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Report on Task I -- Define and Characterize the Offshore Fire Problem

Improved Means of Offshore Platform Fire Resistance (UCB Eng-7429)

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Define and Characterize the Offshore Fire Problem

Introduction

Over the last twenty years, many significant advances have been made in offshore technology, yet relatively little attention has been directed toward improving the inherent level of platform fire resistance (endurance). We continue to believe that this is largely due to the lack of a rational definition of the offshore platform fire problem in terms of imposed demands and structural system capacity. Even today, over three years since the Piper Alpha incident, U.S. offshore structural design criteria (API RP -2A) fails to account for thermally imposed demands.

As stated in our original research proposal, an overall goal was to develop background information that can be used to define design guidelines for protecting structures from explosions and fires on offshore platforms. Our immediate objectives were

(a) to clearly define the offshore fire protection problem, and
(b) to develop an engineering approach to improving offshore platform fire resistance by extending structural fire endurance.

To accomplish these objectives the research effort was subdivided into three primary tasks:

Task I Define and characterize the offshore platform fire protection problem and develop a database.

Task II Determine appropriate levels of thermally imposed loading criteria for given return periods, i.e. summarize fire demand.

Task III Identification of appropriate fire-based structural performance design criteria.

This report addresses the progress made on Task I, defining and characterizing the offshore fire problem. A significant part of Task I was the initial data gathering effort. We performed a survey to develop an overview of what has been done to date from a design applications standpoint, as well as assess currently emerging design innovations in light of recent incidents. Appendix A of this report presents relevant findings of this survey.

Task I specific activities included:

- investigating present design practices and conditions.
- examining state-of-practice design trends/innovations.
- assessing critical areas of vulnerability from thermal impact.
- reviewing the historical database of thermal damage to the structure and associated life-safety issues
- identifying failures of existing protective measures and alternative means of improving fire resistance that offer the greatest utility and cost benefit.

Anticipated heat flux levels, the rate of heat release, fire growth, and fire duration, i.e. **fire severity**, are key parameters in assessing the predicted rate and extent of progressive failure of structural elements and degradation of system capacity. While the thermal characterization of offshore fire demands will be addressed in Task II, some overlap with Task I activities naturally occurred as reported herein.

Characterizing the Offshore Fire Problem

From the start, we recognized that it is essential to clearly characterize the offshore platform fire protection problem in terms of design constraints and requirements, i.e., the (thermal) demand and (structural) capacity sides of the "design equation." We have learned much from reviewing the experiences of past failures, and analyzing the existing historical database for both lessons learned, as well as past successes.

Figure 1, **The Offshore Fire Problem**, is an influence diagram illustrating the key elements and their relationship. Fire severity, in terms of the rate of heat release, fire size & growth, and fire duration, are the heart of the demand side of the equation. Structural response to these demands is a function of both fire severity and **thermal** robustness (or lack thereof). Simulating platform behavior under fire conditions by "exercising" structural designs/configurations under thermal demands allows analyses of reserve and residual strength requirements needed to achieve fire-based reliability targets.

Such an analysis also provides insight into dependencies (or couplings) that may not be significant in conventional load analysis, but are vital to the maintenance of redundancy and robustness during fire exposure. For example, structurally "decoupling" components of the support system serving critically important life-safety functions such as accommodation module support frames and escape-ways, may prove to be an alternative approach to improve structural fire endurance in a selective and cost-effective manner. Increasing inherent fire endurance through increased thermal mass may also prove to be a low-cost approach to meeting fire-based reliability targets.

In the case of a major fire incident, a primary objective is to be able to maintain the structure's integrity for a sufficient period of time to permit fire and damage control measures to arrest continued deterioration of capacity and progressive failures, while permitting evacuation of operating and maintenance personnel. This involves the analysis of strategies and alternative approaches, within the context of defined restraints and service requirements.

Figures 2 and 3 characterize the offshore fire problem in terms of fire demand load, system capacity, and exposure duration. The structure must be able to maintain its safety functions for a required performance time, given highly probable fire and explosion damage. This requires appropriately placed redundant elements, provision of ductility (ability to re-distribute loadings), and excess capacity (ability to withstand increased loadings), i. e., fire-based structural design criteria. As stated in our original proposal, we believe that a baseline knowledge of how structural design factors are influenced by thermally imposed demands is necessary before additional mitigations, such as fire resistive coatings, can be rationally specified.

Fire risk on any given platform depends on a very large number of variables. Of greatest concern are fires involving the release of hydrocarbon-based fuels under high pressure and flow rates. Such fires, referred to as high momentum jet fires, cause the highest fire demands offshore. Jet fires, both single phase (all gas or liquid) and two phase (a combination gas and liquid) produce the highest heat release rates and heat flux loads, and are the most difficult fires to suppress. In addition, jet fires involving liquids will often form a pool of burning liquid on a platforms deck that may spread fire to other uninvolved areas.

Jet fires may occur at any location on a platform where oil and gas is produced, processes, or transferred under pressure. Their occurrence may be directly due to a mechanical or material failure, such as the failure of a flange gasket or pump seal, or due to human error such as cutting into an operating pipeline with a torch. Additionally, jet fires may also be the result of an escalating fire/explosion scenario that began somewhere else on the platform and has caused the failure of a pressure-containing element of the process system.

Offshore platforms are especially vulnerable to an escalating fire scenario due to the necessarily close spacing of high-pressured equipment and the nature of the operations conducted offshore. Any fire that is not quickly detected and suppressed is of great concern; especially on those platforms with accommodation facilities where life-safety is at issue. In general, it can be said that an offshore platform has all of the fire-safety concerns found in a typical onshore commercial or industrial occupancy, plus several additional factors that greatly increase the risk (both likelihood and magnitude) of a significant event.

Some of these risk factors unique to offshore operations include:

- Unprotected (unfireproofed) structural steel support systems and hydrocarbon-handling equipment that can fail within a few minutes when subject to direct flame impingement.
- Layouts and spacing arrangements that do not allow for adequate separation of high risk equipment items or operational areas.
- Accommodation facilities located on the same structure as drilling and production operations.
- Unprotected egressways and exiting/escape constraints, especially in environmentally hostile areas such as the Cook inlet and the Beaufort Sea, Alaska.
- Requirements for self-sufficiency in the event of an emergency; reliance on timely outside emergency response is usually not a viable alternative.

- High reliance on system integrity (both mechanical and electrical/control system) to secure safety; extremely vulnerable to consequences of inadequate inspection, maintenance, equipment testing, lack of redundancy, etc.

- Minimal time to respond to impending emergencies to avoid escalation; incidents not controlled within the first few minutes can pose a grave danger to welfare of entire platform and crew.

- Highly susceptible to explosion damage and incident escalation -- especially where: equipment areas are enclosed (subject to accumulations of flammable concentrations of gas) and ventilation systems are inadequate or not maintained; no provisions have been made for blast resistance or explosion venting; a high reliance placed on active water spray systems for fire protection (very susceptible to damage from local explosions); no automatic gas detection has been provided; redundant fire pumps are not adequately separated or segregated, etc..

- Unprotected data highways for critical control and shutdown systems; open cable trays in grouped configurations employing polymeric coverings that propagate fire and liberate toxic gases when ignited.

- Vulnerable control centers that are susceptible to damage from fires and explosions, leading to loss of control and escalation of the scenario.

- Multiple operations, many hazardous in nature, being conducted simultaneously on the structure, e.g., simultaneous drilling, production, and work-over operations, multiple construction/inspection/maintenance operations, some of which involve hot work and equipment disassembly, occurring simultaneously during normal operations, use of contract personnel not familiar with platform or inadequately trained, etc.

- Especially vulnerable to the consequences of human error; however, tends to place high demands on accuracy of human response; platform networks that require coordination between multiple platforms interconnected by pipeline may be affected by decisions of offsite personnel in emergency situations; communication systems/personnel susceptible to failure/misunderstandings.

Notwithstanding the preceding considerations, offshore platforms must perform their intended functions under a variety of difficult conditions including severe storms, seismic events, and other "acts of God" that are beyond the control of management. The risk of fire and loss of (fire) control are further compounded during such events. With the exception of fire and explosions, all recent platform designs account for environmental factors such as seismic and storm-induced loads. This is accomplished by evaluating the expected severity and frequency of such loads, and applying appropriate load factors based on the risk of experiencing a given load severity.

A taxonomy of key risk factors that affect the probability and potential severity of fire on an offshore PDQ platform has been developed in Task I (Appendix B). We believe it is important from a risk-assessment perspective to identify what these factors are and to understand how they affect both the probability of fire occurrence and the propagation fire after initiation.

However, three axioms of fire risk offered by Watts[1] are appropriate to recognize: 1) the risk of fire is <u>always</u> greater than zero, 2) a universally acceptable level of fire risk (e.g. what constitutes "safe") does not exist, and 3) a totally objective or scientific way to measure fire risk does not exist. As Watts points out, this does not mean that fire risk analysis is necessarily arbitrary or invalid; rather, fire safety decision models can be an effective decision-making tool despite their heuristic nature.

A validated model for the risk-ranking of offshore platforms is not available. A model that presents a simple but effective means for assessing the relative fire risk and level of fire safety offshore, both for existing platforms as well as new designs has yet to be developed, and probably never will. While an accepted standardized methodology does not exist, establishing risk factors and their correlation to the overall level of fire risk is an important first step. Reliance on subjective evaluation and expert opinion cannot be avoided; however, as Watt points out, this should not detract from the usefulness of this technique. No doubt, through the application of the offshore fire risk index taxonomy proposed herein, subsequent suggestions for improvements will be forthcoming.

Offshore Construction Practices

The marine environment presents a unique set of conditions that dominate the methods, equipment, designs, and procedures employed in offshore construction. The design of offshore structures is based to a substantial degree upon our ability to construct, and our ability to adapt to environmental aspects as they affect construction[2,3]. Most methods and approaches used onshore usually prove totally impractical for marine design and construction projects. However, there are proven analytical techniques for analyzing the effects of fire in onshore steel frame buildings that can also be brought to bear on the offshore fire problem.

