of freedom structure. A good example would be that part of a deck between primaries.

The Biggs chart shown in Figure 5 may be used to estimate the benefit to member capacity resulting from plastic deformation and dynamic effects (with the assumption that the chart is applicable to the structure and loading considered).

This 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 . Each curve is for a different ratio of (plastic) resistance R_m to peak load F_1. It is assumed that the RX diagram for the structure may be idealised as a bilinear function.

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 static 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 or load ratio curves for allowed ductility levels of 5 and 10 are shown in Figure 6. The curves indicate that the 'Headline Overpressure' value may be reduced by the load ratio to give an overpressure to be used for Primary structure design and dimensioning. 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 and the triangular nature of the loading time history. If smooth idealisations of the pressure time history curves are used then it is expected that these ripples will disappear. The validity of these curves is still dependent on the assumptions implicit in the Biggs method.

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 similarity between Figures 5 and 6 is not co-incidental as similar effects are accounted for in the derivations. Figure 6 is not based on reactions but capacity ratios and so direct comparisons between Figures 4 and 6 are not straightforward.

Biggs [17] does give a method for derivation of reaction loads which are correct in shear but the onset of membrane/tension effects at large deflections invalidates the method and the assumption of a bi-linear RX curve.

The reaction loads are applied to the primary frame structural model. If local plasticity is allowed in the Primaries then further reductions in effective load for elastic analysis may be gained further reducing the Dimensioning Explosion Overpressure.

Ductility of the Primary Steelwork

The response of the primary frame will differ from that of secondary panels and deck spans. The loads in the frame are transmitted by frame action, which is a combination of bending, compressive and tensile response. The frame will also have a fair degree of redundancy and the resistance displacement curve (the RX curve) will reflect the sequential formation of plastic hinges. One form of this function has been postulated in Reference 16 and is shown in Figure 7.

Both the displacement 'X' and the resistance 'R_{max}' are scaled on the elastic limits for these variables 'Xe' and 'Re'. R_{max} is an asymptote and is never reached a value is chosen to be close to but just above the maximum predicted load/resistance for the ductility level explosion '‘$_{Rmax}$'.

The expression for Primary structure corresponding to equation 3 for the secondaries is given in Reference 16 as:

$P_{max}/R? = T/(?t_d)\ .\ ?A + A/2?/(1 + 2T/(?t_d))$ (5)

Where: A is the area under the curve (energy absorbed) up to the maximum deflection 'X_{max}' divided by the area under the curve up to the end of the elastic section, at deflection 'X_e' .

Therefore:

$A = \ln((\cosh(K.X_{max}/R_{max}))/\ln(\cosh(R_e/R_{max}))$ (6)

As, by definition, $K.X_e = R_e$

A is a measure of the ductility based on energies absorbed to maximum deflection and energy absorbed to effective yield. If we define the ductility ? as X_{max}/X_e , then equation 6 may be written:

$A = \ln(\cosh(?R_e/R_{max}))/\ln(\cosh(R_e/R_{max}))$ (7)

This enables the required resistance at ductility ? 'R?' to be calculated from T, t_d, R_{max} and R_e where these quantitries now refer to the primary frame characteristics. For an elastic analysis $R_{max} = R_e$ and A = 1 by definition. In this case Equation 5 then represents only the effect of dynamic amplification/alleviation.

Other Effects

The above treatment enables the Dimensioning Explosion load for (static) elastic analysis to be estimated. There are however two further effects which may be considered.

1. The material strain rate effect will enable the design yield strength to be increased. This will depend on the local strain distribution but common practice in the Nuclear Industry is to increase the yield stress by 20%.
2. The scale effect which takes into account the finite extent of the loading relative to the typical dimensions of the Primary frame.

These effects are dealt with in more detail in Ref. [12] which also describes methods for deriving the Dimensioning Explosion based on Accidental and Elastic limit state equations and details modified code check methods which may be used in explosion assessment.

It should also be borne in mind that the explosion load case is an accidental load case which allows stresses in the structure to approach yield. This will affect the interpretation of the utilisation factors from the elastic frame analysis.

