Sloshing analysis of LNG membrane tanks
FOREWORD

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SECTION 1 GENERAL

1 Introduction

Sloshing is the term used to describe the violent resonant motion of the free surface of a liquid cargo inside a moving tank, e.g. standing waves in partially filled cargo tanks on board an LNG carrier. The moving liquid will exert forces on bodies inside the tank and on the tank walls. These forces are normally referred to as sloshing loads, and need to be considered in the design of the cargo containment system, pump tower and pump tower supports, as well as the inner hull structure that supports the cargo containment system.

Sloshing can induce various types of loads on the tank structure. Violent sloshing may cause breaking waves and high velocities of the fluid surface. In this case the fluid can cause impact loads on the containment system. The intensity of the impact loads vary considerably during stationary vessel motion conditions. The most severe impacts are characterized by a high pressure with short duration acting on a limited area, and occur only occasionally. Moderate standing wave type liquid motion cause dynamic pressure loads with magnitude similar to hydrostatic pressure based on the instantaneous wave elevation. The load period is approximately the same as the sloshing resonance period.

Severe and violent sloshing typically occurs when the vessel moves with motion periods close to the highest sloshing resonance period for the liquid in the tank. For a tank with an approximate rectangular cross-section, the highest resonance period for two-dimensional standing wave sloshing motion can be estimated by the following formula:

\[ T_{res} = 2 \sqrt{\frac{B}{g \cdot \tanh\left(\frac{\pi h}{B}\right)}} \]

Where:

- \( h \) = the filling height in the tank
- \( B \) = the length of the free surface
- \( g \) = the gravity constant.

The formula provides good estimates of the resonance period for longitudinal liquid motion and transverse motion for filling levels outside the chamfer areas of prismatic membrane tanks. Reasonable estimates can also be achieved for transverse motion and fillings inside the upper chamfered area if the free surface length is adjusted accordingly.

Figure 1 shows a plot of the estimated resonance period as function of the ratio between filling height and the free surface length for a rectangular tank with breadth 38 m. This is a typical breadth between longitudinal bulkheads of a membrane LNG tank. The resonance period for the tank is about 7 seconds for high filling levels, and it increases for decreasing filling levels. Motions of large vessels like typical LNG carriers will contain motion components in the resonance period range for a wide range of sea conditions. This is one of the reasons why sloshing is a key design factor for membrane LNG tanks.
Figure 1 Estimated sloshing resonance period as function of relative filling height for a typical membrane LNG tank with length 45 m and breadth 38 m.

The prismatic shape of the membrane tanks has been designed to reduce the risk of severe sloshing induced impact loads. The chamfers affect the sloshing resonance periods and reduce the impact loads by changing the impact angle between the free surface and tank boundary.

Figure 2 and Figure 3 illustrate the sway motion and acceleration of a standard size LNG carrier for beam and quartering waves, respectively. Transverse sloshing is excited by a combination of sway acceleration and roll motion of the tank. Note the large effect of wave heading, and the magnification of acceleration at lower periods. Figure 3 illustrates that the vessel has a large acceleration response around sloshing resonance. Deep sea trading vessels will frequently encounter sea-states dominated by waves with periods within the sloshing resonance range.
Figure 2 Sway motion normalized by wave amplitude as a function of wave period for bow quartering, beam seas and stern quartering waves. Highlighted sloshing resonance range.

Figure 3 Sway acceleration normalized by wave amplitude as a function of wave period for bow quartering, beam seas and stern quartering waves. Highlighted sloshing resonance range.

Sloshing is characterized by different fluid flow phenomena depending on the filling level in the tank as illustrated in Figure 4 to Figure 8. The flow changes depending on the ratio of filling height divided by the length of the tank in the excitation direction, e.g. tank filling height divided by tank breadth for transverse sloshing modes. The tank height is an important parameter for tank roof impacts. In the following sections...
the filling ranges are given as a percentage of tank height, and the ranges are based on the ratios of tank breadth and length to tank height for tanks 2 to 4 on board a 140 000 m$^3$ LNG carrier.

