R506 RESISTANCE TO ACCIDENTAL AND VERY EXTREME EXPLOSIONS

Abstract

The resistance of a blast wall to accidental explosions is studied. Analysis with a nonlinear finite element code is compared with results form single degree of freedom analysis (SDOF). SDOF analysis provides quite good results and is conveniently used in parametric studies of the sensitivity to blast duration, pulse shape etc. An approach to assessing the resistance of a floating platform deck structure to explosion pressures beyond the design accidental values is described. The resistance of the deck up to the point of global collapse is explored.

Introduction

The design against explosions is traditionally based upon simplified methods. Notably, has the so-called Biggs' method (Biggs, 1964) become popular. Biggs' approach is based upon a single degree of freedom (SDOF) idealisation of the complicated non-linear response to an explosion. This facilitates simple estimates to be made of the maximum displacement of a structural component.

Alternatively, the response may be calculated by means of nonlinear finite element analysis (NLFEA) of structural members and subsystems. This eliminates most of the simplifying assumptions that has to be made in the Biggs'- or other methods. However, it is generally not a trivial task to perform nonlinear analysis, e.g. the modelling, and execution of the analysis is often demanding with respect to both man-hours and skills.

The simplified methods warrant therefore, still their existence; notably for screening of the severity of the explosion scenarios, reserving NLFEA for the critical cases. Design requirements are often based on simplified methods e.g. the Interim Guidance Notes (1991) (IGN) and the NORSOK STANDARD N-004 for steel offshore structures subjected to accidental actions (NORSOK 1998).

In previous papers, Amdahl (2003) and Amdahl et. al. (2003) it has been shown that SDOF analysis may give good results for beams and stiffened plates. The purpose of the present work is to further investigate the adequacy of SDOF ANALYSIS for an explosion panel. Furthermore, the response of a deck structure subjected to very extreme explosion pressures, beyond the design accidental pressures, is investigated.

Blast wall plate/stiffener analysis

The blast wall between main deck and BOP deck is considered. The wall consists of 6 mm plating stiffened horizontally with L-profiles, L150 x 75 x 9, at 1200 mm intervals. The stiffeners are supported by vertical girders HE 300B, at various intervals. The most typical spacing is 4500 mm, which is considered in the following. First, the response of an individual stiffener with associated plate flange is studied.

Static and dynamic analyses are carried out with the explicit, nonlinear finite element code USFOS (Eberg et. al., 1993). The finite element mesh of stiffener and plate is shown in Figure 506.1. The size of each shell element is 60 x 62 mm; the width of the model corresponds to the stiffener spacing. Symmetry conditions are assumed at mid-span, so only half of the stiffener is modelled. At the end the stiffener and plate are welded to the vertical girders. It is assumed that the girder is strong enough to carry the loads transmitted to them from the stiffener and the plating, without being deformed. This is modelled by means of rigid beam elements at the end section of the model. The beam elements are strong enough to carry the loads transferred to them, so the model can be supported in one node only, allowing for easy modification of axial - and rotational boundary conditions. If the pressure is assumed to be uniformly distributed over several stiffener spans, it is natural to assume the stiffener ends rotationally fixed, because of symmetry.

At the plate long edges, rotational symmetry conditions are applied. It is not easy to specify the exact transverse boundary condition. The edge is neither free nor fixed against inward displacement. A straight edge condition might have been a good choice. However, the difference between free - and full fixity against transverse displacement is marginal, and most analyses

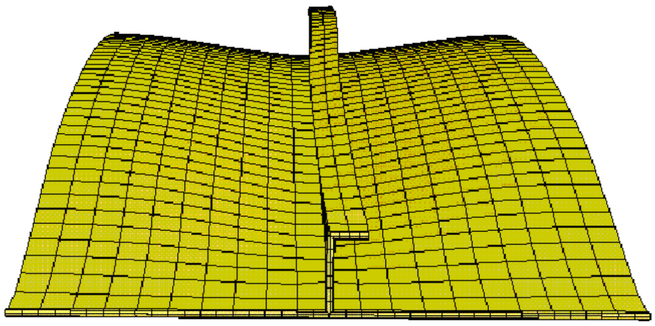

Figure 506.1 Finite element mesh of plate/stiffener.

