SEISMIC RISK TO LIQUEFIED GAS STORAGE PLANT IN THE UNITED KINGDOM

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The effect of different levels of earthquake on examples of liquefied gas storage plant has been modelled. At the highest levels of earthquake expected in the United Kingdom some significant displacements and peak stresses are predicted. Mechanical failure causing release of contents is not seen as a likely consequence but safety margins of individual designs should be checked and may need to be increased if the very low probabilities of failure desirable with this type of plant are to be assured.

INTRODUCTION

The U.K. is situated in a region of low seismic risk. Historical evidence indicates that the pattern of the last few hundred years has been one of comparatively small tremors with the largest of these recurring at only infrequent intervals. There has been only limited damage to buildings and little, if any, loss of life or serious injury. Understandably, little or no attention is paid to seismic risk in general plant and building design, although there have been exceptions to this policy. The U.K. has no earthquake code and there are no statutory requirements for earthquake resistance in buildings and other structures.

Until halfway through this century this typically low loss level represented about the worst that could be expected as a consequence of an earthquake affecting the U.K. This situation is now changed. The last two decades have seen the widespread appearance of industrial operations which handle hazardous materials in such quantities that loss of containment is potentially disastrous. In these operations the consequences of an earthquake could be serious if any mechanical damage, however slight in itself, affected the integrity of the containment.

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The probability of seismic damage in the U.K. affecting the integrity of modern industrial plant is very low, but so are the risk levels demanded by present day society. The risk of a loss generated by earthquake cannot logically be ignored until it has been quantified, compared with acceptable limits of risk, and shown to be at a significantly lower level.

The potential for loss by earthquake induced release will depend on:-

(i) The nature and quantity of the material that could be released.

(ii) The vulnerability of the containment to earthquake damage.

Consideration of both factors together points to the storage of liquefied gases, either under pressure or at low temperature, as posing the greatest risk of catastrophic loss.

Many liquefied gas storage systems in the U.K. will have been installed without thought of seismic risk. Where this peril has been considered, analysis will generally have been based on static design methods which treat the structure as a rigid body and which exclude potentially important dynamic responses. Work is believed to be in hand to evaluate the dynamic responses of some critical containment structures in the U.K. but nothing has yet been published. There is thus no real guidance available on the nature and extent of this risk. The work described in this paper was undertaken as a first step in this direction.

The objectives of this work were simple and limited:-

(i) To determine if the probability of a release is so low that it can be disregarded

(ii) If this is not the case then to obtain some pointers as to the areas in which more detailed investigations may be required.

It was felt that these objectives could be achieved by modelling the response of typical containments to earthquakes of known probability of occurrence in the U.K. This calls for specialist skills in seismology and structural analysis and Soil Mechanics Ltd. were retained as consultants to carry out these aspects of the work. This paper is based upon their findings (1).

The work done fell into three distinct parts and will be described briefly in this order:-

(i) Consideration of the seismicity of the U.K. and selection of appropriate earthquake inputs.

(ii) Selection of representative containments and identification of major failure modes.

(iii) Modelling the responses of these containments to the selected earthquakes.

SEISMIC INPUT

Seismicity of the United Kingdom

Character of Earthquakes in the United Kingdom. The seismicity of the U.K. is best considered in the broad geological and tectonic setting of North West Europe. This is an intraplate region lying within the Eurasian plate well away from plate boundaries and is a relatively inactive area.

Intraplate earthquakes are difficult to explain. In N.W. Europe they generally cannot be related to geological faults visible at the surface. Nor is it possible to demonstrate satisfactorily an overall regional mechanism and stress field which can explain the accumulation of strain energy relieved by an earthquake. However, a number of mechanisms could contribute to the build up of strain energy which is then released in deep old crustal fractures that were last active many millions of years ago. Contributory mechanisms may include, for example, the relative vertical movement of tectonic blocks (block tectonics) and recovery from ice-age loading (isostatic rebound).

Despite the fact that causal mechanisms remain obscure it is possible to draw some conclusions about the probable nature of a damaging event. The characteristics of intraplate and N.W. European earthquakes have been considered by several workers in recent years and emerging from their work is the likelihood that an earthquake in Europe north of the Alps will be characterised by a form of faulting known as a thrust fault.

In addition, the severity of shaking at the surface is related to the depth of the fault. Deeper earthquakes in N.W. Europe, i.e. those with a focus deeper than 25km, produce a noticeably lower epicentral intensity than shallower events of similar magnitude. Thus the most damage is likely to result from a shallow event and this event in Europe north of the Alps is likely to have a strong thrust component.

Magnitude-Frequency relationship and Maximum Event. Data on earthquakes in N.W. Europe are scant compared with plate boundary zones and most are of doubtful accuracy. Very little have been obtained instrumentally and most values of magnitude have been derived empirically from descriptive accounts in contemporary sources. For the U.K. the best data currently available are provided by Lilwall (2), who used 170 years of data, disregarding much earlier material in an attempt to establish a reasonably homogeneous list for statistical purposes.

Lilwall's work suggests the existence of an upper limit to the magnitude so that the log frequency - body wave magnitude relationship, linear below a magnitude of about 5.0, rolls off to a limiting value of about 5.7 (Figure 1). Whilst the general shape of this curve is probably correct, the method used to derive the data must reduce the confidence in the accuracy of the values obtained for return periods and the potential maximum magnitude.

Local variations in seismicity. There are parts of N.W. Europe where seismic activity departs significantly from the regional mean. Within the British Isles for example, activity is significantly higher in the Monmouth area and in the region of Inverness, but lower in Ireland and North East and South East England. Elsewhere in N.W. Europe areas of higher activity include the lower Rhine Graben and the Norwegian Trench. However, the epicentres of the largest earthquakes of the last 100 years have been in areas with little low magnitude activity (e.g. Essex 1884, Oslo 1904, North Sea 1931, Brabant 1938 and Carlisle 1979). On such grounds it is unreasonable to discriminate between one area and another in terms of the low probability, large event.