Recommended Ductility Values

Care must be taken in choosing the values of ductility in the above approach. The appropriate ductility ? must be chosen taking into account the mode of loading and the classification of the component members. Members classified as 'plastic' are able to develop a plastic hinge before buckling and can tolerate larger ductilities before failure.

Flexural members may have ductilities of up to 12, but compressive members. Which may fail in buckling the recommended limit is between 1.5 and 2.

It is stated in reference 16 that a primary frame designed to existing codes should possess a ductility of between 2 and 4. Generally a ductility of 2 leads to repairable damage, whilst a ductility of 4 represents collapse.

It should be emphasised that connections to barriers and joints should be checked against the actual Ductility Level Explosion overpressure. Shear checks for these details should also be performed.

CONCLUSIONS

? 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 earthquake assessment.

? Methods are described which enable the derivation of a Dimensioning Explosion overpressure which may be applied to a static or dynamic elastic frame analysis to assess the structure against the Ductility Level explosion. This will be useful in the early stages of a design or assessment project.

ACKNOWLEDGEMENT

The Authors would like to thank Sirous Yasseri of Kellogg Brown and Root UK, for permission to include information published recently [16] relating to approximate methods for blast resistant design.

This paper reflects the opinions of the authors. The process of generation of guidance requires a wide consensus to be developed. This process is still underway.

REFERENCES

[1] *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, SCI Publication, November 1992.

[2] FABIG, Technical note 1, *Fire resistant design of offshore structures.*

[3] FABIG Technical note 2, *Technical note on Explosion mitigation systems*.

[4] FABIG, Technical note 3, *Use of ultimate strength techniques for fire resistant design of offshore structures*.

[5] FABIG Technical note 4, *Explosion resistant design of offshore structures*.

[6] FABIG Technical note 5, *Design guide for stainless steel blast walls*.

[7] FABIG Technical note 6 *Design Guide for Steel at Elevated Temperatures and High Strain Rates*.

[8] Pappas J., *The NORSOK Procedure on Probabilistic Explosion Simulation*, Paper 5.5.1, ERA Conference, 'Major Hazards Offshore', London 27-28 November 2001, ERA Report 2001-0575.

[9] NORSOK, *Guideline for Risk and Emergency Preparedness Analysis,* Z-013.

[10] *Design of Offshore Facilities to Resist Gas Explosion Hazard, Engineering Handbook*, Ed. Jerzy Czujko, Corrocean 2001.

[11] *Recommended Practice for the Planning, Designing and Construction Fixed Offshore Platforms – Working Stress Design*, API RP 2A-WSD, 21st Edition. API (American Petroleum Institute).

[12] Walker S., Corr B., Tam V., Bucknell J. and O'Connor P., *Simplified Approaches to Fire and Explosion Engineering*, Paper 3.3.1, ERA Conference, 'Major Hazards Offshore', London 27-28 November 2001, ERA Report 2001-0575.

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

[14] 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.

[15] *Guidance for the protection of offshore structures against fires and explosions*, Ref. CH152R002 API Draft 0, November 2001.

[16] Yasseri S., *An Approximate Method for Blast Resistant Design*, Paper R429, FABIG Newsletter January 2002, SCI publication.

[17] Biggs J.M., 'Introduction to Structural Dynamics', McGraw Hill, 1964.