Onshore, steel frame high-rise structures are protected from ductile collapse under fire conditions by applying fire resistive coatings, e.g, fireproofing. Model building codes, such as the UBC[4] specify the amount of fire resistance required for each structural member and assembly in accordance with accepted test methods such as ASTM 4-119[5]. A modern high-rise building typically is required to have three hour-rated fire resistive coating on the structural steel framing. In addition, interior spaces are compartmentalized with fire-rated partitions and ceiling/floor assemblies in order to confine a fire and prevent its spread.

Offshore, however, design constraints and economic margins impose severe restrictions, and, while in some cases attempts to partially compartmentalize adjacent modules with "fire walls" are made, most structural steel-frame components and vertical separations between deck levels are not provided with any degree of passive fire protection.

On an offshore platform, weight, space and environmental ruggedness are primary considerations. Onshore fireproofing materials typically used for high-rise structures or onshore petroleum facilities are unsuitable for salient and corrosive marine service conditions. Concrete, which is often used in onshore petroleum facilities for protecting structural steel from fire, is considered too heavy, space-consuming, and permeable. Some epoxy mastic coatings have been effectively used offshore, but these are perceived as being expensive and, to date, there has not been a general consensus on the need for such protection.

The lack of consensus is partly due to the extent to which platform owners and designers go to prevent fires from occurring in the first place, and partly due to the lack of offshore regulatory requirements (due to marine design constraints) such as those contained in the UBC. This is perhaps the greatest single impediment to achieving effective structural fire protection -- a failure upon the part of the organization to recognize and respond to the problem. Pate-Cornel and Beal[6] have developed a taxonomy of organizational failures that can be directly related to numerous offshore accidents.

Offshore vs. Onshore Construction

Offshore platforms are similar in structure to modern high-rise buildings, but with significant differences. Most offshore platforms used for drilling and producing oil and gas in U.S. state and federal waters are constructed of tubular steel members welded together to create a template or jacket on which drilling and production modules are supported.

Tubular Steel Jacket and Module Support Frame (MSF) Design Parameters

The jacket structure design must account for a variety of load demands including, dead and live loads, wind, wave, ice, and current loads (environmental loads) seismic loads, operational loads (such as during drilling) and transportation and construction loads. In addition, accidental loads such as collision and fire should be accounted for during the design process.

The jacket must be sufficiently robust in terms of capacity, redundancy, and ductility, to transfer the environmental and deck loads to the pile foundation without loss of serviceability (failure) over a wide range of conditions. Failure is realized when structural deformation exceeds the limits of utility. In the case of fire, failure is realized as a result of progressive ductile collapse.

The platform jacket tubular members must support a combination of axial and flexural loads and are referred to as beam-columns[7,8]. Tubular members are connected together (using prefabricated joint connectors or "Cans") into various standard truss configurations, to form a free standing braced-frame configuration, sometimes referred onshore as a "space-frame." There are three kinds of structural elements (braces) that interconnect the jacket legs; diagonal braces in vertical planes, diagonal braces in horizontal planes, and horizontal braces.

As Graff[8] points out, the selection of the framing plan largely depends upon the experience and preferences of the design engineering team. It is also important to note that the reserve and residual strength of the structure, as later discussed, are primarily determined by the structural configuration and design philosophy rather than code requirements. As Lloyd and Clawson[9] note, "structural systems having different member arrangements that satisfy the same code provisions may have widely differing system strengths and redundancies". Good designer judgment is needed to achieve cost-effect systems having adequate levels of reserve and residual strength; **however, what constitutes adequate levels of strength for fire endurance has yet to be characterized**, presently leaving platform structural fire endurance largely to chance.

Figure 4[8] shows several commonly employed designs. The design shown in area 1 of Figure 4 employs the Warren bridge type truss. The other designs shown are common but are not specifically named. Most of the vertical bay sections shown resemble the Pratt of Howe type truss bracing system. The design shown in area 4 uses a common K truss in the transverse elevation and plan view. The finalization of the framing plan should optimize the capacity for lateral and torsional resistance for the environmental load criteria, since there is relatively little weight or cost advantage between these designs[8].

Tubular Member Design Parameters

The analysis of tubular steel jackets must account for many complex factors, including plastic beam analysis and torsional loading, inelastic behavior and post buckling effects, inelastic cyclic loading and fatigue effects, external hydrostatic pressure considerations, etc. The thermal impacts on system capacity and member inter-reactions are also complex and difficult to analyze. Effects include decreasing strength and stiffness, accelerated creep, excessive expansions, eccentrically induced loads, unbalanced parallel load paths, and nonlinear progression.

Bresler and Iding[10] have approached the problem of analyzing building structural deformations, stresses, and load-carrying capacities at elevated temperatures in much the same way as in ultimate strength analysis for wind or earthquake. This subject will be examined later in greater detail as a way of extending current onshore technology to the offshore frontiers.

Two design parameters used by offshore structural engineers that are of interest in regard to structural fire endurance are the slenderness ratio, kl/r, and the tube diameter to wall thickness ratio, D/t, where:

kl is the effective member length depending on end restraint conditions, and

l = actual member length

k is the effective length coefficient: k = 1.0 for a member pinned at both ends; k = 0.5 for a member fixed at both ends; k = 0.7 for a member fixed at one end and pinned at the other end; and k = 2.0 for a member fixed at one end and free from restraint at the other end[7]

r is the cross sectional radius of gyration: $(I/A)^{1/2}$

D is the diameter of the tubular member

t is the wall thickness of the tube.

Tubular members under axial compression can fail either to material yielding, local buckling, or Euler column buckling. The kl/r ratio is used to calculate the Euler critical buckling stress of a member in the formula:

$$Scr = (3.14)^2(E)/(kl/r)^2$$ [7]

For values of $30 < kl/r < 100$, the slenderness ratio is considered to be within the intermediate range. As a rule of thumb, most designers aim to maintain slenderness ratios between 60 - 90, since within this range, the member strength depends on the tangent modulus of the material and on end restraint design. In seismically active regions, the slenderness ratio of primary diagonal bracing in vertical frames is limited to a maximum of 80, and the D/t ratio restricted to 1900/Fy(ksi), e.g., about 53 for ASTM A36 steels.

A brace with a kl/r ratio of above 100 is subject to Euler elastic buckling, which is independent of a material's yield strength. Designers seek to avoid both Euler and local buckling by limiting the upper end of the ratio to less than 90 in order to take advantage of high strength steels[8]. The lower the slenderness ratio, the less will be the post-buckling reduction in compressive load carrying capacity[11]. One-sixth scale tests conducted at the University of California, Berkeley clearly demonstrated that for $kl/r < 60$, the buckling load will be close to the compressive yield load[11].

In order to reduce wave induced lateral loads on the jacket structure, it is advantageous to keep the diameter of the structural members subject to hydrodynamic forces as small as possible, thereby reducing drag. Therefore, design in recent years have employed higher slenderness ratio members in an effort to optimize reserve strength. This, however, has resulted in bracing systems with significantly reduced residual strength[9]. This is of great concern since it is residual strength that is a measure of the structure's ability to sustain damage without failure; in a fire scenario, <u>progressive ductile collapse of the structure may be hastened in designs employing high slenderness ratios, even though such designs may employ a higher degree of redundancy</u>.

The wall thickness modulus, D/t, is a means of classifying tubular as thin or thick wall members, and is a measure of buckling resistance. Tubular members typically used in jacket fabrication will normally buckle inelastically rather than elastically. Tubular members with low D/t ratios (60 and below) are generally not subject to local buckling from axial compression and can be designed on the basis of material failure, i.e., the local buckling stress may be considered equal to the yield stress[12].

As a rule of thumb, designers aim for D/t ratios between 30 and 60. As Marshal[13] points out, the problem of local buckling in tubular compression members and of achieving sufficient rotational capacity can be largely avoided in offshore designs by simply resorting to relatively compact sections -- those having an upper D/t limit of about 50 depending on loading conditions. Less compact tubular members will have limited curvature and rotation capacity beyond the peak strength which can lead to fairly rapid loss of moment carrying capacity. Hence, <u>thinner-wall tubular members are particularly sensitive to failure from</u>

local buckling, either due to fabrication defects or thermal impact.

Zayas, Mahin, and Popov[11] found that under cyclic load testing, tubular braces with lower D/t ratios retained a greater percentage of their original strength with repeated cycles. They report that significant loss of lateral load capacity is mainly associated with the deterioration of brace strength, and that low slenderness ratios of the brace members and low D/t ratios are important factors in achieving good cyclic inelastic behavior of the braced frame. Their report concludes that braces and frames with lower D/t ratios exhibited significantly less deterioration in strength, stiffness and energy dissipation than those with higher values.

Members with D/t ratios under about 25 are considered thick walled and will not float. Consequently their use offshore has been limited to date. However, members with low D/t ratios have much greater inherent thermal mass and fire endurance than thinner wall members, and may find greater use in the future for above-waterline applications for reasons of their increased thermal robustness.

It is well known through repeated testing that structural steel columns must be insulated to prevent failure temperatures of approximately 1000°F under fire exposure conditions. Fire tests performed by Underwriters Laboratories show that for slenderness ratios from 40 to 112, the failure temperature is approximated by the formula:

$$(1040 + 1.8[1/r]) \pm 50F$$ [14]

Research sponsored by the American Iron and Steel Institute (AISI) on the fire endurance of steel columns led to the following empirical relationship developed by Stanzak and Lie[15]

$$T = 10.3(W/D)^{(0.7)} \text{ for } W/D < 10, \text{ and}$$

$$T = 8.3(W/D)^{(0.8)} \text{ for } W/D > 10$$

where T = time in minutes for the column to reach 1000°F

W = linear density of the column in lb/ft

D = heated perimeter in inches

Failure was deemed to occur when the column cross-sectional area reached 1000°F (the temperature at which steel loses about 60% of its room temperature tensile and yield strength, which is the AISC limit for the maximum permissible design stress[16]. The equation was developed for solid columns exposed to a time-temperature curve closely following the ASTM E-119 rate-of-rise time-temperature curve for cellulosic fueled fires.

A study was done by Aramco on the fire resistance of unprotected steel legs for offshore platforms[15] based on the Stanzak and Lie equations. Aramco engineers recognized that there were several additional considerations that should be accounted for in making a determination of the inherent fire endurance of platform legs. The fire demand based on ASTM E-119 was recognized as not being representative of hydrocarbon fueled fires, which have much higher rates of heat release and temperature rise. Also considered, however, was the fact that in an outdoor environment, a much greater quantity of heat will be lost to the surroundings as compared to a furnace test fire environment.