Derivation of Earthquake Inputs

Selection of earthquake test levels. Our present state of knowledge indicates that damaging levels of ground motion in N.W. Europe should be treated as random phenomena in time and space. The selection of seismic design criteria should therefore be made on a probability basis. This implies that the selection is dependent on the chances that those responsible for the plant consider acceptable. Every time seismic design criteria are selected a decision is made, either explicitly or implicitly, of the acceptable level of risk.

In earthquake engineering it is common practice to consider two separate criteria. The first is associated with commercial risk - operations must not be seriously restricted by an earthquake that has a reasonable chance of occurring during the working life of the plant. The second is safety related - the plant must withstand the maximum credible earthquake that could occur without causing a disaster in human terms, although the plant itself may suffer major damage. It was felt that both these design criteria should be represented in the present study.

For the present work a level of 25% probability of occurrence in a working life of 40 years was selected as appropriate for the commercial criteria. This criteria will be called Level 1 and its return period may be calculated from the following equation which assumes that the temporal distribution of earthquakes is Poissonian:-

$$P_D = 1 - \mathrm{EXP}\left(-\frac{D}{T}\right) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (1)$$

Equation 1 gives a return period of 150 years for the selected Level 1 design earthquake, equivalent to a probability of occurrence of 7×10^{-3} events per year. Reference 2 indicates a site Modified Mercalli (MM) intensity of around V1 for this return period, which typically would be produced by an earthquake of body wave magnitude of about 4.5 occurring 15-20km from the site.

The safety related design criteria - Level 2 - is usually taken to be the maximum credible level of shaking which could occur at the site. The possibility of conflicting views on what constitutes the maximum credible level in N.W. Europe makes this a critical decision. The level should be as high as possible compatible with universal acceptance of credibility. In the event it was decided to base the Level 2 criterion on a body wave magnitude 5.25 and a site intensity of MM V11. (2) gives the probability for this level of event in the U.K. as around 5×10^{-4} events per year, equivalent to a mean return period of 2000 years and a risk level of 2% in a 40 year plant life.

A third and higher event (Level 3) was also considered as there are indications of a trend towards lower probability criteria for petrochemical hazards. For instance, a recent NFPA guide (3) recommends a design earthquake with a probability of 10^{-4} events per year for LNG storage. If this recommendation is followed in the U.K. it might well seem natural to adopt the design event at this level which has been used in British nuclear studies. This event was therefore selected for Level 3.

Selection of representative earthquakes. Having selected three levels of event there remained the task of obtaining accelerograms or horizintal motion time histories characterising earthquakes at these levels. Two approaches are possible, either an artificial time history can be synthesised or accelerograms from recorded earthquakes can be used. The latter approach was adopted in this work. In the absence of strong motion data from N.W. Europe, accelerograms to represent Levels 1 and 2 were sought in regions of similar general tectonic features. The following were selected:-

Level 1. New Madrid (Missouri) earthquake of June 13, 1975, body wave magnitude 4.3, 10km epicentral distance.

Level 2. Forgaria-Cornino (Italy) event of May 11, 1976, body wave magnitude 5.2. An aftershock of the Friuli earthquake of May 6, 1976, 5km epicentral distance.

The Level 3 event was selected as one of the Temblor records of the Parkfield (California) earthquake of June 27, 1966. This earthquake had a body wave magnitude of 5.6 and the record was obtained 24km from the epicentre but very close to the fault rupture. This earthquake was not chosen on seismotectonic grounds but because it formed the basis of recent British nuclear plant studies.

Time histories for the three earthquakes are shown in Figures 2 - 4.

LIQUEFIED GAS CONTAINMENTS

Selection of Containments for Analysis

Liquefied gases are stored in one of two ways:-

(i) At low temperature and essentially atmospheric pressure.

(ii) Under pressure at ambient temperature or a reduced level of refrigeration.

Low temperature storage vessels are usually vertical cylindrical insulated tanks. Single skin tanks are normally used for temperatures down to -50°C (e.g. for LPG, ammonia, etc.) and double skin tanks for lower temperatures (e.g. for LNG, ethylene, etc.). Pressure vessels used for storage are usually horizontal cylinders for quantities up to a few hundred cubic metres and spheres for quantities larger than this. Similar types of pressure vessels are used for refrigerated storage under pressure.

Three designs were selected as typical to represent these types of containment (Figure 5):-

(a) A 20,000 tonne (50,000 m^3) capacity double skin cryogenic LNG tank with a suspended roof and constructed with a 9% nickel steel inner shell and mild steel outer shell.

(b) A 1250 tonne (2500 m^3) capacity propane sphere on nine support columns.

(c) A 250 tonne (270 m^3) capacity horizontal cylinder (torpedo) liquid carbon dioxide tank.

All three vessels were considered on a hard rock foundation and in addition the cryogenic tank was also considered on a piled foundation in soft soil.

Identification of Failure Modes

For each type of containment there is the possibility of ground or foundation failure leading to rupture of connecting pipes or other damage. The most common cause of ground failure in earthquake is liquefaction of soft soils with a high water table. This is a phenomenon which cannot be entirely dismissed in the U.K. but one which is very unlikely. Special tanks, such as low temperature containments, would almost certainly be piled in soft soils so that this hazard would not arise. It is therefore felt that ground or foundation failure is an unlikely event with liquefied gas containments in the U.K.

There is also the possibility of brittle fracture failure which is likely to be of greatest importance in refrigerated storage. This is not covered in this analysis but the possibility of this type of failure is discussed later.

With the exception of the above, the major failure modes which could lead to a gross release of contents are listed in Table 1.

TABLE 1 - Postulated Major Failure Modes.

Cryogenic Tank

a. Rupture of connection between tank wall and floor

b. Ripping of tie-down straps

c. Local shell buckling

d. Excessive displacement of base of shell, rupture of connecting pipes.

Sphere

a. Tear at connection of supporting leg and shell

b. Excessive displacement at connecting pipe take-off, rupture of pipe

c. Buckling of support columns.

Torpedo

a. Tear at connection of support and shell

b. Rupture of straps or supports

c. Slide on or roll off supports, rupture of connecting pipes.

For each of these failure modes the most critical points for the measurement of stress and displacement were selected and these points are indicated on Figure 5.