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		

Table 1 Risk Matrix for Explosion (and Fire) Events

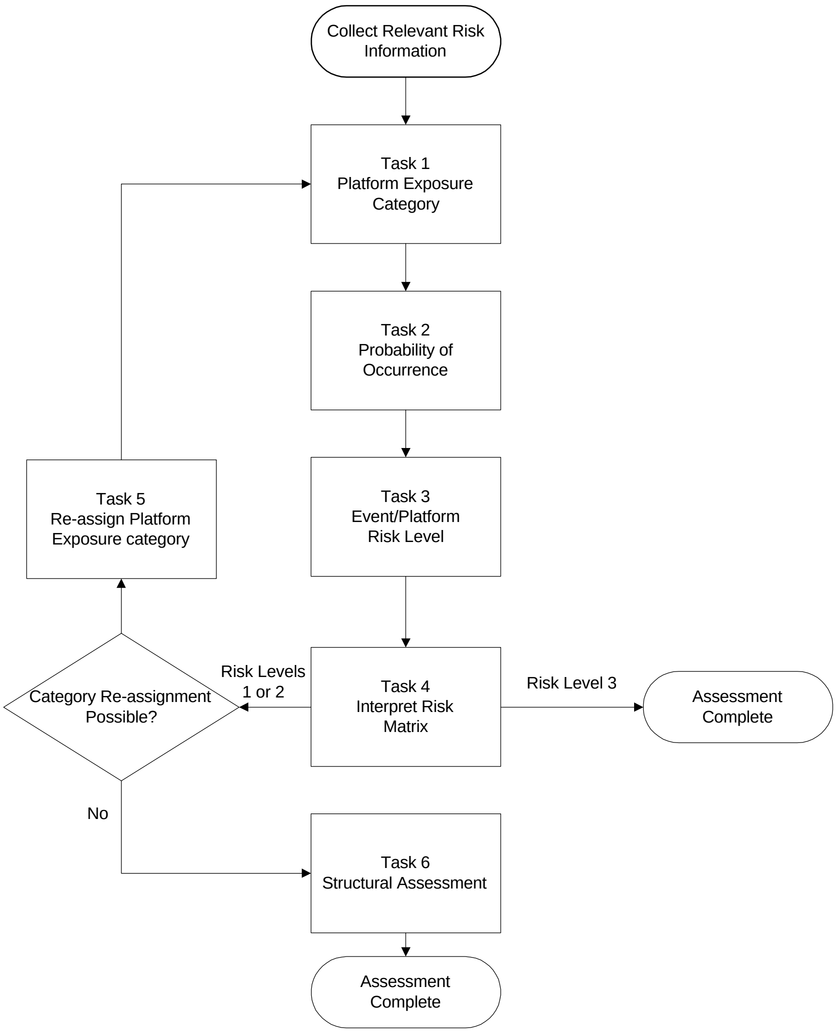

Fig. 1 Overview of the Assessment Process

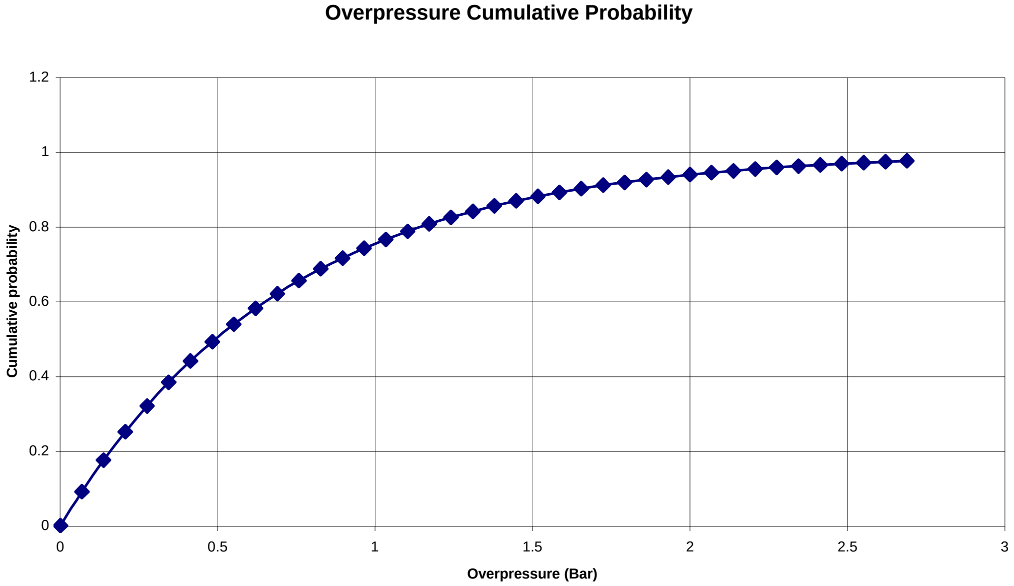

Fig. 2 Example Overpressure Cumulative Probability (from Reference 13)