Aramco used the Stanzak and Lie heat transfer relationship to calculate the inherent fire endurance of platform legs as shown below (minutes to failure) and then applied compensating factors to account for actual field conditions.

Column Diameter	Wall Thickness	W/D(#/ft.-in.)	Time
42"	1.00"	3.23	23
42"	1.25"	4.13	28
42"	1.50"	4.90	32
48"	1.50	5.10	33
71"	1.00	3.34	24
71"	1.50	4.87	31

Aramco reasoned that because platforms legs are in the open, the time values are conservative to a degree approaching 75%; however, no data was offered in support of this supposition. Aramco concluded that unprotected self-supporting columns legs with 1.50" thick wall sections can withstand fire offshore for approximately one hour. This also allows for a 25% increase in fire resistance due to the thermal conductivity of steel and heat sink effects for support legs immersed in seawater. This additional allowance was stated as assumptive in recognition that the "exact value" would require further testing.

It is interesting to note that based on this work, Aramco decided to modify their engineering standards for the construction of offshore platforms so as to <u>not require</u> the provision of fireproofing support legs. They reasoned that if the column leg wall thickness is at least 1.5", then reliance solely on inherent fire resistance is justified. However, if the leg wall thickness was in the range of 0.75" to 1.00", the fire resistance (endurance) is only in the order of 25 to 30 minutes. However, Aramco rationalized that 30 minutes endurance is still sufficient if other parts of the structure are unprotected, because the deck supports would fail prior to the legs, whereas if the deck support system is protected, such as by water spray, then additional protection for the legs may be warranted.

Another way of expressing Aramco's conclusion can be in terms of the wall section modulus. For a 42" diameter leg, where t = 1.0", D/t = 42, whereas where t = 1.5", D/t = 28. This difference accounted for an increase in fire endurance of from 23 minutes to 32 minutes, or an increase of 39% under test conditions.

The W/D ratio is a means of expressing inherent fire resistance and is employed by Underwriters Laboratories[17] for normalizing structural steel fire resistive ratings for fireproofed members. Members having a greater W/D ratio than the rated member size (for a given thickness of fireproofing) are considered larger than the specified minimum size required to realize the desired degree of fire resistance. For example, a design calling for 2" of fireproofing on a W10 x 49 to achieve a 2 hour fire resistive rating could also utilize a W10 x 228, since the heavier member has a higher W/D ratio, i.e., it has greater thermal mass. Conversely, if the design was tested for a given fireproofing configuration using a W10 x 228, then a W10 x 49 could not be substituted without testing, since the specified thickness of fireproofing may not provide sufficient protection for the member.

While the W/D ratio is useful, it has several inherent limitations. Fraser[18] points out that test methods and rating systems that utilize the W/D ratio for correlation fail to account for the geometry of the structural member or how it is used. Also there is no method to translate fire ratings from one structural shape to another, such as tubular members.

These shortcomings are resolved by the British Hp/A section factor rating system[19]. This approach uses the perimeter of the section exposed to fire, Hp, divided by the cross sectional area of the steel member. In fact, the section modulus, Hp/A is analogous with the inverse of the W/D ratio, and will vary between different size members in the same proportion as the W/D ratio. The reported advantage of the Hp/A approach is that the British have developed values for this modulus based on extensive fire tests for most standard structural shapes, including tubular members. Also this method has been adopted in recent years for hydrocarbon fueled fire scenarios and is now accepted by offshore operators and regulators in the North Sea.

The British test work has demonstrated that the same critical temperatures can be used to analyze both standard column and beam shapes and structural hollow sections (SHS) or tubular shapes. Fire resistance tests run to the British test standard BS 476, Part 8, demonstrate that for a fully stressed unprotected steel section, columns exposed on four sides that have a section factor, Hp/A, of up to 50 m^(-1) can achieve a ½ hour fire rating.

Hp/A for tubular is simply the outer circumference of the member divided by the cross sectional area:

$$Hp/A = 12.56\ (O.D.)/[(O.D.)^2 - (I.D.)^2]$$

As a basis of comparison, using the data from the Aramco test work on a 48" platform leg with a 1.50" wall thickness, the corresponding section factor is calculated as:

$$Hp/A = 12.56(1.22m)/[1.22m^2 - 1.14m^2] = 81.2/m$$

Since the section factor exceeds 50/m, the fire endurance of the 48" legs would appear to be less than the one half hour resistance indicated by Stanzak and Lie's correlation. In order to reach a section index of 50/m, the 48" diameter leg would require a wall thickness of approximately 2.6", or a D/t ratio of about 18.5, thus requiring a "heavy wall section" as previously discussed.

Recent Tests on Tubulars

In May of 1989 following the Piper Alpha disaster, Shell U.K. Exploration and Production Ltd., and Shell Research Ltd. "urgently launched" a test program to investigate the behavior of full size structural members (both tubular and standard shapes) in high pressure hydrocarbon fueled jet fires[20]. The tests involved exposing structural members to direct flame impingement from a sonic release of high pressure (882 psig) natural gas, and included both unprotected and fireproofed test specimens.

An unloaded and unprotected 18" diameter tubular member with 0.5" thick walls was tested in a horizontal configuration, simply supported on rollers. The member was located 8.9 meters in front of the jet orifice which produced a 20 meter long flame. The temperature of the member initially increased by approximately 5.9 degrees F/sec. until a steady state temperature of 1850°F was reached in 16 minutes. It was observed that the unprotected tubular member began to sag under its own weight at 12 minutes into the test. The final deflection was approximately 150mm. It was concluded that if the specimen had been part of a normally loaded structural system, its loss of strength could have resulted in collapse (depending on the degree of system redundancy and available alternative load paths).

For comparative analysis, the unprotected 18" diameter tubular member had a D/t ratio of 36 and a section factor of 255/m. In order to attain a section factor of 50/m, the wall thickness would need to be approximately 1.1", making the D/t ratio about 16.4. However, it should be kept in mind that an Hp/A of 50m^(-1) has not been as yet validated in such a severe test environment, and may prove to not provide 30 minutes of inherent fire resistance. This will require further field work.

Graff[8] offers the following practical suggestions for platform designers. For small diameter braces, up to 18" in diameter, the wall thickness should correspond to standard pipe sizes at the starting point for design analysis. For diameters approaching 30 in., the brace wall thickness should initially be taken as ½ in.; and for diameters from 30-36 in., start the design with a 5/8 in. wall thickness. The inherent fire endurance of such designs can be expected to be significantly less than 30 minutes.

There are several reasons why designers seek to minimize brace sizes, in addition to lowering the drag forces and loadings on the structure. It is important to understand these design objectives in order to arrive at meaningful approaches to increasing inherent fire endurance through greater thermal mass. Designers will often use high strength low alloy steels or seek other means to minimize member sizes. Smaller members and higher D/t ratios mean easier welding and less structural weight, both of which equate to significant cost savings during fabrication and transportation.

Increasing D/t ratios, however, need not significantly affect platform initial capital costs, and can result in significantly lower life-cycle costs by providing a safer platform with lower accident-replacement costs. The portion of the structural system impacted is only that portion above water line, which may prove to be much less than 5% of the total structural steel in the jacket for a deep water platform. And, as noted by Graff,[8] the lower portion of the intermediate column range ($30 < kl/r < 60$) could be used for tubular braces, but he finds it difficult to explain why this is not done more often. He suggests that perhaps because normally the designer first chooses the diameter of the jacket leg, and this choice restricts the diameter of the brace since most braces are less than 70-80% of the diameter of the jacket leg.

In any event, it is apparent that both member sizing and structural configuration have significant impact on both inherent fire endurance and overall system reserve and residual strength. However, at present, design decision regarding these critically important aspects of platform safety are left to the discretion of the designer who has no methodology or fire-based performance targets to guide the decision making process.

Hypothesis #1

> Steel jacket-type offshore platforms can realize significantly greater **inherent** fire endurance solely through the application of fundamental design considerations. Specifically, the fire resistance of unprotected steel members, and the endurance of overall system capacity may be enhanced by 1) limiting D/t and kl/r ratios of critical above-water braces to a maximum of 30 and 60 respectively, and 2) opti-

mizing structural configurations of the framing system using X-frame configurations to maximize residual strength.

The offshore platform fire problem can be thought of as two sides of an equation, the fire demand side and the fire response side. It is the analysts' goal to characterize the demands and achieve a cost-effective design that has an overall thermal response behavior that meets performance targets. This requires defining fire-based structural performance criteria and providing sufficient system residual strength and structural robustness to meet the demands for the determined acceptable level of risk.

It is proposed that for all multi-wellhead steel jacket platforms operating on the OCS, **there should be some minimum level of engineered structural endurance provided, either solely as an inherent property of design and configuration, or in combination with some degree of passive fire protection.** Simply stated, there must be adequate thermal robustness in the design to provide the time (endurance) required to accomplish critical tasks, e.g., platform shutdown, fire-fighting response, disembarkation, etc.

In the case of a major fire incident, a primary object is to be able to maintain the structure's integrity for a sufficient period of time to permit fire and damage control measures to arrest continued deterioration of capacity and progressive failures, while permitting evacuation of operating and maintenance personnel. This will involve analysis of strategies and alternative approaches, with the context of defined restraints and service requirements.

Designing the required degree of thermal robustness will require appropriately placed redundant elements, (provision of ductility ability to re-distribute loadings), and excess capacity (ability to withstand increased loadings), i.e., fire-based structural design criteria. The inherent limit of a structure's fire resistance as a function of member sizing and configuration must be **baselined**, i.e., defined and understood, before criteria for additional mitigations, such as fire resistive coatings, can be rationally specified.