In the high filling range (>90% of the tank height) the impacts typically occur on the tank roof at the corners with the transverse bulkheads and the chamfers due to diagonal sloshing motion in the tank. Typically a ‘flat’ fluid surface hits the tank roof at high velocity causing impacts. Impact pressures for this type of impact can be of significant magnitude, but both model experiments and operational experience indicate that the effect is very localized in the corner of the tanks.

![Figure 4](image1.png)

**Figure 4 High-filling (>90%$H$) impact in bow quartering and near head sea conditions**

For filling levels in the range of about 60% to 90% of the tank height the largest impacts occur in the corners and knuckles of the chamfer. If the motion is sufficient to reach the tank roof, it is likely that the highest impact loads occur there. These impacts can be caused by run-ups against the longitudinal or transverse bulkheads or by a ‘flat’ fluid surface impact.

![Figure 5](image2.png)

**Figure 5 High-filling (~70-80%$H$) impact due to a run-up along the longitudinal bulkhead and chamfer**
Figure 6 High-filling (~60-70%H) impact due to a run-up against the longitudinal and or transverse bulkhead

For filling levels in the range of about 10% to 40% of the tank height the largest impacts occur at the longitudinal and transverse bulkheads due to breaking waves. A characteristic phenomenon denoted *hydraulic jump* or *travelling bore* may occur. This wave phenomenon is characterized by a ‘jump’ in the free surface level, which travels at high speed and may cause very high impact pressures. This impact phenomenon is significantly more severe than the phenomena encountered at higher filling levels.

When the liquid level becomes sufficiently low, interaction with the tank bottom and the lower chamfer will disturb the wave and prevent the most violent impact on the longitudinal bulkheads.

Figure 7 Typical low-filling impact in near head sea conditions
Figure 8 Schematic illustration of an *hydraulic jump or hydraulic bore*

For ships designed for unrestricted service, the impact phenomena described for low and intermediate tank filling levels have potential to cause impact pressures beyond the structural resistance of the membrane containment systems. Membrane tanks are therefore normally subject to filling restrictions. Tanks will either have to be filled above an upper filling limit during laden voyages, or below a lower filling limit during ballast voyages. The filling range between the two limits is referred to as the barred filling range. The situation is depicted in Figure 9. The figure also illustrates what is considered to be normal filling practice for membrane LNG tanks in full load and ballast conditions.

Operation within the barred filling range will in many cases be possible at specific sites. The feasibility of such operation, and the potential operational limitations that may apply, shall be determined by sloshing assessments.

Figure 9 Illustration of the filling limits and the barred filling range

Sloshing model experiments are required to assess the impact loads caused by violent sloshing. Sec. 3 gives a detailed description on how to conduct sloshing experiments, and how to establish sloshing design loads.
The sloshing loads vary in magnitude, duration and load area. In addition, the containment system and hull structure have different failure modes. Consequently, a careful analysis of the structural response and strength needs to be conducted for the various loads to assess the structural integrity. A detailed discussion on the response predictions, the failure modes and the strength are given in Sec.5 to Sec.8.

The remaining sections of this chapter outline in more detail the purpose and background of this class guideline.

2 Membrane type LNG tanks

This class guideline considers liquefied natural gas (LNG) membrane tanks as used in LNG carriers and LNG floating production and/or storage units.

Membrane type LNG tanks are defined according to the International Code for the Construction and Equipment of Ships Carrying Liquefied Gases in Bulk, see the IGC Code /1/, and RU SHIP Pt.5 Ch.7 Liquefied gas tankers. They are non-self-supporting tanks which consists of a thin layer (membrane) supported through insulation by the adjacent hull structure. The membrane is designed in such a way that thermal and other expansion and contraction is compensated for without undue stressing of the membrane.