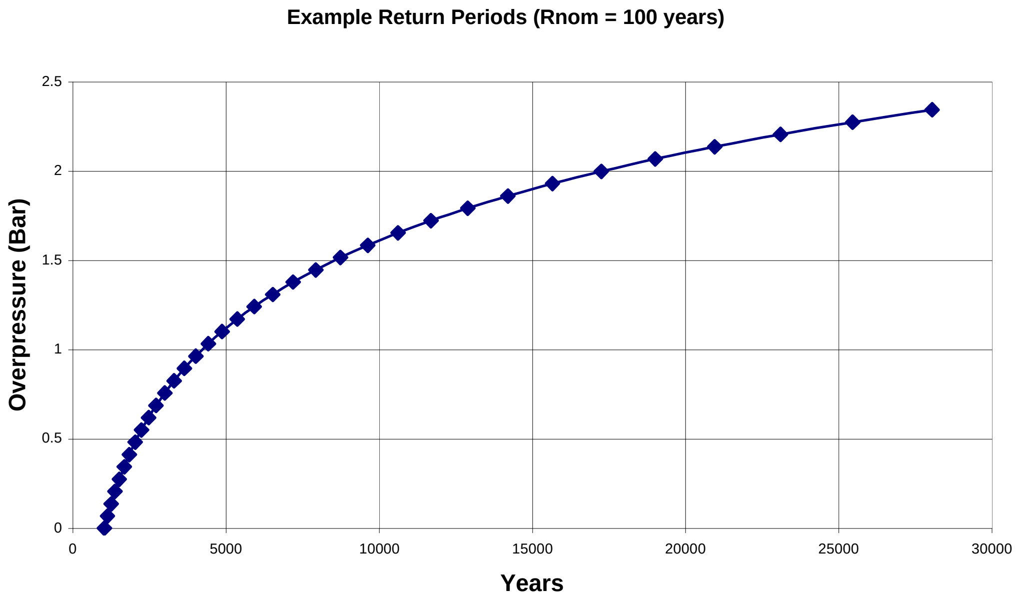

Fig. 3 Example Return Periods (Rnom = 100 years)

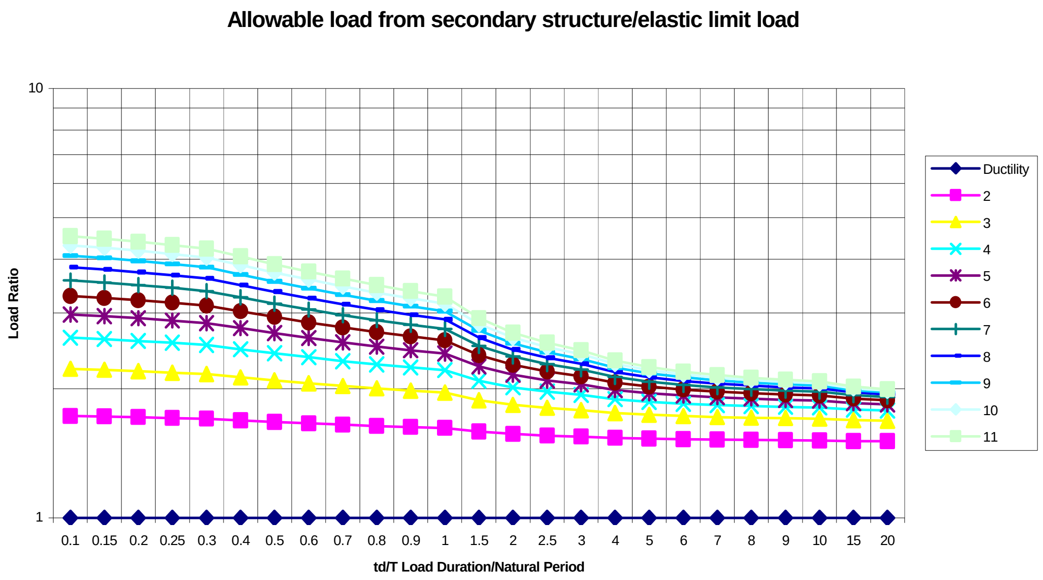

Fig. 4 Load Reduction Factor – Load from secondary structure Vs ductility and load duration