MODELLING OF EARTHQUAKE RESPONSE

Method of Analysis

There are three basic approaches to the analysis of soil-structure systems. These are the spectrum method, which is cheapest in computer time, the Fourier method, which is slightly more costly but offers improved modelling of soil-structure damping and time dependent characteristics, and the time step method, which offers much more exact modelling but is extremely costly in computer time.

The Fourier method was selected as the most appropriate to use. Its advantages over the spectrum method were considered significant for this work, and it was preferred to the time step method because the level of modelling did not justify the extensive computer time required by the latter.

Modelling of Containments

The containments were numerically modelled by a three dimensional lattice of simple two noded beam elements with six degrees of freedom at each node. As an example, the model for the sphere is shown in Figure 6. The mass of each element was incorporated in a manner consistent with the bending of the element and viscous damping was incorporated as a percentage of critical. These elements together modelled the stiffness, damping and mass characteristics of the complete structure. In addition, complex stiffness (incorporating stiffness, damping and mass) could be added at each of the nodes. In this way due account could be taken of the stiffness of the rock or soil in the foundation and the movement of the contained fluid (sloshing) in the cryogenic tank. For the other two tanks it was assumed that the tanks were full of fluid and appropriate masses were added at the nodes. As the ground shaking is likely to be only low to moderate in intensity, the action of the soil was assumed to be substantially linear. For the case of the piled tank, 20m of soil was assumed to overlie the rock on which the piles were founded.

This method of modelling generally distributes the stiffness, damping and mass of the structure in a realistic manner. As a result, the modes of vibration of the structure as a whole are modelled fairly accurately. However, no attempt has been made to accurately model or predict the response of small structural details (e.g. stresses around a pipe entry).

For each model the points of critical stress and displacement most appropriate to the failure modes identified in Table 1 were selected. These points and the corresponding failure modes are indicated in Figure 5. The models were then analysed to obtain responses in terms of stress and displacement at these points.

In the Fourier method the earthquake acceleration-time record is converted to an equivalent Fourier spectrum (Figure 7) using a standard computer program. The analysis then proceeds in two stages. First each structure is analysed by computer at a seried of frequencies identical to those covered in the Fourier spectrum. In each case the structure is shaken at its foundations by a force equivalent to an earthquake of the form of a steady state sine wave at a given frequency with an arbitrary value of 1g. acceleration amplitude. From the output of this program the stresses and displacements at the selected structural points are obtained as a function of frequency at this normalised input of 1g. Typical response spectra are shown in Figures 8-13.

The second stage of the analysis is to combine this frequency response for a specific structural detail with the Fourier spectrum for the desired earthquake using a further computer program. The output of this program gives the

response of the structural detail to the earthquake as a function of time. Figures 14-19 show examples of responses obtained to the three earthquakes for the frequency responses illustrated in Figures 8-13.

RESULTS

Response curves such as those of Figures 8-19 give a valuable insight into the behaviour of the structures but the main features can be summarised in terms of the maximum value of response obtained. These are listed in Tables 2-4. In all cases the values of the dynamic stress quoted are additional to any static response of the structure, wind loading, etc. Design codes tend to allow for up to 25% of allowable static stress for transitory effects such as seismic loading.

Responses to Level 1 Earthquake

The containment responses to the Level 1 earthquake are all small. It is unlikely that this earthquake would cause any distress to any of the tanks examined.

TABLE 2 - Maximum responses to Level 1 Earthquake

			Maximum Stress			
Containment	Foundation	Maximum Horizontal Displacement x 10³	Location	Direction	Stress x 10^{-6}	% of Allowable*
Cryogenic	Rock	0.7	Inner Shell Base	Vertical	2.6	1
			Tie-down Straps	Vertical	4.2	3
Cryogenic	Piles	0.7	Inner Shell Base	Vertical	1.8	0.7
Sphere	Rock	1.3	Shell	Vertical	0.02	0.01
			Leg	Vertical	0.9	0.04
Torpedo	Rock	0.7	Shell	Longitudinal	1.4	0.6
			Leg Base	Horizontal	0.4	0.2

* In Tables 2-4 the allowable stress is assumed to be 160 x 10^6 for the steel of the tie-down straps and 250 x 10^6 for all other steels.

TABLE 3 - Maximum responses to Level 2 Earthquake

Containment	Foundation	Maximum Horizontal Displacement x 10^3	Location	Maximum Stress Direction	Stress x 10^{-6}	% of Allowable*
Cryogenic	Rock	220	Inner Shell Base	Vertical	120	48
			Tie-down Straps	Vertical	230	140
Cryogenic	Piles	210	Inner Shell Base	Vertical	53	21
Sphere	Rock	220	Shell	Vertical	0.6	0.2
			Leg	Vertical	30	12
Torpedo	Rock	210	Shell	Longitudinal	7	3
			Leg Base	Horizontal	9	4

Responses to Level 2 Earthquake

Some of the responses to the Level 2 earthquake are significantly high.

Displacements. At first sight these appear to be the most serious of the responses with uniformly high values of about 0.2m for each containment. The possibility of displacements of this magnitude occurring in practice could be a matter of concern, depending on the nature of pipe connections and other tank details. However, the displacements are measured relative to a fixed point in space and are largely due to the low frequency displacement of the ground under the tanks. The displacements of the tanks themselves are generally similar to those of the ground below and immediately surrounding them with small (possibly up to 0.02m) differences due to dynamic effects. Consequently, similar displacements will occur in areas of plant adjacent to the tank, although there will be a phase difference between the displacements of the tank and other areas of plant. Within 150m of the tank for the rock site and 30m for the soil site the low frequency ground displacements will be sufficiently in phase to restrict the relative ground displacement to not more than ten per cent of the absolute displacement. In this case, relative displacements between, for example, the two ends of pipe connections between the tank and other plant, would be expected to be much smaller than those listed in Tables 2 - 4. Any detailed consideration of the effects on a containment of excessive horizontal displacement must therefore be made with reference to other plant connected to it.

Stresses. Significantly high stresses were obtained with the cryogenic tank characterising failure modes a, b and c in Table 1. However, these high stresses did not persist for a long time, lasting about 5 seconds at 70% of maximum value for the rock site but only about one second on the piled foundation.