The stiffeners are welded to the girders at the web and flange. Rat holes are modelled by omitting the two lower web shell elements at the support. This is not visible in Figure 506.1, but is observed in Figure 506.3

The L-profile is unsymmetrical. This may cause the stiffener tripping about weld toe. In order to avoid possible suppression of this deformation mode, the stiffener is assigned a sinusoidal lateral imperfection, as illustrated in Figure 506.1. The displacement amplitude is taken as the tolerance requirement, which is equal to 0.0015 x member length.

Figure 506.2 shows lateral pressure versus midspan displacement for the plate/stiffener as obtained in static analysis. The influence of the stiffener is particularly evidenced by the curve for free axial motion of stiffener ends, i.e. no membrane force is developed. The collapse pressure is then ~ 0.4 bar, corresponding to the formation of a three-hinge mechanism. Plastic theory yields a collapse pressure of 0.35 bar, being in good agreement with the analysis.

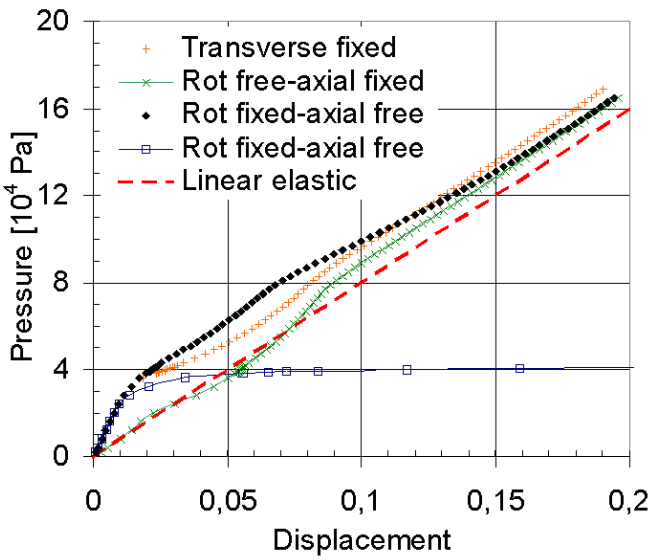

Figure 506.2 Pressure versus midspan displacement of plate.

If the ends can be considered rotationally - and axially fixed, the resistance increases considerably in the large deformation range due to the beneficial development of membrane forces. If the stiffener ends are assumed simply supported with respect to rotation, but axially fixed, it is observed that the initial stiffness is considerably reduced, but the resistance at large deformations is virtually identical to that for rotationally fixed ends. This is due to the fact that the resistance is entirely dominated by plate membrane stresses at large deformations. In all analyses the long edges are free to move in the transverse direction. Fixing also the long edges has a moderate influence on the resistance.

It is interesting to observe that a linear elastic resistance-deformation relationship for the plate (Refer NORSOK APPENDIX A 6.124) corresponds well with the simply supported solution.

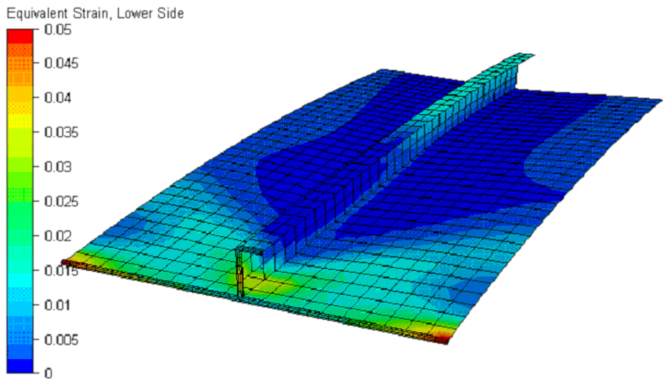

Figure 506.3 Equivalent strain for a pressure of 1.4 bar.