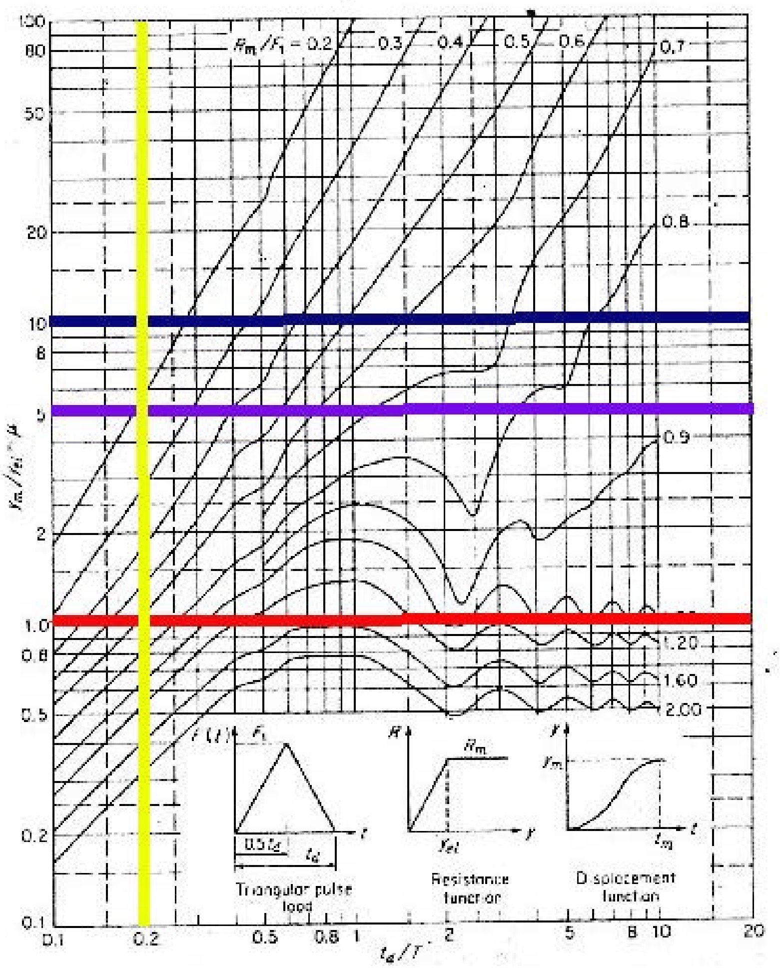

Fig. 5 Biggs' Response Chart [17]