Structural Fire Endurance

There are three general approaches to determine the fire resistance of steel members and systems: 1) empirically derived correlations, (e.g. W/D ratios), 2) heat transfer analyses, and 3) structural analyses[21]. Using these approaches, the fire endurance of platform jackets and module support trusses (frames) can be analyzed and predicted. The basic methodology has been applied to buildings as presented by Bresler and Iding[22], [23] and more recently has been used offshore (see Appendix A)

The approach to analyzing platform structural fire endurance can be broken down into five basic steps: 1) characterization of the offshore fire demands in a reliability based format, 2) identification of the thermal and structural characteristics of the design being evaluated (this included both member and system characterization), 3) analyzing the structure's design configuration, boundary conditions (restraint), and the impact of fire-proofing (initially only unprotected steel members and systems will be analyzed to base-line inherent fire endurance as a function of configuration, 4) numerical discredization and solution for temperature distribution and thermal response (deflection, creep, etc.), and 5) comparisons with failure criteria as determined by utility demands.

Critical Temperature

From a parametric perspective, time and temperature are the two primary variables that must be analyzed to determine structural fire endurance. The fundamental question that must be answered is how much time is required (available) for a specific structural design configuration to fail under any given fire load. This in turn requires knowledge of the time-temperature history of any given structural element or group of elements under fire exposure. Failure can be deemed to be when a specific average or maximum temperature is reached in the element, or when deflection(s) has (have) exceeded specified limits. The critical temperature is defined as the temperature of the steel at which its material properties, specifically the modulus of elasticity and yield strength, have decreased to the extent that the steel member is no longer capable of carrying a specified load or stress level[21].

The critical temperature can be calculated knowing how the material properties vary with temperature. The critical temperature is usually taken to be 1000°F since, as previously mentioned, this is the temperature at which most steel lose about 60% of their room temperature tensile and yield strength, which is the AISC limit for the maximum permissible design stress.

If a platform is designed using a factored load limit state format, then the critical temperature range must be accounted for as a probability distribution, and the statistical distributions for both the changing loads (due to thermal expansion and creep) and changing resistances (due to loss of strength and stiffness) must be accounted for. In effect, for a progressive ductile collapse scenario, one must account for a reliability index that is changing as a function of time[24].

Numerous approaches have been developed for calculating the critical temperature of steel members during fire exposure, including several computer programs[21]. One such program, FIRES-T3, originally developed by Iding, Bresler, and Nizamuddin[25] at the Department of Civil Engineering at the University of California at Berkeley has been experimentally validated by Jeanes and Milke[21] and is widely accepted. Current work includes models developed by SINTEF (see Appendix A)

<u>Characterizing the Offshore Platform Demand</u>

Task II will address characterizing the offshore platform fire demands in a reliability format expressed as a family of risk-based fire demand curves. The fire demand can be explicitly represented in terms of a return period similar to platform environmental loads such as a 100 year storm wave. Figure 5 illustrates an arbitrary set of risk-based fire demand curves in terms of heat flux v. duration. The top curve represents a 100 year fire which is deemed to be a low probability -- high consequence event such as the failure of a production riser below the lower deck, i.e., Piper Alpha.

Similarly, a higher probability, lower consequence event is represented as a 50 year fire such as the failure of a pump seal on a hot oil pump. The same 50 year fire curve is shown, but with mitigating factors accounted for such as an automatic water spray system installed above the pump. The corresponding fire demand is reduced accordingly, reflecting the ceasing intensity of each fire scenario.

Hypothesis #2

> Offshore platform fire demands can be characterized similarly to environmental loads in terms of a return period, e.g., a 100 year fire. Each return period fire demand has its own time-temperature/heat flux relationship such that the overall risk of fire has an associated consequence (fire demand) a specific probability of occurrence. Further, the range of fire risks can be presented as a family of risk-based fire demand curves for the particular operating and design characteristics (risk factors) of any offshore platform.

Each platform must be evaluated for the possible range of fire demands. A quantitative risk assessment should be performed accounting for the many variables that go into a consequence analysis and risk evaluation such as wellhead pressure, reservoir characteristics, nature of operations, corrosiveness of the production fluids, etc. New development fields that have no historical basis for evaluating the risk of blowouts must include a high degree of uncertainty in the analysis.

For a small platform in shallow waters with one or two low pressure wellheads, the family of fire demand curves can be expected to impose lower thermal loads than those of larger complex platforms handling large volumes of high pressure fluids. Design base fires (DBF's) using hazard and operability (HazOp) techniques can be helpful in identifying and evaluating the range of fire scenarios. For the purposes of this report, a family of fire demands curves has been developed to illustrate this proposed methodology. Figure 6 presents steel failure criteria v. heat flux levels for both jet flame and pool-type hydrocarbon fires[26].

The probability of structural system failure depends on the size, duration, and location of the design base fire, e.g., Pf/DBF. In its simplest form, failure can be expressed as exceeding the limit state:

$$g(X,t) = R-S \leq 0$$ [27]

where X represents several random variables associated with the fire demand and structural response models, and R and S are time dependent capacity and load effects.

Overall system reliability cannot be addressed until the fire demand curves have been developed. Figure 7 illustrates how the structural reliability can be expressed in terms of fire demand fragility curves. The cumulative probability distribution for system failure from progressive ductile collapse Pf[system] is calculated by summing the conditional probability of failure for any given fire demand times the probability of realizing that demand[24]. This same approach has been used by Bea to calculate the impact on platform reliability of design errors[28]

Fire severity can be treated as a random variable with an exponential probability distribution[29]. Further, structural fire resistance in general may be regarded as a log-normal or exponential probability distribution. This allows for the application of a convolution integral, or the stress-strength model as it is frequently referred to in reliability theory, to analyze the degree of fire resistance needed to meet performance targets for various levels of fire severity[30,31].

Thus the probability that the fire resistance, R, is greater than the fire severity, S, can be expressed as $p\ (R \geq S) = p(R/S) \geq 1 = p(Z) \geq 1$ where $Z=R/S$ and $\ln Z = \ln R - \ln S$. Since the linear combination of independent random variables tends to be normally distributed regardless of the distribution of R and S, the distribution of X = ln Z is a normally distributed random variable, thereby allowing the probabilities to be obtained from a standardized normal distribution table.

Failure

Failure must be described in terms of a limit state in order to be meaningful. The simplest generalization for steel structures is in terms of temperature; any single point on a member that reaches 1200°F or when the average temperature reaches 1000°F constitutes failure in ASTM E-119, where Tavg. = ¼ (Ttop flange + Tmid web + 2Tbottom flange)[10]. Lie and Stanzak determined that the critical temperature for slender axially loaded steel columns is about 940°F[32] based on the Euler critical elastic buckling equation, as previously noted.

More explicit performance criteria based on midspan deflection and the rate of change, as developed long ago by Ryan and Robertson[33] is preferred by Bresler and Iding[10]. The following limit state is often used to connote failure of an end-supported beam or floor/roof assembly subject to a standard fire test.

Failure occurs if $D \geq (L/800)(L/d)$ and $R \geq (L/150)(L/d)$ where D is midspan deflection in inches, R is the hourly deflection rate (in/Hr), L is the span length in inches, and d is the distance between the upper and lower extreme fibers of the member in inches.

The issue of critical temperature vs. deflection as a failure criteria was recently studies by Skowronski[34]. He compared deflections of steel beams having the same calculated critical temperatures but differing in length and load values. Next he compared the critical temperatures and deflections of beams having the same length but different loadings. Skowrosnki found that critical temperature for short beams, which results from the ultimate load bearing capacity, is lower than the one resulting from the limit state of deformation of these beams. The critical temperature of long beams, which results from the ultimate load-bearing capacity, is higher than the one resulting from the limit state of deformation.

As Chen [7] notes, the behavior of a beam-column can best be described by its axial load-lateral deflection behavior. Skowronski[34] concludes that there is a beam length for which the critical temperature resulting from the ultimate strength is the same as the critical temperature resulting from the limit state of deformation. This suggests that there may be two different failure criteria appropriate to apply to offshore platform beam-columns as a function of the slenderness ratio. Braces of high kL/r values may in fact "fail" (reach limit state deformation) at temperatures lower than the calculated critical temperature due to excessive buckling.

Residual Strength and System Capacity

Lloyd and Clawson[9] have shown that residual strength is a consequence of redundancy, and, with increasing member slenderness ratios, the reserve strength of the system is increased but the residual strength is decreased. Residual strength is necessary to meet platform reliability targets and utility goals in a damaged condition. Two questions that must be addressed are how much residual strength is needed and how can this be effectively achieved.

Redundancy has been defined by De, Karamchandani and Cornell[35] as the conditional probability of system failure given first failure of any member. Cornell[36] has analyzed the example platform described by Lloyd and Clawson[9] to quantify system redundancy and reliability (safety) index using system reliability techniques. This study allows us to identify system members that are most critical to maintaining structural integrity. One immediate consequence of this study is to allow designers to identify potential weak links in the system and understand how progressive ductile collapse may occur. Using this approach, system vulnerability to thermal impact may be analyzed to identify critical system components upon which structural integrity depends.