The sloshing frequency for the cryogenic tank is 0.15 H_z, (Figures 8, 9). The analyses did not show a dominant response at this frequency (Figures 14, 15) which indicates that significant sloshing would not occur. This reflects the low energy content at that frequency of the Fourier spectrum (Figure 7). If significant sloshing did occur, much greater stresses would be produced in the inner shell.

In the case of the sphere the maximum shell stress is very low and should not cause any problems. The stresses in the support columns are much higher being up to 12% of the allowable stress. This could be of concern as although it is only half of the increase in allowable stresses for transitory loads, additional buckling effects and differences in the design of individual tanks could become critical for the slender columns.

The maximum stresses for the torpedo tank are all below 4% of the allowable stress and should not cause any problems.

TABLE 4 - Maximum responses to Level 3 Earthquake

Containment	Foundation	Maximum Horizontal Displacement x 10^3	Location	Maximum Stress Direction	Stress x 10^{-6}	% of Allowable*
Cryogenic	Rock	71	Inner Shell Base	Vertical	180	72
			Tie-down Straps	Vertical	310	190
Cryogenic	Piles	68	Inner Shell Base	Vertical	76	29
Sphere	Rock	74	Shell	Vertical	0.6	0.2
			Leg	Vertical	29	12
Torpedo	Rock	68	Shell	Longitudinal	6	2
			Leg Base	Horizontal	3	1

Responses to Level 3 Earthquake

Displacements. As with the Level 2 event the maximum displacement of each containment is almost identical, but although it is a larger earthquake these displacements are much smaller, only 0.07m. compared with 0.2m. at Level 2. This is a result of the differing frequency characteristics of the two events, those of the Parkfield earthquake at the Temblor monitoring site being relatively stronger in the middle range and less strong at the lowest frequencies from which the horizontal displacements derive.

Stresses. The maximum stresses produced by the Level 3 earthquake also follow an irregular pattern compared with the Level 2 event. They are higher for the cryogenic tank, similar for the sphere and lower for the torpedo. This relationship reflects the relative strengths of the seismic spectrum at the dominant containment frequencies.

Effect of Frequency

The dominant frequency in the cryogenic tank response is at 2.6 Hz corresponding to a rocking mode of oscillation (Figure 8). Both the Level 2 and Level 3 earthquake spectra peak at about this point and the larger Level 3 event has the larger component. The dominant frequency in the response of the sphere is at about 1.5 Hz. (Figure 10). The Level 3 spectrum has a local low at this point and its strength is comparable to Level 2. The dominant frequencies in the stress responses of the torpedo are in the very low and very high ranges and the Level 2 event produces the larger responses as it is the more strongly represented at the extreme ends of the spectrum.

Influence of piled foundations

The response of the inner shell of the cryogenic tank to all three levels of earthquake was always lower on the piled foundation compared with the rock foundation. A similar relationship can be expected for the outer shell and tie-down straps, and also for the response of the sphere on piled and rock foundations. Further work was therefore limited to the rock foundation only, as this was seen to be associated with larger responses.

DISCUSSION

Scope and Limitation of Work

This investigation was intended to be generic in nature and each of the storage installations analysed represents a particular class of containment. In practice, differences in size and design within a class may produce noticeable differences in responses but the general order of magnitude should remain the same. Thus the responses recorded in Tables 2 - 4 should be viewed in terms of their order of magnitude.

As in all modelling exercises a number of assumptions and simplifications have been made. Some of these are conservative and some are non-conservative. A major non-conservative factor is that the stresses evaluated are average stresses. Local peak stresses, particularly at non-uniformities, may be much higher. The major conservative factor is probably the seismic data base used which is thought by most authorities to overestimate historic seismic intensities. This latter factor is though to predominate so that the overall bias is towards conservatism, but in practice this advantage may be nullified if acceptable probabilities are set at lower levels.

Interpretation of Results

The Level 1 or commercially related earthquake produced no significant responses in the containments modelled, but both the safety related Level 2 event and the larger Level 3 earthquake produced significant displacements and stresses.

Displacements. Problems of potential failure caused by excessive horizontal displacement should not be too difficult to resolve. Except over large separations the relative movement between units will generally be much smaller than the absolute values for displacement obtained in this work. Many installations will already incorporate adequate flexibility in their pipework for other reasons. Exceptions should not be difficult to identify either in existing plant or on the drawing board, and provision of extra flexibility, where necessary, should not prove onerous.

Stresses. The question of the high predicted stresses is a more difficult one. Significantly high vertical stresses were obtained in the inner shell of the cryogenic tank and the tie-down straps for the outer shell. At a somewhat lower level, significant vertical stresses were predicted in the support columns of the sphere.

This shows that within these two classes of storage, earthquakes approaching the largest expected in the U.K. may induce stresses in individual designs which exceed the maximum allowable design stresses, possibly by a considerable amount. This is not to say that structures will necessarily fail catastrophically. High stress levels will exist for short periods only and even where permanent deformation occurs containment may not be lost. Design codes allow considerable margins of safety but these are intended to absorb many other adverse factors and may already be partially eroded. The question then is not, "will a hypothetically perfect structure fail at this level of earthquake?", but rather, "will the further erosion of possibly already eroded safety margins leave the probability of failure unquestionably acceptable?"

The probability of loss of containment following an earthquake, for a single failure mode of a single tank, may be defined as:-

$$P_L = P_E . P_F \quad . . . . \quad (2)$$

The probability of resulting loss, either of life or property, will in general be less than P_L as to obtain this the value of P_L must in turn be multiplied by other probabilities; e.g. for a fire, it must be multiplied by the probability of ignition, or for a toxic release, by the probability of an unfavourable wind direction, etc.

It is worth noting that P_F is related to P_E by an equation of the form:-

$$P_F = f\left(\frac{1}{P_E}\right) \quad . \; . \; . \; . \; . \; . \; . \; . \; . \quad (3)$$

The maximum value of P_L will not generally correspond to either the maximum value of P_E (where P_F approaches zero) or P_F (where P_E approaches zero) but will occur at some intermediate value of both.