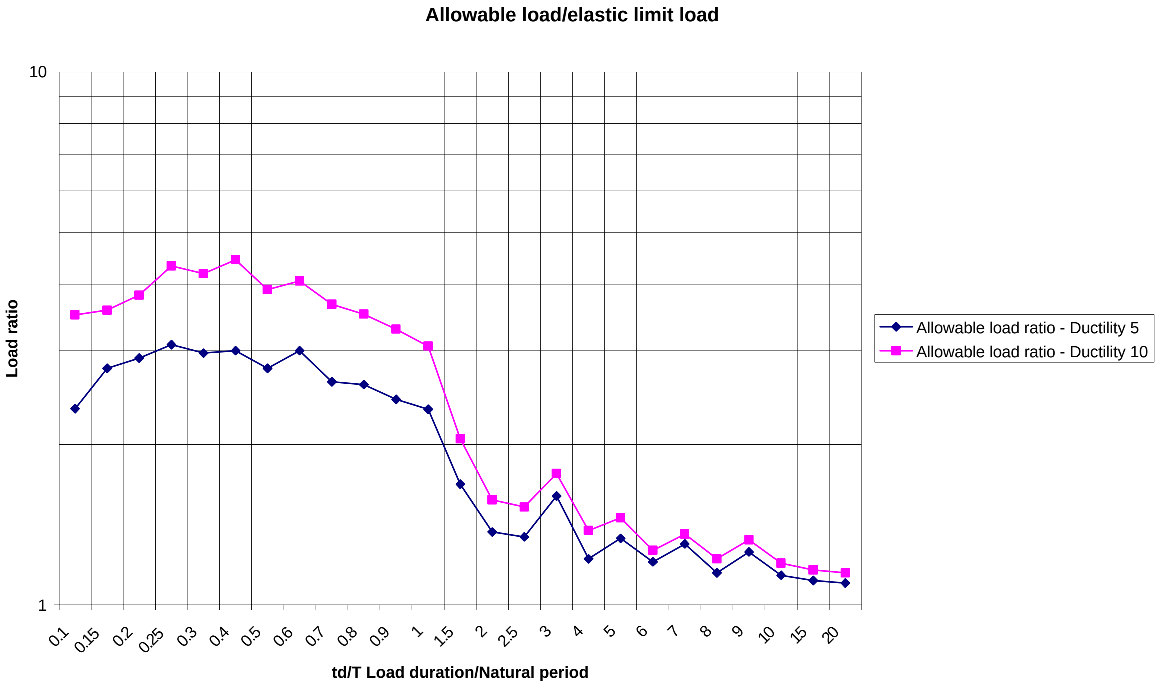

Fig. 6 Capacity Ratio Curves (Ductility Factors)

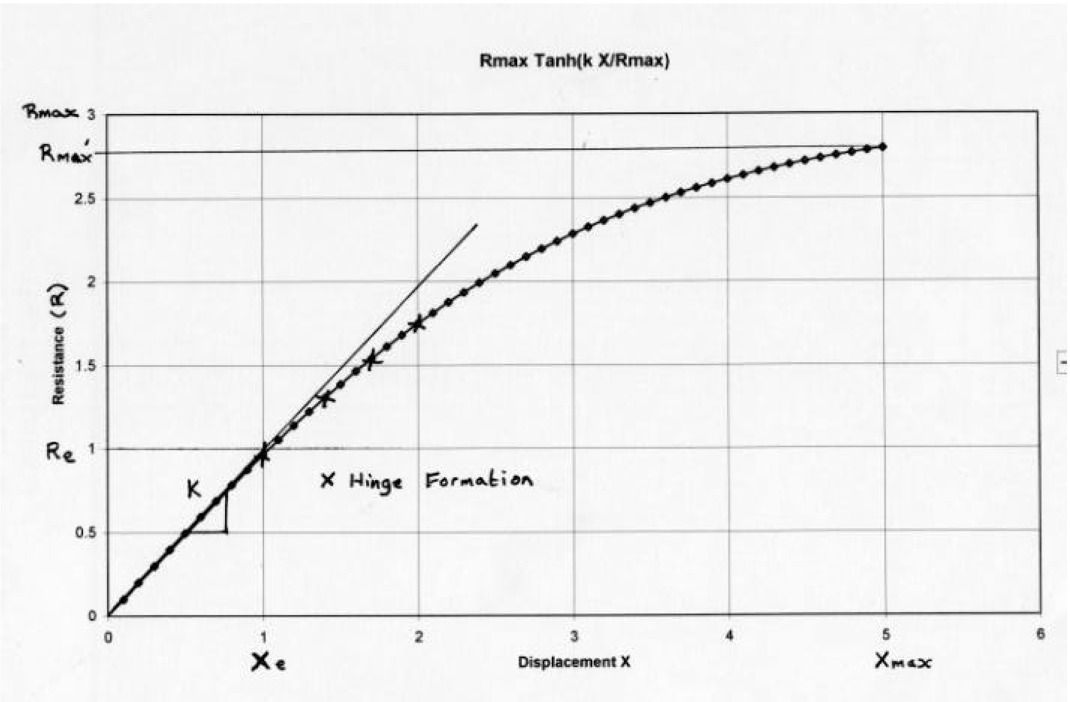

Fig. 7 Proposed Resistance/Displacement function - Primary Frame [16]