3 Purpose of this document

The purpose of this class guideline can be summarised as follows:

— specify the Society’s requirements for approval of membrane type LNG carriers and floating structures exposed to liquid sloshing in their cargo tanks.
— provide the industry with the necessary methodology to assess sloshing loads, and to evaluate its impact on the design of membrane type containment systems for LNG, the supporting hull structure, and the pump tower structure.
— provide the industry with guidance on how to use this methodology to comply with the Society’s class requirements.

4 Applicability of this document

This class guideline is mainly applicable to sloshing assessment of membrane LNG carriers or other vessels designed to carry LNG in membrane cargo tanks, for which acceptable safety against sloshing damages are not considered proven through service. Typical application examples are:

a) Novel LNG carriers designed for operation within the barred filling range as defined in Figure 9. This will typically be vessels with increased cargo capacity, increased cargo tank size, or different cargo tank proportions compared to sea proven conventional size vessels as defined in Sec.2 [1.2].

b) LNG carriers or other vessels designed for operation within the barred filling range at specific sites or on specific trade routes, e.g.:
   — vessels designed to operate without filling restrictions or with other filling restrictions than normally accepted.
   — ships designed to operate on a special trading route or in a restricted sea area.
   — ships that will load or discharge cargo at offshore terminals.
   — vessels that will operate as offshore terminals, production units, storage units, regasification units or similar. (FLNG, RV, FSRU, FPSO etc).

c) Evaluation of the filling limits that define the barred filling range.

For applications in category b) above, the outcome of the sloshing analyses may be operational restrictions for the vessel in terms of e.g. wave height and wave heading. The operation of the vessel will in that case require suitable operational procedures to ensure operation within these limits.

The strength assessment methodology used throughout this class guideline is based on operational experience with membrane type LNG tank systems, and this limits its application to the Gaztransport &
Technigaz licensed Mark III and NO96 membrane type LNG insulation systems. It is, however, believed that the methodology in some cases may be extended to cover novel systems based on a similarity consideration with the existing systems. This possibility shall be discussed and agreed with the Society based on a review of the system.

The strength acceptance criteria documented in the class guideline are specific to the Mark III and NO96 membrane type tank systems. The structure and formulation of the criteria may be useful in the formulation of the strength criteria for novel systems. The class guideline provides detailed information on the specification, execution and analysis of sloshing experiments. Consequently, the procedures described herein may be used to assess other applications than the specific applications listed above.

5 Applicable rules and class guidelines

The classification of LNG carriers is governed by the following rules and regulations in addition to the requirements for main class:

— DNV GL rules for classification of ships RU SHIP Pt.5 Ch.7 Liquefied gas tankers.

In addition to these documents, the Society enforces limits on the allowable tank filling levels and limits on the allowable cargo tanks dimensions. These limits are subject to continuous considerations, and the applicable limits can be obtained upon request to the Society.

5.1 Approval procedure for non-standard ships or ship operations

This document describes how a sloshing load and strength assessment can be conducted in order to achieve class approval from the Society. The basic approach as outlined in this class guideline is illustrated in Figure 10. A short summary of the 2nd column describing the sloshing load and strength assessment is given in the following.

The first step starts by describing the design basis and the limit states for the vessel. The result of this is a detailed list describing the specifications, which are used to develop a scope of work for the sloshing experiments. It is strongly recommended that this first phase is discussed with the Society.

Sloshing experiments are conducted resulting in a set of sloshing loads, which are used in the strength assessment of the containment system. Strength acceptance criteria for the containment system(s) is needed to carry out a strength assessment. For the known GTT systems, i.e. NO96 and Mark III, acceptance criteria are established and described in this class guideline.

If a new containment system is under consideration, the failure modes shall be identified and the strength acceptance criteria need to be defined.