Figure 506.3 shows the equivalent strain at a pressure of 1.4 bar. The strain levels are small, except at the plate flange “tips”. The high strains here are not realistic; the rigid beam elements used to model the boundary conditions introduce an artificially large stiffness in this area.

In conclusion: The resistance of the plate–stiffener is not critical with respect to the explosion actions. The plating carries the maximum pressure with little yielding. The stiffeners have a moderate influence on the resistance for large deformations if the ends can be considered axially restrained. Whether this is always a valid assumptions may be subject to discussion, but as will be seen, the major resistance will be in the transverse direction of the stiffeners when also the girders are taken into account. Consequently, in order to simplify modelling, the stiffeners will be omitted in subsequent analyses.

Panel analysis

Figure 506.4 shows the finite element mesh of the explosion panel. The dimension of the model is approximately 7 x 7 m. The model includes 1.5 girder spacing and extends from main deck to BOP deck. (As a consequence of the arguments above, the stiffeners are omitted from the model). In order to simplify modelling the girder spacing is assumed 4500 mm. The element size in the plating is 150 x 150 mm and 100 x 150 mm in the web. The panel can be assumed to represent the west end of the south blast wall. The end of the panel is welded to the main steel columns. This is modelled by keeping the nodes fixed. The near end of the panel is midway between the vertical girders and symmetric boundary conditions are assumed.

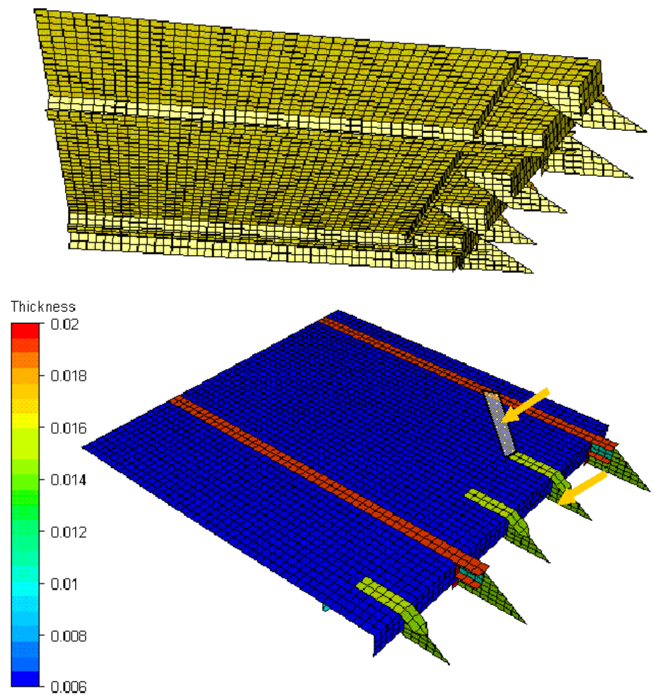

Figure 506.4 Finite element mesh and thickness of explosion panel.

At the main deck (to the right in Figure 506.4), the support of the panels are modelled. The geometry is not exactly represented, e.g. the web ends 25 mm behind the flange of the support, thus yielding a smother transition compared to the finite element model. The longitudinal stiffener adjacent to the support is modelled, as this may have some influence on the local strains. At the upper end the panel is fixed at the intersection with the BOP deck plating. Fixation of nodes is of marginal significance for the plate behaviour, but may be somewhat optimistic as concern the girder resistance.

The static resistance versus maximum displacement of girder and plate is shown in Figure 506.5. As expected the plate starts deforming at a low pressure, while the stiffener collapses at about 0.5 bar. The plating can, however, resist a significant pressure because of the membrane effect.