As a simple example, consider the hypothetical case of an installation where a large release is virtually certain to cause a serious loss, i.e. where the product of the other probabilities referred to above is unity. It has been suggested that the minimum acceptable risk to the public from a major hazard can be set at 10^{-7} events per year (4). Substituting this value for P_L in equation 2, and 5×10^{-4} (the Level 2 event probability) for P_E yields:-

$$10^{-7} = 5 \times 10^{-4} \cdot P_F$$

$$\text{or} \quad P_F = 2 \times 10^{-4}$$

i.e. for the risk to be acceptable the probability of the Level 2 earthquake causing failure must be less than 2×10^{-4}.

Equation (2) refers to a single failure mode for a single tank. If there is a plurality of tanks or failure modes, then the probability of at least one failure will be increased. This can only be decreased to an acceptable level by reducing individual values of P_F, in the case of the example cited this means to substantially less than 2×10^{-4}.

The problem of the high stresses predicted in Tables 3 and 4 can now be placed in perspective. For a design to be acceptable, safety margins must be large enough to ensure that the probability of failure caused by these stresses is less than a value which, in practice, may be in the region of 10^{-5} or less.

Precise assessment of structural behaviour at such low probability levels is rarely possible and if there are any doubts as to the ability of the structure to meet these requirements then safety margins should be increased, i.e. the structure strengthened, until confidence is completely restored. In order to achieve this level of confidence it may be necessary to examine the response of individual

tanks to earthquake in greater detail than was done in this study.

Seismic Design Criteria

A consistent approach to seismic risk within the process industries calls for the establishment of seismic design criteria and these should reflect the nature of the plant to which they are to apply. The response curves generated in this work show that containment structures are sensitive to some parts only of the frequency spectrum. Thus the response of a structure will be affected by the peak levels of acceleration at its most critical frequencies rather than by the overall nominal value of earthquake magnitude or intensity. This can be seen in the relative responses obtained to the Level 2 and Level 3 earthquakes. Seismic criteria must therefore be selected with care to match the type of plant involved.

Brittle Fracture Failure

This was not considered as a possible mode of failure in the main part of this investigation. A combination of three factors is necessary before brittle fracture can occur (5):-

(i) The presence of a defect or severe stress concentration.

(ii) Tensile stresses.

(iii) Material of low fracture toughness.

It is believed that the three tanks examined would not present a high risk of brittle fracture failure because of the particular materials and design conditions adopted. A very different situation could arise, however, with other types of storage tanks and designs, partocularly with steel tanks to BS4741, or AP/620 Appendix R, that are used for refrigerated storage down to -50°C. There is thus a need for further work to determine the extent of this potential risk.

CONCLUSIONS

1. A level of earthquake which occurs sufficiently frequently to be likely to be felt during the normal working life of a plant poses a negligible risk of damage or loss of containment to liquefied gas containments in the U.K.

2. A larger and less frequent earthquake, approaching the maximum magnitude credible in the U.K. could produce:-

(i) appreciable horizontal displacements in all types of plant

(ii) high stresses in certain critical parts of some containment structures which exceed the dynamic stress levels allowed in current design codes.

Subsequent loss of containment is deemed unlikely but installations and designs should be examined to confirm that existing safety margins are adequate to ensure an acceptably low probability of failure. Where this cannot be confirmed with confidence the nature and quantity of the contained material may make it desirable to strengthen the containment to increase existing margins of safety.

SYMBOLS USED

P_D = probability of occurrence or exceedance of earthquake level during working life of plant

D = plant working life (years)

T = return period of earthquake (years)

P_L = probability per year of loss of containment as a consequence of earthquake

P_E = probability per year of occurrence or exceedance of earthquake level

P_F = probability of a given mechanical failure at a given level of earthquake

P = probability of earthquake magnitude not exceeding a given level during the given time interval (Figure 1)

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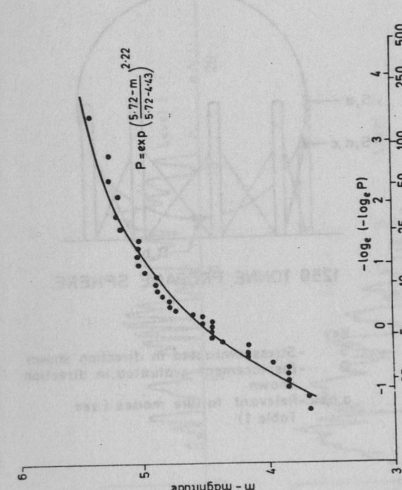

Figure 1 Seismicity of Britain (from Reference 2)