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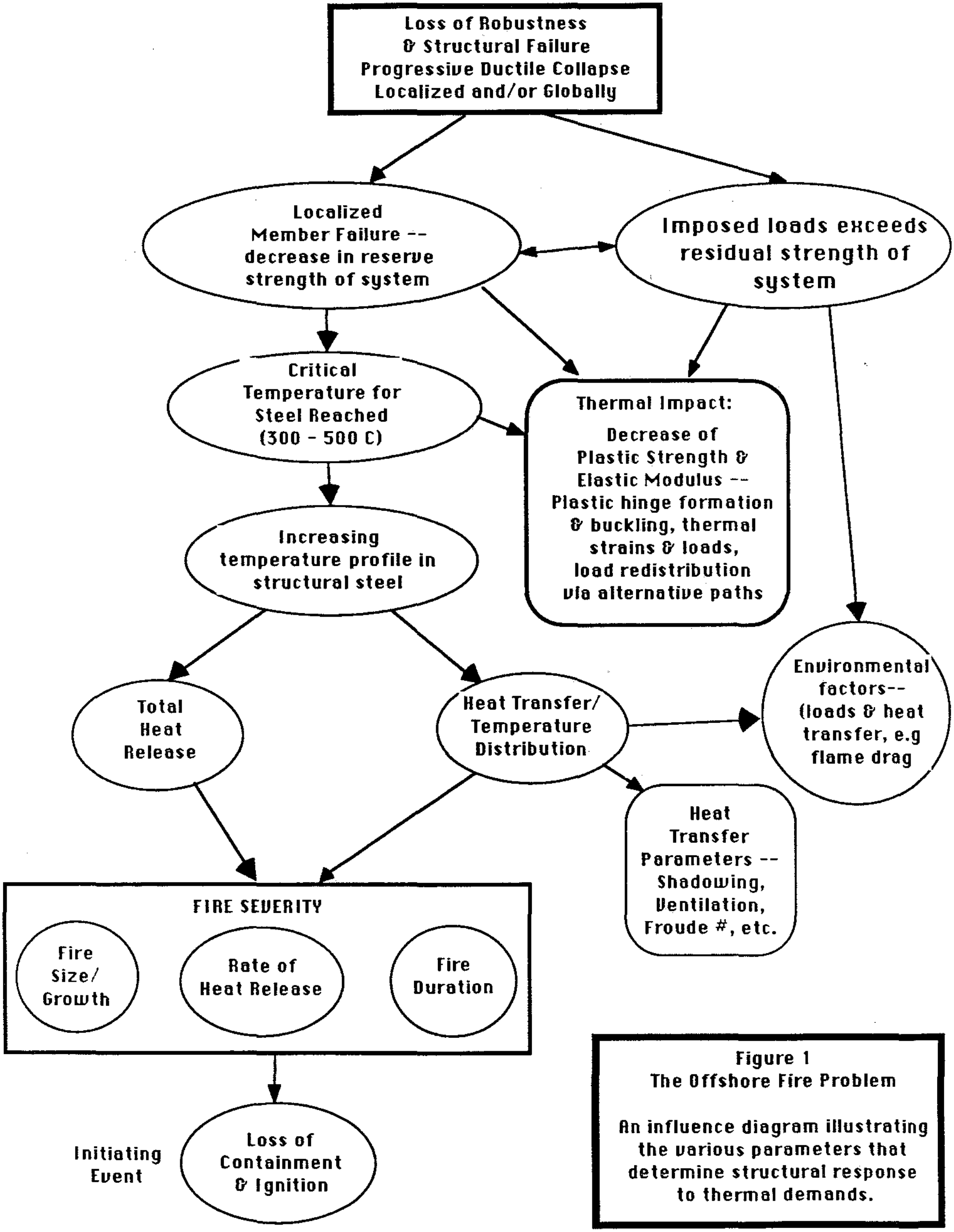


Figure 1
The Offshore Fire Problem

An influence diagram illustrating the various parameters that determine structural response to thermal demands.

Figure 2
Idealized platform performance
Intact and damaged

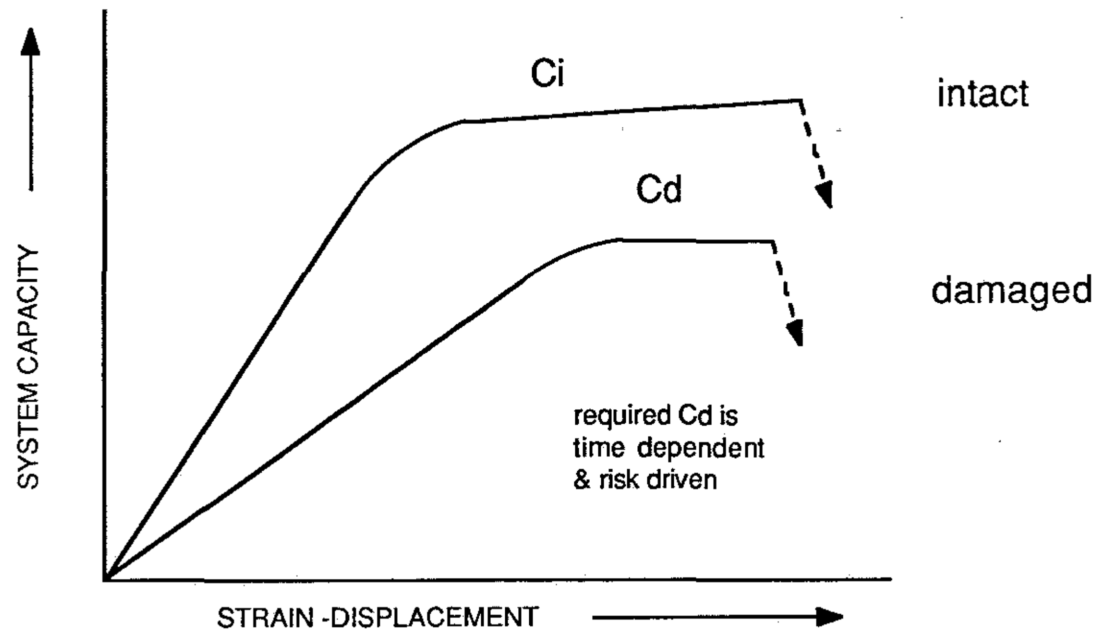
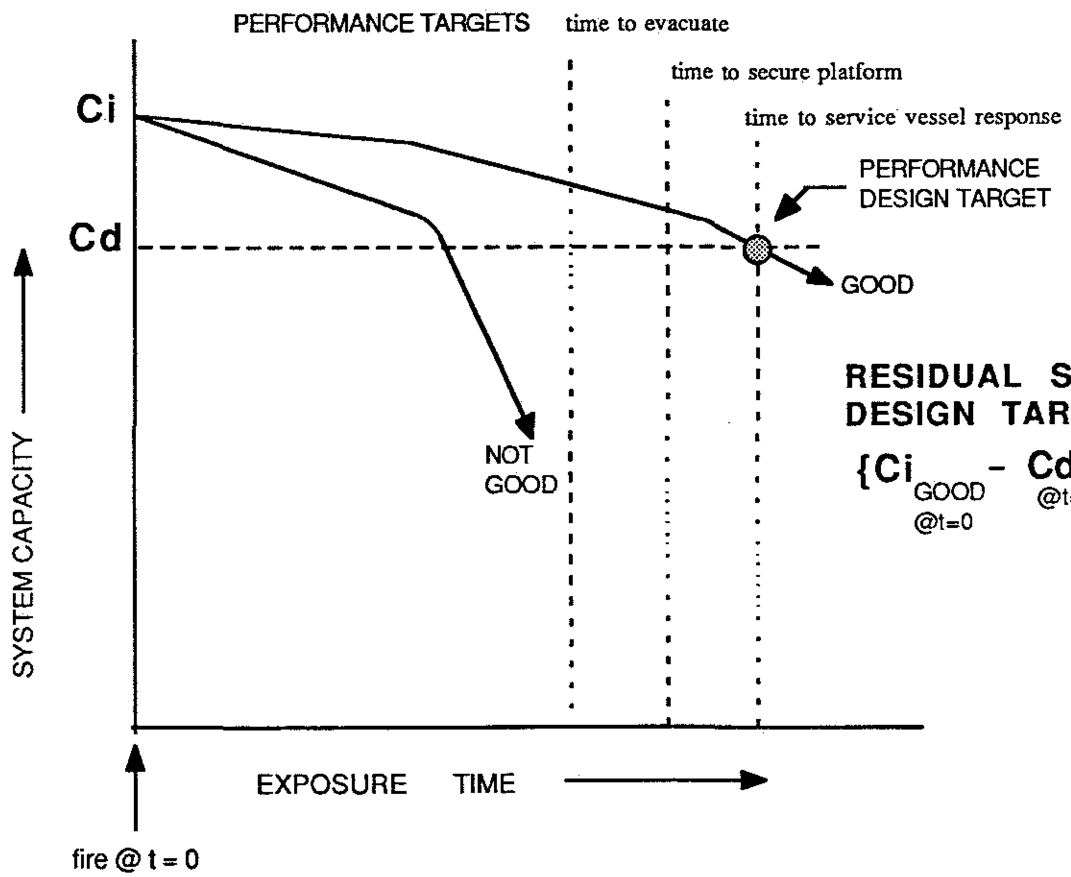