The resistance using plastic theory for the HEB 300B girder with/without accounting for the 6 mm plating is also plotted. The curves are based upon NORSOK Appendix A 6.9.2 clause. It appears that disregarding the plate contribution is overly conservative, when the plaiting is included a significantly better agreement is obtained, but still the simplified formulas are conservative. A pressure of 1.4 bar yields a girder displacement of 0.24 m for the girder and 0.29 m for the plate.

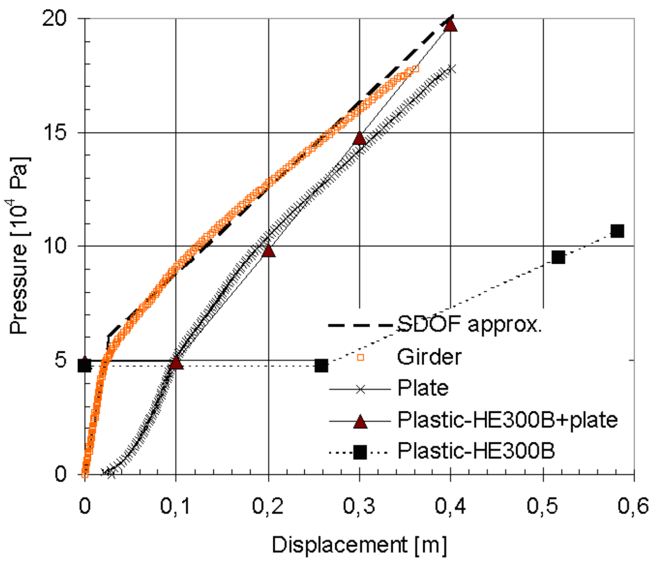

Figure 506.5 Resistance versus deformation –static analysis.

Figure 506.6 shows the degree of plastic utilisation. At 0.4 bar the plate is little utilised, but yielding is observed in the plate at the transition to the supports. There is reason to believe that the effect is exaggerated, because the transition is not accurately modelled. At a pressure of 1.4 bar, virtually the whole plate field yields. The pressure is primarily carried by membrane forces and the geometry effect caused by large deflections.

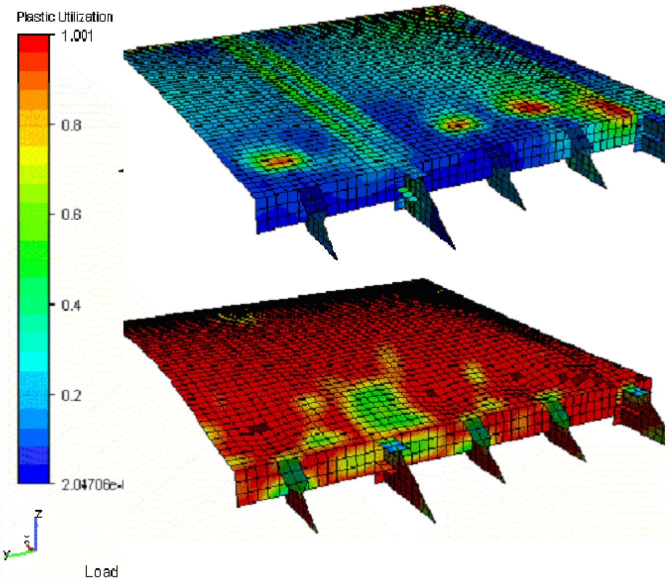

Figure 506.6 Deformations and plastic utilisation at a pressure of 0.4 bar and 1.4 - static analysis.

The response obtained in a dynamic analysis is shown in Figure 506.7. The pressure pulse used is triangular, with equal rise and decay time, and a total duration of 200 ms. The pressure amplitude is 1.4 bar. The maximum displacement is 0.32 m for the girder and 0.36 m for the plate, giving a dynamic magnification factor of 1.3 and 1.25, respectively.