6 Documentation

The following documentation shall be provided and submitted to the Society as part of the design review and approval:

— the design basis used for the design of the vessel, i.e. the target case
— the sloshing experimental programme, including the ship motion predictions, the test set-up, the test scope, the post-processing and the results
— the analysis of the sloshing experimental programme
— load and/or strength analysis for containment system and hull strength
— load and strength calculations for the pump tower and supports
— drawings of containment system, pump tower, and supports.
Figure 10 Schematic description of the application of this class guideline to assess sloshing in membrane LNG tanks for the containment system load and strength assessment
SECTION 2 DESIGN BASIS AND PRINCIPLES

1 Design basis

1.1 Materials
The methodology for assessment of the hull structure and the pump tower presented in this class guideline is based on the presumption that materials are selected in accordance with the requirements given in the DNV GL rules for classification of ships RU SHIP Pt.5 Ch.7.
The methodology for the strength assessment of the containment systems is based on the presumption that the materials satisfy the requirements specified by the license owner, Gaztransport & Technigaz S.A.s.

1.2 Comparative basis (reference case)
The basis for the comparative assessment of new LNG carrier designs and operations should be a reference case which represents the LNG carrier designs and operations proven acceptable through service.
The reference vessel is a 4-tank LNG carrier with a total cargo capacity of 130 000 m$^3$ to 140 000 m$^3$ utilising the Gaztransport & Technigaz Mark III or NO96 type containment system. The containment system should be of the same type for the reference case as for the target case. The selection of an appropriate reference vessel may be subject to discussion with the Society.
A design speed of 19.5 knots is assumed.
Wave climate representing the trade routes for the existing fleet of reference type vessels should be considered.
The tank fillings are between 90% and 98.5% of the tank height.

1.3 Containment systems
For the purpose of this class guideline, the target LNG carrier utilises a membrane type containment system applied in prismatic chamfered tank configurations.
The structural strength assessment methods described in this class guideline is applicable to the Gaztransport & Technigaz Mark III and NO96 tank insulation systems, and moderate evolutions of these systems in terms of modified plate thickness, modified support spacing, and modified configurations of internal load bearing structure.

1.4 Cargo tank environment
The strength of the insulation system should be evaluated on the basis of a cargo temperature of –163°C, and a temperature of 20°C at the level of the steel structure in the cargo tank compartment.
A cargo density of 500 kg/m$^3$ is recommended to be used in the analyses.

1.5 Normal operation of LNG carriers
With the exception of vessels trading as specified in [1.7], it is assumed in this class guideline that LNG carriers mainly trade with tank filling levels above 90%H, and only occasionally at filling levels between 90% H and the minimum acceptable upper range.
1.6 Novel vessel designs operated within the standard approved filling range

This refers to LNG carriers with design modifications compared to carriers known to and approved by the Society in the past, and where the design modification is expected to alter the sloshing characteristics of the tanks. This could be an LNG carrier with increased tank dimensions, different tank proportions, different tank dimensions relative to ship dimensions, etc. compared to proven LNG carrier designs.

What is considered to be a proven LNG carrier design will evolve over time, and the designer is requested to contact the Society for the current status.

The number of tanks may be specified by the designer.

The design speed shall be specified by the designer.

It is assumed that the vessel trades on a World Wide basis. Dedicated trade routes should be considered if these could be more severe in terms of sloshing.

A minimum design life of 25 years is assumed for the fatigue calculations of the pump tower.

The carrier will operate with a barred filling range, i.e. the tank filling will be maintained below a specified maximum filling height during operation with low tank filling, and above a minimum allowable fill height during operation with high tank fillings. The filling limits may have to be changed to achieve compliance with the strength requirements for the containment system.

1.7 Operation outside the standard approved filling ranges

This section applies to vessels or offshore units designed to operate outside the standard approved filling range during part of its service life. Examples are vessels that spend limited time discharging at an offshore terminal, or vessel that operate as an FPSO and is permanently moored at an offshore site.

The number, shape and configuration of tanks may be specified by the designer.

The designer needs to define a mooring configuration to be used by the vessels when at site, e.g. spread mooring, bow-turret mooring or mooring alongside an offshore structure. The use of dynamic positioning system (DP) or other means to obtain more optimal relative wave headings should be accounted for.

The designer need to define the cargo discharge procedures.

The forward speed profile of the vessel shall be defined.