Figure 3
Required residual strength to meet performance targets



RESIDUAL STRENGTH DESIGN TARGET

$$\{Ci_{\substack{GOOD \\ @t=0}} - Cd_{@t=0}\} = R_{fire}$$

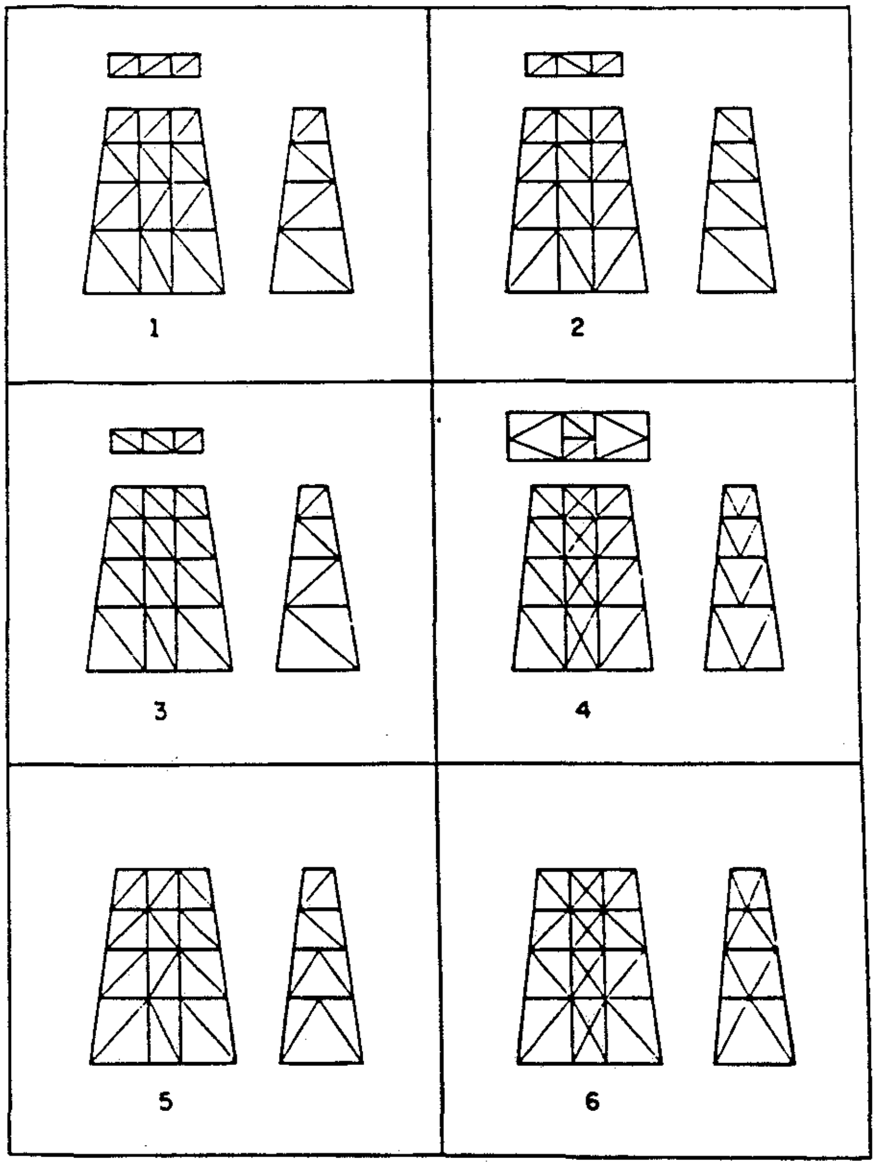


Figure 4

Commonly Used Truss Design for Offshore Platforms

Structural Design
for Fires on
Offshore Platforms

Problem Characterization - Thermal Intensity

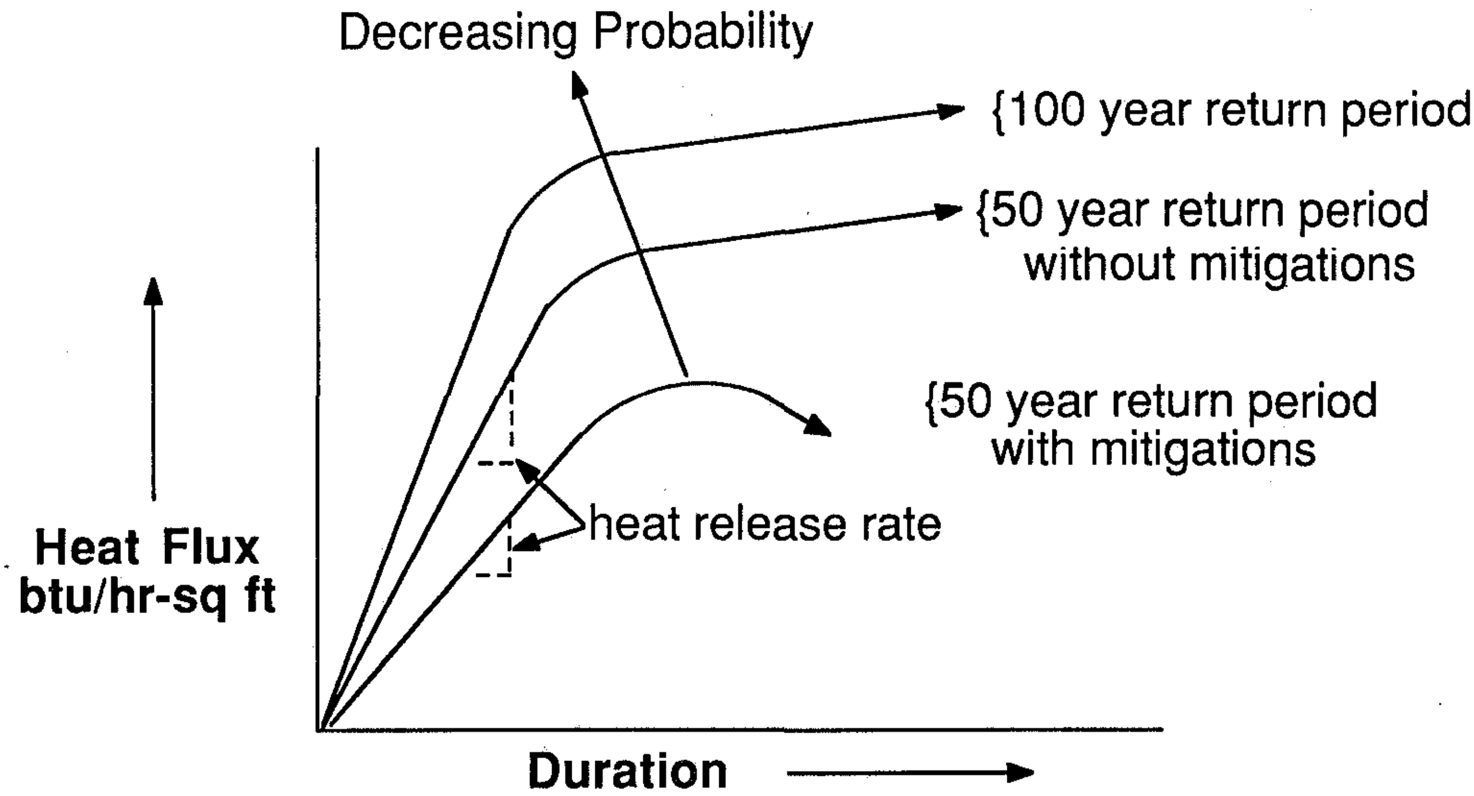


Figure 5

Criteria for the failure of steel members, firewalls, and risers under hydrocarbon fire impact are shown below. They are derived from various sources (Technica) for the Piper Alpha investigation.

Under explosion overpressures, the following criteria was established:

- Firewalls and steel walls are blown out at 2 psi
- Decks are blown out a 9 psi
- External module walls are punctured at 1.5 psi
- Process equipment within modules is ruptured at 5 psi
- The column and tubulars are heavily damaged at 15 psi
- Structural failure of a platform could occur due to an explosion of a vapor cloud located below the module support frame, i.e., surrounding the jacket. A vapor cloud explosion occurring above or beside the upper deck level would only be expected to severely impact the topsides.
- No structural impact is anticipated from a "flash-fire."

<u>Heat Flux Endurance (minutes) of Structural Elements</u>

Element	Jet Fire Flame (high-rise)	37.5 kw/m^2 Jet Fire (test)	Pool Fire Flame(hi-rise)	37.5 kw/m^2 Pool Fire (test)
unprotected load bearing structural steel beam	10	20	10	30
unprotected nonload bearing steel plate	5	10	10	30
A60 Firewall	10	20	30	60
A60(H) Firewall	15	30	60	120
H120 Firewall	60	120	120	240
Protected structural steel beam	15	30	60	120
Riser	10	20	10	30
Jacket Leg	15	60	30	120

Figure 6

Steel Failure Criteria[26]

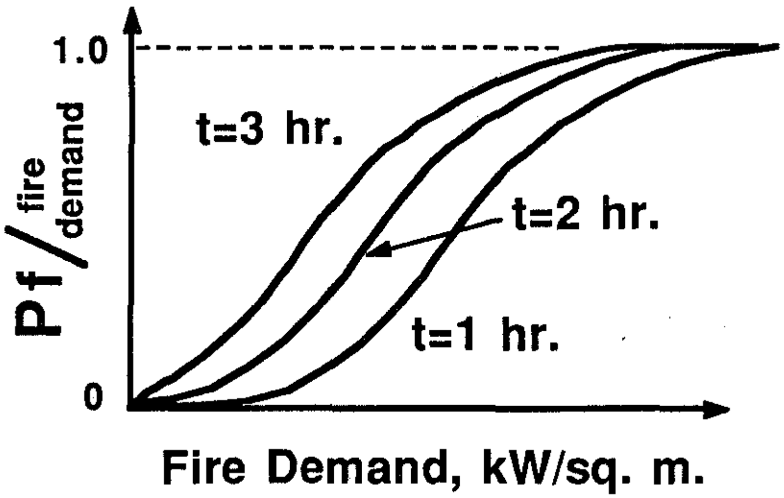
Structural Design for Fires on Offshore Platforms

System Reliability $Ps = 1 - Pf$

To develop Pf, we can use:

Fire Demand Fragility Curves

$$Pf[system] = \sum Pf/^{\text{fire}}_{\text{demand}}\,[p(fd)]$$



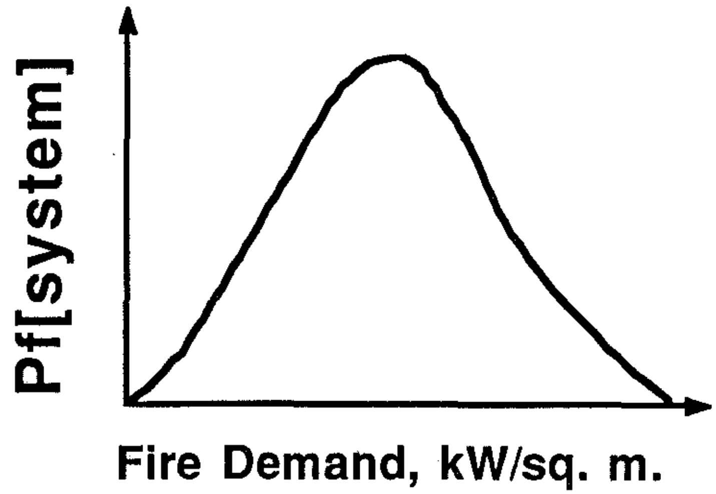


Figure 7

APPENDIX A

SURVEY OF CURRENT RESEARCH

During this past year, several interesting research papers on offshore platform fire analysis have been published that will be briefly summarized in the following section of this report with the intention of highlighting new analytical approaches to assessing the offshore fire problem.

1) Jacek T. Gierlinski, Chris I. Middleton, and Paul J. Schofield of WS Atkins Engineering Sciences Ltd., U.K., and Michael J. Baker of Imperial College, London, "Novel Approach to the Assessment of the Fire Resistance of Offshore Structures", Offshore Mechanics and Arctic Engineers Conference of the American Society of Mechanical Engineers, Vol. II, Safety and Reliability, 1991.

This paper presents a new methodology that combines probabilistic and deterministic techniques for assessment of structural response of steel jacket platforms to fire. The approach employs both elastic and nonlinear analysis based on a computational technique called the Virtual Distortion Method (VDM). VDM allows stochastic applications of nonlinear analysis by permitting numerical efficiency of a much higher order than that of traditional incremental/iterative approaches, and has been used for reliability analysis of platform structures under various environmental loadings.