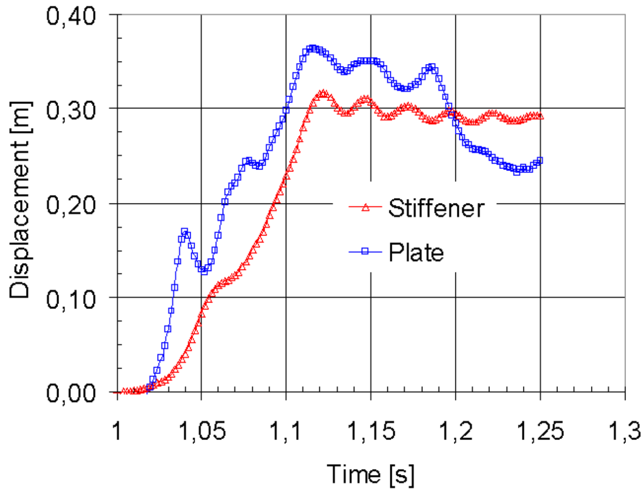

Figure 506.7 Displacement versus time – dynamic analysis.

Figure 506.8 shows the equivalent strain as a function of girder displacement for various critical locations. Locally the strains attain ~ 8 %, but this includes significant strains induced by plate bending; the membrane strain is in the range of 3%. The largest strain in the plating occurs in a diagonal at the right corner of the panel as indicated in Figure 506.4. This is due to the high restraint against deformation in this area. Elsewhere in the plate the strain is mostly below 1.25%, which is quoted as the tolerance limit for the passive fire protection used. Local failure in the form of cracks may be expected close to the supports and the lower corners where the strains are relatively large. Luckily, such cracks will occur at locations with where the heat fluxes are likely to be small in case of fires. Whether the PFP will sustain the violent speed / acceleration is an open question.

What is the acceptable strain level with respect to fracture? According to NORSOK Appendix A, clause A 3.10.3, the average *membrane* strain should not exceed a certain level, which depends on the mesh size used. For the present mesh with an element length of 150 mm and plate thickness 6 mm, the acceptable *membrane* strain is 4.6%. Hence, the strain level is acceptable according to the NORSOK criterion.

The strains in the girders may reach high values, 12%. This is, however, considered of minor importance; probably the strain levels will be reduced by more appropriate boundary conditions. Furthermore, the strains in the web may relax by web buckling.

Large strain concentrations occur in the webs of the supports, ~ 10%. The equivalent strain is dominated by membrane strains and is therefore twice the acceptable level according to the NORSOK criterion. On the other hand, should a fracture occur in the web of the support, this may not be be critical for the load-carrying; the plate stresses may be transferred into the main steel-work via the infill steel.

It should be emphasized that the strains depend heavily on the actual local modelling used. It is likely that a better modelling will reduce the strain levels somewhat at considered supports. Nevertheless, the risk of local fractures close to the supports can not be entirely disregarded.

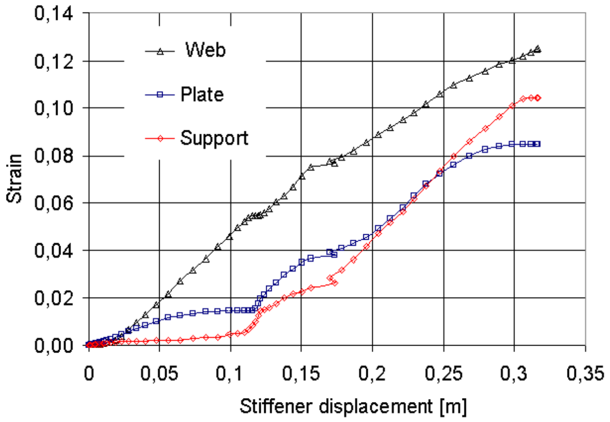

Figure 506.8 Strain versus girder displacement – dynamic analysis.