The fractions of the vessel’s operation time spent moored on-site and being in normal trading shall be specified. The annual risk of failure during both parts of the operation needs to be within acceptable limits.

To assess this both the relative wave heading profile and the tank filling profile in the different phases of the vessel operation need to be assessed.

During filling and discharging, vessels will normally experience all filling levels inside the cargo tanks. Emergency procedures should be specified in order to account for these in the assessment.

2 Design principles

2.1 Limit state design principles

The sloshing assessment shall be based on the principles of limit state design. This means that all relevant failure modes of the considered structure or structural component should be identified and assigned criteria that define the limits for when it satisfies its required function. The limit is referred to as a limit state, and is formally defined as a condition which the structure, or part of the structure, no longer satisfies its functional requirements.

The limit states will be classified as follows:

1) ULS – ultimate limit states.
The ULS concern the ability of the structure (or system) to resist the action of (extreme) the maximum expected loads or load effects during the design life of the ship. The limit state corresponds to the maximum load-carrying capacity (or strain or deformation) under intact conditions.

2) ALS – accidental limit states.
   The ALS concerns the ability of the structure or system to resist accident situations. The limit state concerns the safety of life, property and the environment in:
   a) intact conditions under the action of abnormal loads
   b) damaged condition under the action of normal loads.

3) FLS – fatigue limit states.
   The FLS concerns the ability of the structure to resist time-varying (cyclic or repeated) loading.

4) SLS – serviceability limit states
   The SLS concern the ability of the structure or system to resist the action of expected loads or load effects encountered during normal use without causing deformations or damages that may cause deterioration of the durability or function of the structure/system.

If the ULS assessment is based on structural strength criteria that ensure insignificant permanent deformation of the insulation system or the membranes, as will be the case if the criteria given in this class guideline are used, the SLS can be disregarded when considering sloshing loads. If, however, the ULS acceptance criteria accept higher deformations within ranges acceptable with respect to the integrity of the membranes, additional serviceability limit states may have to be defined to ensure a suitably low frequency of permanent deformations of insulation and membranes during operation. No abnormal environmental conditions are considered for the ALS, e.g. freak waves, tsunamis, 10 000 years storms.

The possible severe consequences of loss of containment of the membrane system, and the severe economic consequences of damages to the system implies that the functional requirements defining the accidental limit state will be the same as the ones defining the ultimate limit state and the fatigue limit state.

The overall functional requirements that need to be considered when determining the limit states for the structures governed by this class guideline are:
   — integrity of the primary and the secondary membranes
   — the insulation system should be able to withstand the sloshing impact loads without suffering damages or excessive deformations compromising the support of the primary and secondary membranes
   — the inner hull structure should be able to withstand the combined effect of global hull stresses and sloshing impact loads without compromising the support of the tank insulation system
   — the pump tower including its supports should be able to withstand the combined effect of gravity, temperature loads, inertia loads, and drag forces induced by liquid sloshing without suffering damage.

2.2 Risk based acceptance criteria
Acceptance criteria should be defined for all identified modes of failure of the considered structure by the use of risk assessments as outlined in RU SHIP Pt.1 Ch.1 Sec.1 [2.5.8]. Acceptance levels should be set to obtain a risk level for the considered operation consistent with maritime industry standards, or higher standards set by owners or operators.

The general acceptance criteria specified in the class guideline will be applicable to other regulatory regimes, such as the offshore industry, provided that the acceptable risk level is adjusted accordingly.

Risk is defined as the product of probability of failure times the consequence associated with failure. A cost optimal design is achieved when the risk associated with all modes of failure is uniform, and it follows that the acceptance level for the failure modes should depend on the consequence of failure.

Full quantitative risk assessments will normally not be realistic due to insufficient information about the large number of variables and parameters involved. A simpler qualitative assessment as described below is considered to be a more realistic alternative.

Failure modes should be ranked according to the perceived consequences of exceeding the associated limit states. It is recommended that this is done by introducing consequence classes, where a step in consequence
class represents a uniform change in consequence. A factor of ten (logarithmic) between consequence classes is commonly used. A convenient framework for evaluating failure consequences can be found e.g. in the Society’s document DNVGL RP A203.