The researchers model fire growth, intensity, and the temperature distribution history and corresponding material property variables treating them as stochastic processes, but with a simplification based on a quasi-stationary approach in which fire growth is approached as a deterministic time-independent sequence where temperature is considered a random variable, i.e., the fire scenario is discretized into a series of time-step temperature increments. Calculation of times to structural failure for the primary and secondary load paths are made taking into account the non-linear behavior of three thermal loading characteristics: thermal stresses, reduction in stiffness modulus, and reduction of plastic strength. Failure criteria utilized include displacement of selected nodes, spread of plasticity as indicated by the number of plastic hinges formed, and measure of total collapse, e.g., the determinant of the system matrix. Advanced Monte Carlo simulation is employed for reliability analysis to assess alternative load paths and reduce the variance of the estimate of failure probability.

Linear-elastic and non-linear analyses for elasto-plastic collapse analysis are accomplished using the ASAS package of programs which is based on the VDM approach and includes buckling and shear failure modes as well as conventional plasticity criteria per BS 5950. Platform reliability calculations were performed using RASOS. The research was performed as part of the BRITE Project P1270" Reliability Methods for Design and Operation of Offshore Structures. This work was funded from the Directorate General for Science, Research and Development of the Commission of the European Communities in conjunction with unidentified industrial sponsors.

2) Sergio Rodriguez of Petrobras, Fernando Torres and Marcelo Mendes of PENTA, "Structural Analysis in Offshore Platforms Due to Fire Accident" together with a companion paper by Marcia Araujo of Petrobras and Marcelo Mendes and Fernando Torres of PENTA, "Temperature Distribution in Offshore Platforms in the Case of Fire," papers presented at the 1991 (First) International Offshore and Polar Engineering Conference in Edinburgh, U.K., August 1991.

These two papers discuss the analytical procedures and techniques recently developed by Petrobras (Brazil) and PENTA for assessing fire impacts on the new Petrobras VIII production platform, a large single deck floating production system (semi-submersible) for deepwater production in the CAMPOS basin, offshore Brazil. The researchers used 3-D finite element models to determine the temperature distribution resulting in the structure and topside equipment from fire exposure scenarios, and the associated structural responses. Fire scenarios were deterministically developed based on a risk analysis; the fire duration was then calculated for a normalized fuel release rate that provided the maximum thermal demand based on the inventory of fuel within equipment, with the caveat that ESD and depressurizing systems function. In this manner, the researchers felt that they could identify the worst-case credible fire demand (based on personal conversation between Marcelo Mendes and

William E. Gale at ISOPE 91). The temperature distribution model developed was comprised of 3585 shell elements and 4535 connecting joints.

Once the temperature distribution for the structure was calculated, it was applied to an elasto-plastic finite element model to evaluate structural response. Beams were represented by frame elements having six degrees of freedom per node. Plate stiffness between beams was accounted for by increasing the cross-sectional area and the moments of inertia. In-plane stiffness was also modeled since this was recognized as important to account for thermal expansion effects. The structural model was composed of 1096 beams, 66 tubes, 358 truss elements, connected by 962 nodes.

To model plastic behavior, a kinematic bi-linear hardening model was developed, accounting for plastic strength at elevated temperatures. Failure criteria was based on maximum displacements and stress levels corrected for nodal temperatures.

As a result of this work, the researchers concluded that much of the passive protection previously being specified in Petrobras standards could be deleted at considerable savings in weight and cost. However, by way of critique of this work, we offer one note of caution: the fire scenarios developed by the optimization approach developed by the researchers appears to result in unrealistically low fire demands, at least when compared to a typical steel jacket production platform.

For example, the researchers determined that the flame length from a leak in a "gas vessel" operating at 11.4 atm. would produce a flame almost 2 meters high, and similarly a failure in an oil and gas separator would have a flame length of 8.4 meters high. The largest flame size used in the analysis was 18 meters resulting from a hole in a 100mm gas riser operating at 110 atm., corresponding to a 4mm hole. As explained to William Gale by Professor Mendes, the postulated leak rates and flame sizes result in fire scenarios that purport to cause the greatest thermal demand on the structure, and while larger leak rates may produce larger flames, their shorter duration would result in an overall lower thermal demand (e.g., the integral of the time-temperature curve would be less). However, we remain skeptical of the resulting flame length derived from this approach, and must question the assumptions used in developing the thermal design demands employed in the analyses.

3) Jun Xu and Rolf Kirkvik of Aker Engineering, Norway, "Design Against Explosion Loads in Offshore Structures," presented at ISOPE-91 in Edinburgh, U.K., August 1991.

This paper is included for review for reasons of structural analysis techniques that can also be applied to thermal demands. The Sleipner A topsides were extensively analyzed for explosion loads using nonlinear structural analysis techniques that accounted for ductility. In addition, the effects of fire after a blast wave were also analyzed. The tests applied were that the structure should not suffer a global collapse from explosion, and that after local damage has been incurred, sufficient residual strength should be maintained to resist thermal impacts. The criteria consider that "under explosion loads, it is expected that a properly designed structure will suffer damage, but will not collapse, cause loss of human life, or pollution due to the ductility of its members.

Included in the analyses is an assessment of topside piping systems to dynamic blast wave effects, as well as fire walls and structural members. Of particular interest are the scenarios in which interference between process piping and wall penetrations during periods of maximum deflection can lead to overstress and loss of containment, as may have occurred in the Piper Alpha incident.

Two blast response analysis methods were considered: simple response analysis methods used to predict the reaction of deck structures and components such as beams and plates where boundary conditions are well established; and nonlinear finite element methods where blast wall and piping system failure modes cannot be predicted with sufficient accuracy using simplified approaches. The program package ABAQUS was employed for FEM 3-D analysis which allows updating of the structural stiffness matrix according to plasticity and geometric effects.

Failure analysis was based on two types of criteria: fracture criteria and deflection. Ductility criteria for both steel and thermal insulation was considered. This is important for fire wall analysis, since the thermal insulation critical strain was about 6% compared to 20% for steel.

Hence deflection for fire walls must be checked to ensure that the thermal insulation will remain inplace after a blast, exactly when subsequent thermal demands are most likely to occur.

One of the results of the analysis was that the central deck beams of some modules were connected to the corresponding beams of adjacent modules in order to maintain continuity instead of retaining a system of simply-supported beams without axial constraint, thereby considerably reducing blast deflections. It was found that the structural response is very sensitive to strain hardening, and that this in fact accounts for the retention of structural integrity in the plastic regime. Without accounting for hardening, the analysis would unrealistically indicate that plastic deformation would expand rapidly and the deck beams would collapse.

This paper clearly demonstrates the importance of accounting for the nonlinear behavior of the structure in fire and blast analysis and offers the following suggestions: designers can take advantage of the inelastic behavior of structures by increasing the axial restraints of the elements, the ductility of thermal insulation, and increasing the in-plane bending stiffness of deck beams by using rectangular box section beams or creating a continuous diaphragm as previously described. Blast wall designs should include connecting the walls to the main structure with out-of-plane as well as in-plane restraints. Also the point is made that the ductility at welding joints and the critical strain of insulation materials must be considered when designing for fully ductile structures.

The Sleipner A topsides was initially designed to withstand a general overpressure of 0.15 bar. After a detailed risk analysis, it was realized that some areas of the platform could in fact be subject to pressures more than four times this, and new criteria were developed for a maximum overpressure of 0.7 bar for certain main deck areas and blastwalls. In subsequent conversation with Rolf Kirk-vik and Jun Xu at ISOPE-91, William Gale was told that the new Hibernia platform project (Nova Scotia) will employ a topsides blast overpressure design criteria of 1.2 bars, over a 71% increase from the revised criteria used in the Sleipner design.

4) Jens Holen, Bjorn Hekkelstrand, et al., "Modelling of Hydro-carbon Fires Offshore" Final Report, STF25 A91029, SINTEF, Trond-heim, Norway, June 28, 1991.

This extensive report describes several newly developed analytical models that have been developed by SINTEF, the Norwegian Foundation for Scientific and Industrial Research at the Norwegian Institute of Technology in Trondheim. The work was initially begun in 1984, and in 1986 the Norwegian Council for Scientific and Industrial Research (NTNF) and three oil companies, Statoil, Norsk Hydro, and Saga formed a research program called "Modelling of Fire, Fire Fighting and Smoke Dispersion Offshore."

This research project is the most advanced and extensive program devoted to assessing platform thermal demands and structural responses to these demands that we have identified to date. Several important contributions have been made by the researchers, including:

- the development of KAMELEON by SINTEF for simulation of fluid flow, mass and heat transfer and its extensions, KAMELEON FIRE O-3D, a three dimensional field model for open hydrocarbon pool fires, and its counterpart, KAMELEON FIRE E-3D for enclosed hydrocarbon fires.
- the development of zone fire models CFIRE-1 and its extension, CFIRE-X by the Battelle Institute.
- the development of FISCO-3 by Intellex GmBH, a PC-based three dimensional field model for enclosed pool fires, and an extended version, FISCO-3L that treats the application of water spray in the fire zone.
- the development of large scale test facilities at SINTEF for hydrocarbon fires used for model validation.
- empirical correlations for temperature development of medium scale enclosed liquid hydrocarbon pool fires and a set of scaling rules for enclosed pool fire parameters.

The KAMELEON program set calculates numerical solutions for the basic equations of the conservation of mass, energy, and momentum, and consequently requires a powerful computer such as a CRAY XPM 28, VAX 8600 or an Apoppl 3500. A typical fire growth model for a pool fire of about 15 minutes duration can be executed with a grid of about 2000 points in about 20 - 30 hours on a VAX 8600 or about one CPU hour on a CRAY.

KAMELEON FIRE E-3D is able to model enclosed fire scenarios under different ventilation conditions. Ventilation effects on the flow field are evaluated together with the corresponding effects on fire development, temperature distribution, and smoke/toxic gas dispersion. Its companion code, KAMELEON FIRE O-3D, for open fires is able to model wind effects (flame drag, etc.) on open pool fires, and can be applied to a variety of fuels, albeit pure components. Crosswinds over a burning surface increases air entrainment within the combustion zone and turbulent mixing of the air-fuel mixture. Highly turbulent mixing promotes increased burning efficiency and higher flame temperatures, but excessive air also tends to reduce flame temperature. The program calculates the maximum fuel burning rate for given wind speeds, and can also accommodate the effects of fuel temperature (preheated fuels). SINTEF has successfully applied O-3D to analyze pool fires on the sea below a platform. The program is not able to predict fuel spread on a water surface.