The above magnification factors are obtained for a given explosion duration (200 msecs.). It is of interest to assess how much the maximum displacement is influenced by the explosion duration. For this purpose a dynamic analysis with a single degree of freedom (SDOF) model is used. (The approach is identical to the one used for developing the response charts in NORSOK) The resistance function is taken as the bi-linear representation of the static resistance function for the girder, shown in Figure 506.5. Figure 506.9 displays the non-dimensional maximum displacement (y_{max}/y_{el}) versus the non-dimensional blast duration (t_d/T). y_{el} signifies the limiting elastic displacement; i.e. at the knee of the SDOF curve. Thus, the maximum *static* displacement for the girder, 0.24 m, comes out to be $y_{max}/y_{el} = 9.2$. The maximum *dynamic* displacement according to the SDOF analysis is 12.9 and is obtained for a load duration t_d/T ~5. Consequently the maximum dynamic magnification factor is 1.4. This is just slightly larger the result obtained in NLFEA with blast duration $t_d = 200$ ms, namely $y/y_{el} = 1.3$.

In order to compare the results from NLFEA and SDOF analysis, the eigenperiod for the plate/stiffener response needs to be estimated. A practical approach is to estimate the eigenperiod from the oscillations identified in the displacement plots in. Figure 506.7. If the stiffener response is used the eigenperiod is T ~25 ms, using the plate response there is obtained T ~ 36 ms. The normalised pulse durations are then either 8 or 5.5, the associated dynamic amplification factors are 1.08 and 1.3, respectively. The latter figure happens to be identical to the value obtained with NLFEA. It is likely, in the light of the large mass of the plating with PFP compared to the girder mass, that using the $t_d/T = 5.5$ is the more representative figure for estimating the dynamic response.

The analysis shows that there is a non-negligible dynamic magnification factor, even if the load duration is relatively long compared to the eigenperiod. The dynamic shell analysis with pulse duration of 200 msecs may therefore be considered virtually the worst case as concerns blast duration.

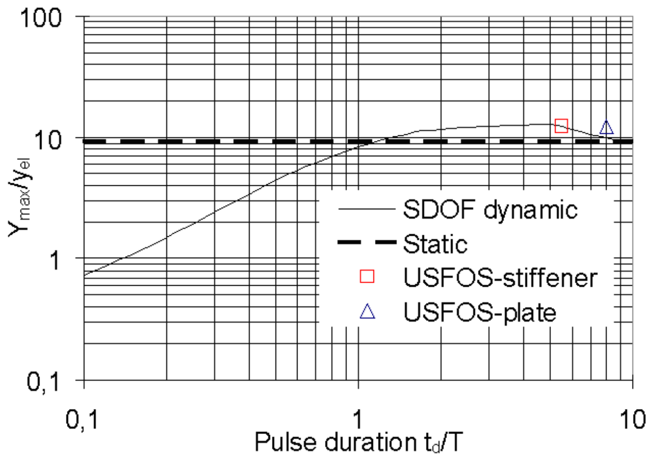

Figure 506.9 Dynamic response chart for panel based on SDOF dynamic analysis – NLFEA(USFOS) result based on either plate or beam period of vibration.

SDOF analysis has also been used to assess the significance of pulse duration and pressure spikes on the global response for a 3D frame model of a drilling/well-head module. The accuracy depends on the characteristics of the deformations. If the response is dominated by a single mode SDOF analysis is good. In any case, it is a versatile tool to identify which pressure histories, pulse shapes etc. that may be critical for the structure and should be subject to NLFEA.

Deck structure subjected to very extreme explosions

In Norway the design accidental pressures are determined in a risk analysis. On the basis of these pressures the platform is designed to resist the explosions. Damage is accepted, but generally the severity of the damage is such that not all the resistance of the platform is fully exhausted. In some projects, safety analysts address the question what will happen if the the structure is subjected to a very rare and extreme explosion, with a pressure that may exceed the design accidental pressure substantially. Will the platform survive, or collapse entirely?