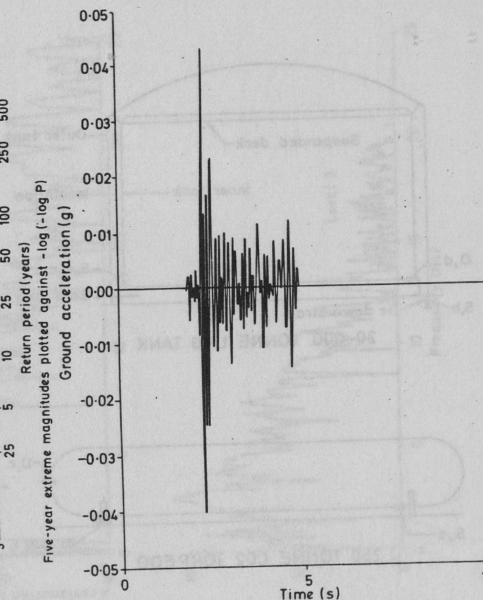

Figure 2 Time history of Level 1 earthquake

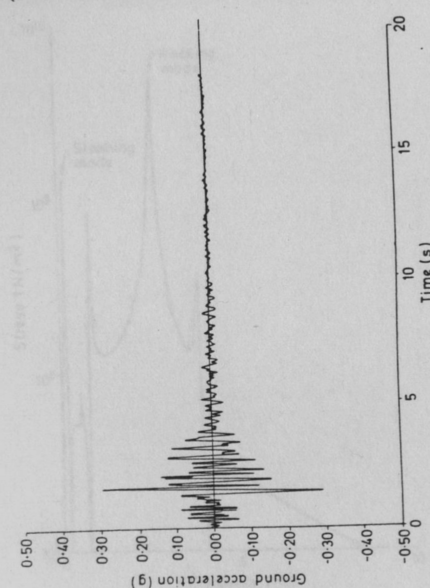

Figure 3 Time history of Level 2 earthquake

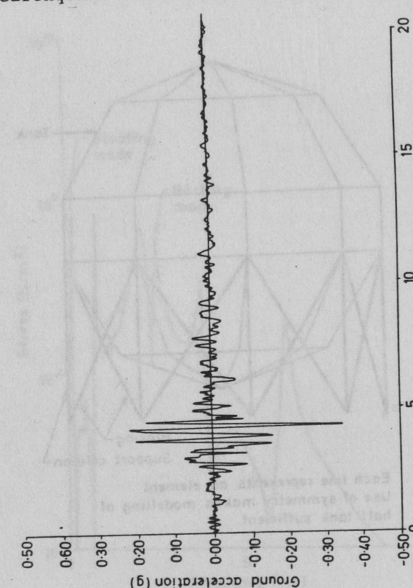

Figure 4 Time history of Level 3 earthquake

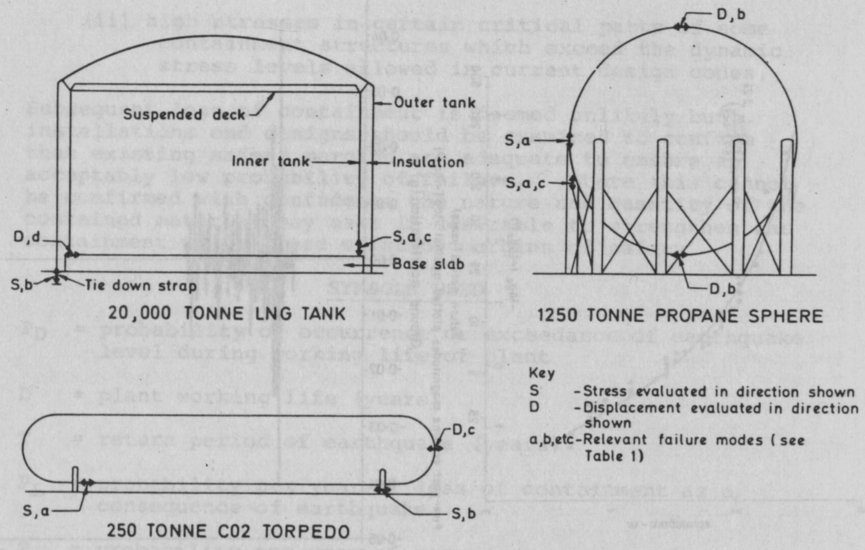

Figure 5 Arrangement of the tanks modelled showing points of evaluation

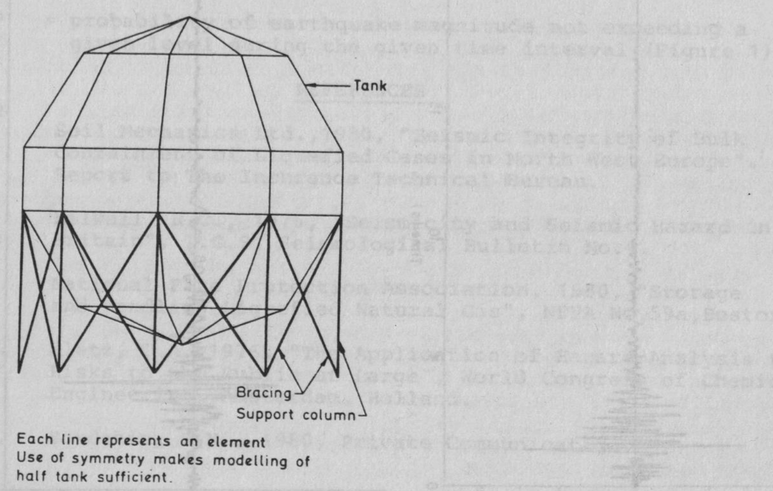

Figure 6 Pictorial representation of spherical tank model

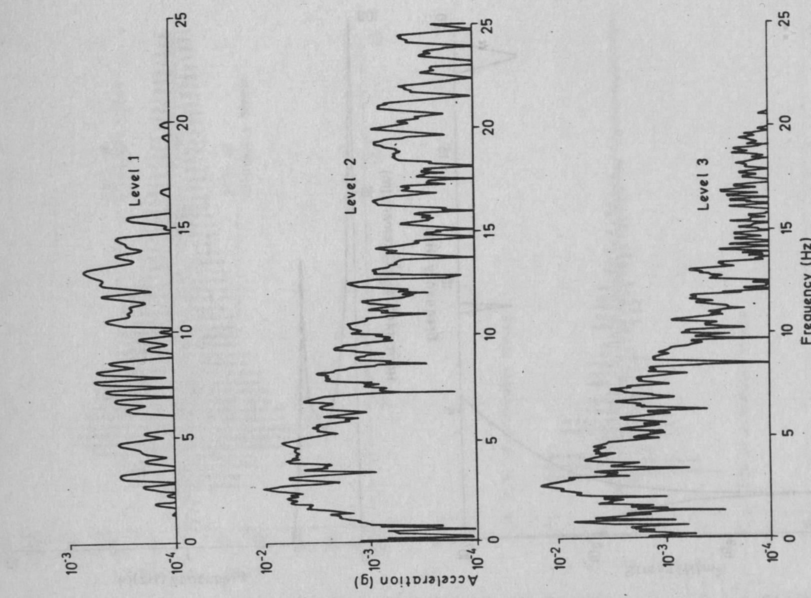

Figure 7 Fourrier spectra of the three earthquakes

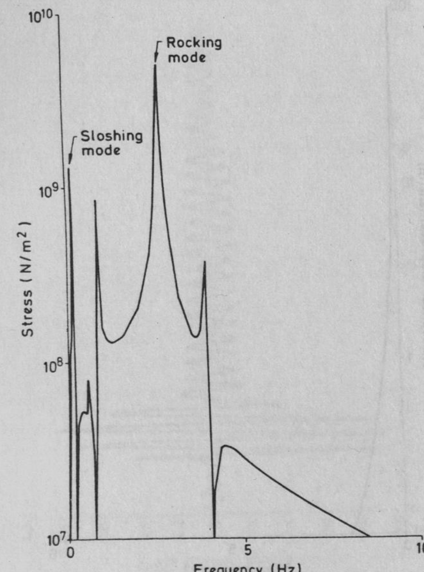

Figure 8 Response spectrum, cryogenic tank on rock, inner shell stress

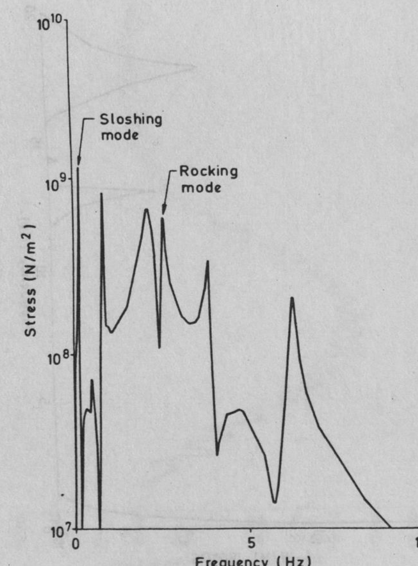

Figure 9 Response spectrum, cryogenic tank on piles, inner shell stress

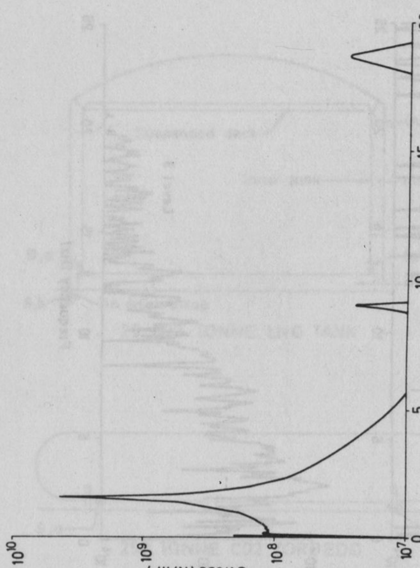

Figure 10 Response spectrum, sphere on rock, vertical stress in leg

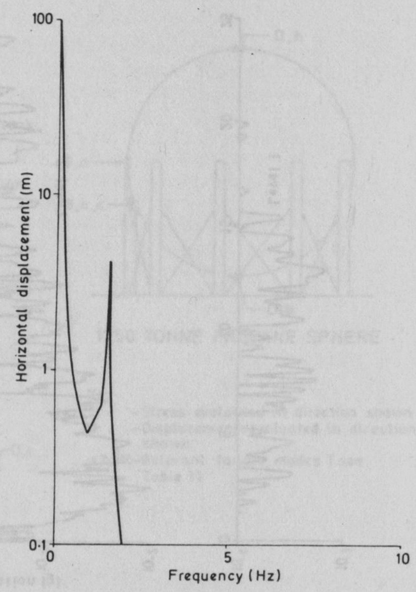

Figure 11 Response spectrum, sphere on rock, horizontal displacement

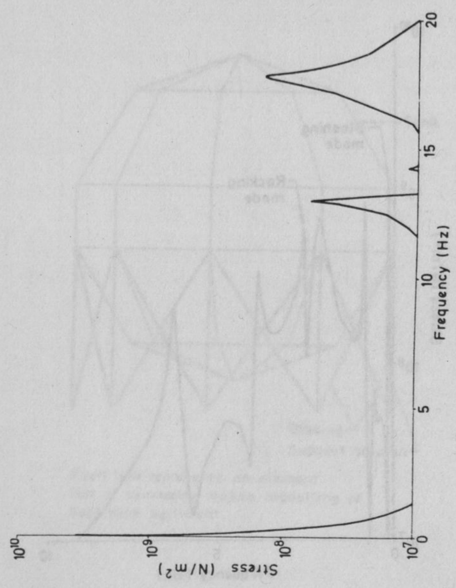

Figure 12 Response spectrum, torpedo on rock, horizontal shear stress in leg

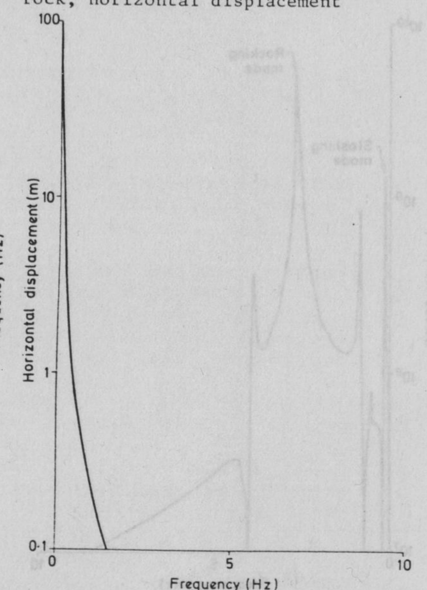

Figure 13 Response spectrum, torpedo on rock, horizontal displacement

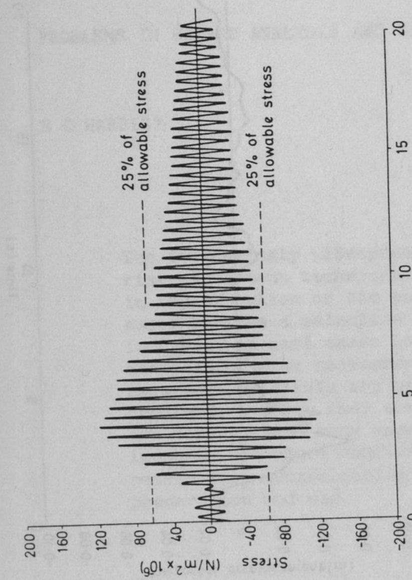

Figure 14 Response to Level 2, inner shell stress, cryogenic tank on rock

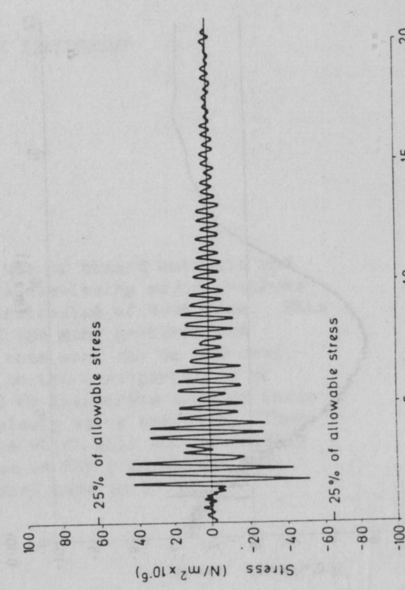