CFIRE-X is a zone model that considers fire gases within an enclosure to be divided into two layers, an upper layer of hot gases and a lower layer of cooler gases. CFIRE-X solves the energy and mass balance equations, simulating fire growth with time. The fire sources may be a pool fire, a jet fire, or unignited ordinary combustible (cellulosic) materials. Conductive heat transfer through the enclosure's boundaries, e.g., walls, etc., as well as the fire source (pool) and objects in the room (heat sinks) is calculated. Conductive heat transfer is calculated by one-dimensional Fourier heat transfer equations to solve the temperature distribution at depth. Heat absorption by a compartment's boundaries and by the fuel area is considered to be a combination of radiation and convective heat transfer. Convective heat transfer is based on a Nusselt function for natural convection. For liquid pool fires, the gasification rate is calculated by the Kawamura and Mackey model which accounts for the change of vapor pressure with temperature and fuel type, in which the evaporation rate is determined as a function of the Schmidt number, the hydraulic diameter of the pool, and the velocity of the overlaying gas layer. When a liquid pool is boiling, the evaporation rate is considered to be dominated by irradiation to the liquid target which is determined by CFIRE-X's energy equation and radiation models.

Output from CFIRE-X gives all numerical data generated during a run including gas and surface temperatures, gas concentrations, and heat and mass flows; however, unlike the KAMELEON package, it does not support any graphic outputs. Another drawback is its run time. Users of CFIRE-X have to be prepared for long run times, which the SINTEF report cautions as "simply uncontrollable." For further details about these models, the reader is referred to SINTEF report # STF25 A91029.

In personal conversations with Jens Holen and Bjorn Hekkelstrand, Seksjonsleder (Section leader) for the SINTEF fluid dynamics group, William Gale was told that the two primary areas in which improvement is being pursued are 1) the incorporation of gas jet fire scenarios into the KAMELEON packages and 2) incorporation of the effects of equipment within platform modules to account for turbulence and other significant factors affecting fire simulation similar to the present capabilities of the FLACS code for blast overpressure calculations developed at the Christian Michaelson Institute in Norway. Both Holen and Hekkelstrand strongly feel that the work on offshore platform fire modelling at SINTEF is the most advanced effort of its kind in the world, and has received consistently high levels of funding over the years with an annual research budget in the area of $6 - 7MM.

5) E. M. Donegan, "The Behaviour of Offshore Structures in Fires," The Steel Construction Institute, U.K., OTC # 6637, the 23rd Annual Offshore Technology Conference, Houston, Texas, May 6-9, 1991.

This paper does not address the characterization of offshore fire demands, but rather considers material behavior and structural response to such demands. In recent personal meetings with Professor R. B. Williamson and William Gale, Emmett Donegan discussed the difficulties of modeling the structural response to fire demands with regard to attempting a time domain solution using finite element techniques, as explained in the subject paper. Donegan points out the importance of accounting for thermal strains imposed on restrained members and the associated system effects. He notes that the effect of thermal expansion is of primary importance to triangulated beam and column structures, and that an axially restrained compressively loaded column will fail at a temperature which may be less than 300°C. Locally heated elements experience a reduction in axial stiffness compared to cooler parts of a frame which can make the effective restraint greater than otherwise expected, leading to premature failure.

Donegan discusses the use of linear elastic analysis for progressive collapse calculations and recommends that the removal of failed elements (from the model) should be avoided; it is better to reduce the elastic constants to represent failure. The European Convention for Constructional Steelwork (ECCS) equations for column stability under fire-loading can be used for the basis of a fire condition "code-check". Allowable stress factors should be within the API/AISC criteria and Donegan explains that the load factors used should not be overly conservative if the basic gravity loads can be calculated with any degree of accuracy. The ECCS formula for strut stability is referenced as the basic "code-check" equation that can be used for offshore fire purposes, and a modified AISC which is comparable to the ECCS equation also can be applied for design purposes. Variation of both E and Fy with temperature allow both compressive, bending, elastic stability and compactness checks under fire load. Donegan notes that the experimental correlation curves developed by ECCS for fire performance show close agreement with the modified AISC code-check procedure.

Donegan notes that secondary steelwork supporting walkways, stairs, and other escape routes need to be studied in detail, and suggests that a reduced fire-loading for the design of these systems may be appropriate, as indicated by the Cullen Report. He points out that there is a convergence of performance (criteria) and usability from a human tolerance standpoint of around 10 to 12 kW/m^2 and that this suggests "an efficiency in design of secondary structures." Strategies which can be used to increase the fire endurance of secondary structures include increasing thermal inertia, i.e., use low section moduli, use of high temperature "fire-resistive" steels such as Ducol, NFR50A, etc., use of water filled members, and provision of water spray systems for exposure protection.

Further to conversations with Emmett Donegan, William Gale met with Dr. Graham Owens, acting co-director of the Steel Construction Institute, and Dr. Bassam Burgan, Principal Engineer, in Silwood Park in August to discuss SCI's ongoing Fire and Blast Joint Industry Project (JIP). This massively funded research project for the characterization of offshore platform fire and explosion demands and structural response was nearing the completion of Phase I. Dr. Owens who represented Dr. Jurek Tolloszko, SCI's Director as well as Project Manager of the JIP, explained that in September or October of 1991, a Phase I report is to be issued comprising newly formulated design guidelines for blast and fire demands for offshore platforms. As reported by Dr. Graham, the upper heat flux levels contemplated for a worst case fire demand is about 300 kW/m^2 based on a high pressure gas release scenario.

The next phase of the program, Phase II, is expected to be an even more massive multi-year undertaking in which large scale fire tests will be run to validate the analytical algorithms and empirical data developed in Phase I. Dr. Graham was optimistic about funding for this next effort, and felt that members of the JIP realized the importance of this work and will continue to support the research efforts. An unofficial liaison relationship was established between Dr. Graham and the University of California, Berkeley research effort, with an expression for mutual cooperation and an exchange of information among all concerned.

Conclusion

Since the Piper Alpha tragedy there has been a tremendous response from researchers all over the world seeking to characterize offshore fire demands and the associated structural response. No doubt there are other projects and developments not mentioned in this report that are noteworthy. Unfortunately, research activity on offshore platform fire endurance and fire risk management within the U.S. appears to be very limited, and significant funding for this important area of research has not been forthcoming. Our current role (Task I) is one primarily of observance rather than one of being an active participant.

In Task II, the next phase of this work, a methodology for determining the return period for thermal demands will be pursued. This is planned to be the subject of a paper to be presented at the ASME/OMAE-92 Conference in Calgary. A report on this work will be issued on June 1, 1992.

APPENDIX B

FIRE-RISK FACTORS

<u>Drilling/Production Risk Factors</u>:

- number of drilling rigs
- design of BOPs
- kill provisions
- mud tank reserve capacity
- mud pump reserve capacity
- provision of emergency DC power
- provision of mud level alarms
- gas detection system provided
- new or established area/extent of field knowledge base/delineation of reservoirs/pressure profiles/subsurface formation evaluation accuracy
- experience of drilling crew(s)
- Kick procedures followed (drillers methods, other)
- wellhead pressure & flow rates
- total platform throughput
- temperature of produced crude
- number of wellheads
- spacing between wellheads
- location of wellbay
- location of wellheads (subsurface v. topside)
- pressure drop across choke
- flowline pressure rating > wellhead shut-in pressure?
- number of platform risers
- gas/oil ratio (GOR)
- API gravity of crude oil
- viscosity and pour point of crude
- BS&W in crude
- number of stages of separation onboard platform
- operating pressure of 1st stage separator
- gas compression horsepower onboard
- design of compressor (centrifugal, reciprocating, other)
- gas dehydration scheme employed
- presence of fired process equipment
- pressure relief provisions/time to depressure
- design of flare system
- Emergency Shutdown System Design
  - surface safety valve system design
  - subsurface safety valve system design
  - failsafe features/supervisory provisions/reliability/redundancy
- design/reliability of process control system
- flammable liquid inventory capacity

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<u>Facility Fire-Risk Factors</u>:

- age of platform
- size of platform and equipment density

- topsides configuration and deck layout
- electrical area classification philosophy/enforcement
- remoteness of location/availability of offsite assistance
- percent of process area enclosed
- ventilation provisions for enclosed areas
- size of accommodation module
- structural robustness of design
- materials of construction
- shipping and metering provisions (P/L, SPM, other)
- presence of gas reinjections/gas lift facilities
- design of size/number of power generation packages

Fire Protection Risk Factors:

- design of fire protection system
  - capacity and number of fire pumps
  - location/reliability of standby pumps
  - provision of fixed fire suppression systems (water spray, halon, dry chemical, foam, other)
  - automatic fire detection system design
  - automatic fire alarm system design
  - combustible gas detection system design
  - number/size/location of manual fire protection provisions
- provision of passive fire protection
- means of egress and evacuation
- location of wellbay and platform configuration
- provision of fire walls
- provision of blast-resistant walls
- provision of explosion venting for enclosed modules
- capacity/design of deck drains/sump pile
- isolation of ignition sources and fuel sources/electrical area classification

Operational Risk-Factors:

- simultaneous operations onboard (drilling/producing/workover/construction/other)
- attended v. unattended platform
- extent of platform automation
- helicopter frequency/distance from shore
- location of platform WRT shipping lanes
- number of personnel onboard
  - normally
  - maximum
  - minimum number required to adequately staff platform
- frequency of inspections
- adequacy of inspections
- adequacy of preventative maintenance program

Human Risk Factors:

- adequacy of training
- experience/competence of personnel
- level of staffing
- management safety culture/organizational incentives/awareness
- safety procedures and practices
- management of change
- accountability and documentation
- emergency response preparedness/training

Environmental Risk Factors:

- severity of environmental conditions
- fatigue effects/stress corrosion cracking/transition temperature and loss of ductility/etc.
- sand erosion propensity/frequency of problem wells requiring rework
- distance from shore/means of transport