In this context, main focus is set on the possibility to evacuate the platform after the explosion. The damages in the process area may be extremely large; fatalities may occur in the areas directly affected by the explosions, the structure may undergo severe deformations, but it should not collapse completely during the time required for evacuation of personnel from safe haven.

Hence, the scope of the global analysis goes substantially beyond that normally considered in the accidental limit state (ALS); e.g., the requirement to residual strength according to the ALS criterion is not assessed, viz. the resistance against 100 year environmental loads in damaged condition. The present approach may rather be denoted the limit state of safe evacuation or Evacuation Limit State, (ELS)

As a consequence of the extreme damages considered, the demand for accuracy must be somewhat relaxed compared to conventional analysis in the ALS. At present state of the art it is virtually impossible to perform rigorous simulation of the progressive collapse of each individual panel and subsystems, including initiation and propagation of cracks. Instead, the damages will be modelled by simple techniques as removal of panels, girders etc. This simplifies considerably an otherwise extremely complex and time-consuming task. However, it is believed that the approach does not really represent a limitation; it rather focuses on the major load-carrying behaviour of the global structure, omitting details of minor importance.

Explosion pressures

Various computer codes based on computational fluid dynamics principles are available for simulation of spatial and temporal distribution of explosion pressure. Often a large number of simulations are performed for the purpose of establishing probabilistic values of the maximum pressure. However, this is not sufficient to perform simulation of the nonlinear structural response. Unless the response is in the quasi-static range, it is also necessary to know the *duration and shape* of the pressure pulse as well as the *area* to which the explosions pressure applies. This information is sometimes missing.

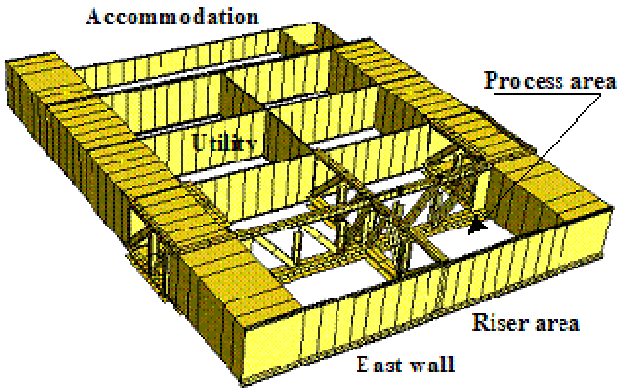

Figure 506.10 Deck structure of floating production platform.

Figure 506.10 shows a model of the main load-carrying system of the deck of a floating production platform. The columns (in the four corners) are not included in the model. The process area consists of a hybrid truss-work/plate girder system. The pressure on the east wall due to an internal explosion in the process area is plotted in Figure 506.11. Threshold values are selected such that red colour signifies pressures exceeding 5 bars. It is observed that for this extreme case a large part of the wall – from cellar deck to main deck and a width of ~ 10 m, is subjected to maximum pressures in excess of 5 bars. The total duration of the pressure pulse is fairly long; in the range of 100-200 msecs. In the analyses the duration is assumed to be 200 msecs, with equal rise and decay times.

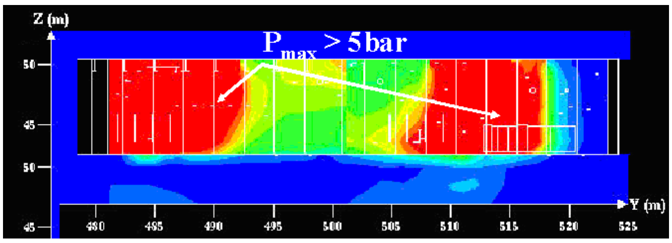

Figure 506.11 Distribution of maximum pressure on east wall.

Mechanical response

The mechanical responses analyses are carried out in two main steps:

- estimate structural damage for panels and subsystems for the different explosion “levels”, (pressure);
- check the global integrity for the different damage conditions.

This stepwise approach is selected because it gives a better overview of the governing parameters involved. It should be emphasized that studies of this kind have many uncertainties, which increases with increasing explosion pressure.