Figure 15 Response to Level 2, inner shell stress, cryogenic tank on piles

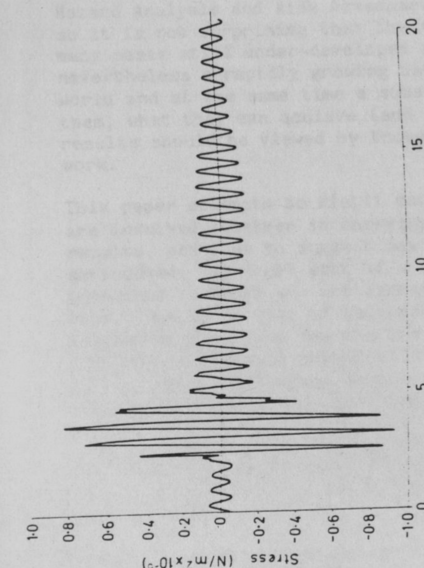

Figure 16 Response to Level 1, vertical stress in leg, sphere on rock

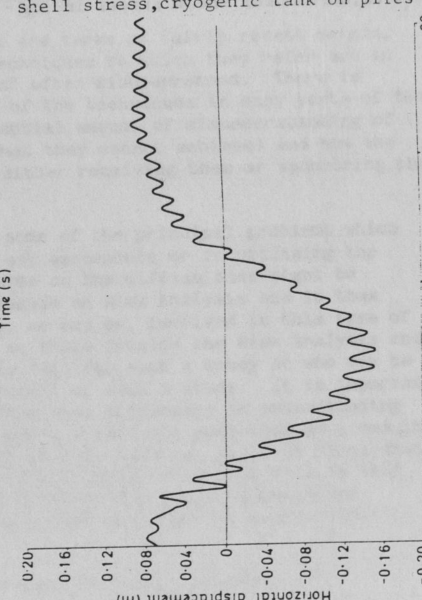

Figure 17 Response to Level 2, horizontal displacement, sphere on rock

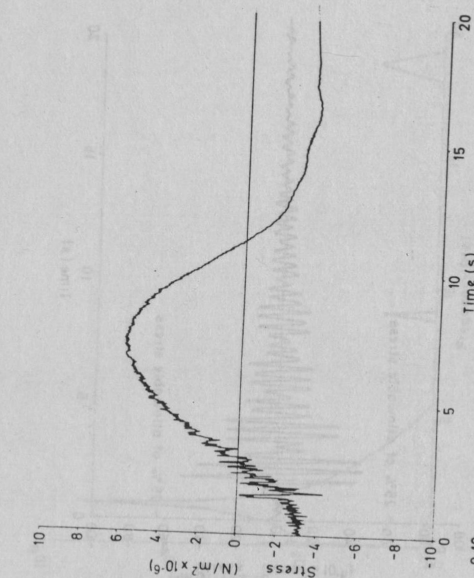

Figure 18 Response to Level 2,torpedo on rock,horizontal shear stress in leg

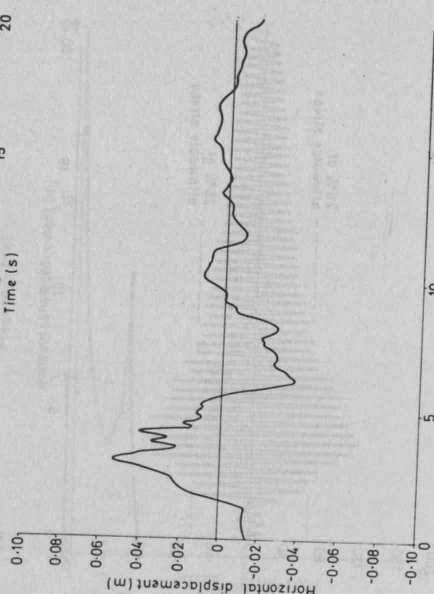

Figure 19 Response to Level 3,torpedo on rock,horizontal displacement

PROBLEMS IN HAZARD ANALYSIS AND RISK ASSESSMENT

N C HARRIS*

The increasingly widespread use of hazard analysis and risk assessment techniques is disclosing major problems in the execution or the understanding of such work. This paper reviews a selection of the many problems and indicates in some cases how this work may be improved. Examples of risk assessment in the transportation of hazardous materials are used to illustrate some of these problems, and how they are slowly being improved. There are nevertheless many aspects which will not be readily improved and where confidence in the predictions must remain low, necessitating extra care in their preparation and use.

INTRODUCTION

Hazard Analysis and Risk Assessment are terms of fairly recent origin, so it is not surprising that the techniques to which they refer are in many cases still under-developed and often misunderstood. There is nevertheless a rapidly growing use of the techniques in many parts of the world and at the same time a substantial amount of misunderstanding of them, what they can achieve (and what they cannot achieve) and how the results should be viewed by those either receiving them or sponsoring the work.

This paper attempts to distil out some of the principal problems which are involved, whether in carrying out asessments or in utilising the results, and also to suggest how some on the difficulties might be surmounted. It forms part of a session on Risk Analysis and is thus presented to those who are already, or may be, involved in this type of work. But it is also of interest to those outside the Risk Analysis and Assessment field who may require to call for such a study or who may be involved in decision making as a result of such a study. It is important not to forget that these people often have difficulty in understanding Hazard Analysis, or may take apparently irrational decisions as a result of lack of understanding of the report they receive, so it is vital that those actually conducting the work and writing it up bear this in mind and discharge their responsibility to present a fully reasoned and calculated assessment, which can be a true and positive contribution to the improvement of safety.

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