For moderate pressures (1-2 bar), most of the structure responds elastically, with moderate permanent deformations and small strain levers. This response can be predicted with relatively high precision.

For higher pressures, (~2-3 bar), larger parts of the structure responds *in*-elastically; permanent deformations in the order of 100-300mm are typically obtained. Strain levels in the order of a few % are predicted, and, depending on how the details are designed and fabricated, local failures may take place. These predictions are associated with more uncertainty.

For severe pressures, (3-5 bar), components are likely to fracture locally and could cause damages of the structure. Failures of some components/areas could lead to collapse of the areas close to the explosion.

For severe to extreme pressures, (5-10 bar), components exposed are likely to fracture and cause severe damages to the structure. Such predictions cannot be given with high precision. Failures of some components/subsystems could lead to progressive collapse of larger areas. As fracture initiation and propagation is a local phenomenon a very detailed mesh is required to simulate this with a high degree of precision. The computational task easily becomes overwhelming.

As an alternative to simulating the damage process in detail, one may start to remove important load-carrying components and check whether the deck structure will remain intact for the functional loads. An example is giving in the following.

If the explosion pressure is in the range of 3-5 bars, it is likely that the east wall will undergo severe damage with extensive fracture at cellar deck or main deck level. The contribution to the global resistance will be modest. Hence, a large part of the east wall is removed from the model as indicated in Figure 506.12. Upon applying functional loads, severe deformation takes place in the girder, the structure is on the brink of collapse, but as illustrated in Figure 506.13, it survives (Displacements magnified by a factor of 2). Even though the number of fatalities in the process area may be high, personnel in the utility area and accommodation unit, ~ 60 m away from the process area, may be able to evacuate the platform.

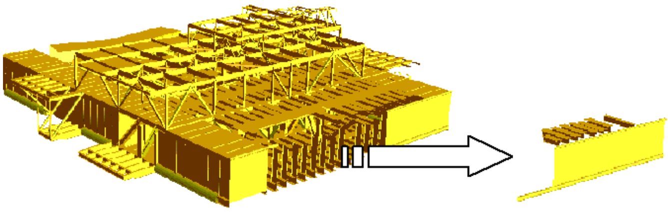

Figure 506.12 Damaged members removed.

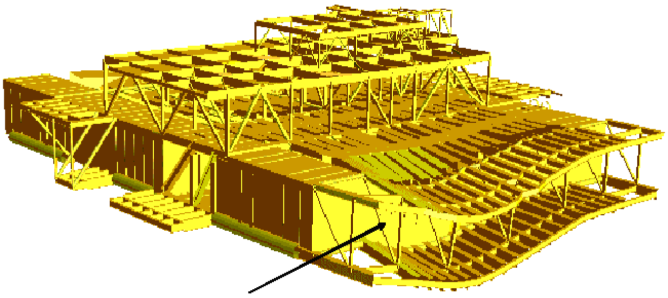

Figure 506.13 Deformation of damaged structure.

Conclusions

The large displacement analysis of the blast wall has shown that the plating contributes by far to the resistance while the stiffener contribution is almost negligible. From the point of view resisting extreme explosion pressures the stiffeners might as well be omitted. This observation tends to be quite general for stiffened panel utilized into the large displacement range.

SDOF analysis of the blast wall is useful. Previous observations that the response is precisely estimated, provided that the resistance function is accurately modeled, are confirmed. This allows easy sensitivity studies on the influence of pulse shape, duration etc.

SDOF analysis may also be used with success for 3D structures, provided that the response is dominated a single mode.

NLFEA of the response to pressures beyond the accidental limit state values may be useful in conjunction with explosion risk assessment. A major challenge is to predict initiation and growth of cracks due to excessive straining. In order to circumvent this problem, it is proposed to check the survivability of the structure with omitted members, identified as critically utilized in local -/ subsystems analysis.

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