The acceptable probabilities of exceeding the limit states for the considered failure modes should be set to achieve the defined acceptable risk level associated with all modes of failure. If the failure modes are ranked in consequence classes as described above, the acceptable probabilities of failure will be linearly dependent on consequence class. 

In a risk matrix format, as illustrated in Figure 1, the failure modes for an optimal design according to the procedure described above will occur on the diagonal of the matrix describing that the risk is uniform.

![Figure 1 Example of a risk matrix](image)

Acceptance levels for the limit states for the structures should be discussed and agreed between the designer and the Society.

### 2.3 Scaling of model test loads to full scale

The conditions for designing a valid model scale experiment, where the relationship between all physical quantities in model and prototype are consistent and well defined, can be derived from the mathematical equations governing the physical problem. The conditions are normally expressed in terms of non-dimensional quantities that describe how each physical quantity that enters the problem should be scaled to achieve similarity. For quantities measured in the model experiment these relationships describes how the quantity should be scaled to represent the conditions in a full scale tank (prototype).

It is well established that the wave motion and hydrodynamic pressures in liquids in a confined tank can be studied in model scale provided that the model experiment is designed so that the dimensionless Froude number is similar between model and prototype. Hydrodynamic pressures at prototype scale can easily be obtained from the model by applying the geometric model scale factor. Under idealized conditions this would also apply to the impact pressure. However, the impact pressure is influenced by other phenomena such as the local shape of the wave at impact, the flow of gas between the wave and the wall prior to impact, behaviour of gas trapped between wave and tank wall, etc. Additional similarity conditions shall be met to ensure physical similarity between impacts at model and prototype scale, and hence ensure well defined relationships between impact pressures in the two scales. This is discussed in e.g. /8/ and /9/.
Ref. /9/ identifies the following potential effects that may happen locally at the same time or in sequence during an impact:

1) Escape of the gas in between the liquid and the wall when possible. This leads to a transfer of momentum between liquid and gas.
2) Compression of the gas fraction during the last stage of the impact. The gas is partially entrapped (bubbles or pockets) and partially escaping.
3) Partial condensation of the compressed fraction of gas depending on the speed of condensation with regard to impact duration.
4) Rapid change of momentum of the liquid forced to adapt its shape to avoid the obstacle.
5) Possible creation of shock waves: pressure wave within the liquid and strain wave within the wall. This will happen when the Density Ratio is low or when the modulus of compressibility of the gas is low. The liquid cannot sufficiently adapt its momentum before the impact. Shock waves are necessary to absorb the momentum gap at the surface of discontinuity between liquid and impacting wall.
6) Hydro-elasticity effects during the fluid-structure interaction.

Deviations from the proper similarity conditions will influence the relative importance of the various phenomena. This will affect impact pressure intensities and pressure signatures in a way that cannot be fully accounted for by a scaling law for e.g. pressure magnitude or duration.

Similarity between model scale experiment and prototype is determined by the liquid test medium, the ullage gas, and the thermodynamic states of the liquid and the ullage gas (temperature and pressure). Current research indicates that the main parameters of influence are compressibility of the gas and the density ratio between liquid and gas. It has also been concluded that the target similarity conditions cannot be simultaneously met for these parameters at the scales commonly used in model experiments. Current research does not yet provide conclusive recommendation regarding test liquid, ullage gas, and scaling. A practical engineering approach as described in the following is proposed.

The factor for scaling of model test loads to full scale should be determined by a sloshing assessment of the reference vessel specified in [1.2], considering the operational experience relevant for this kind of vessels. Details about how to establish and use this scale factor is given in [4].

The model tests for the reference vessel and the vessel under consideration, referred to as the target vessel, shall be carried out on the same test rig, and using the same measurement equipment and data acquisitions system. Further, the data processing and analysis methods, as well as the response and strength analysis methods shall be similar for the reference and the target cases. Consistent treatment of the reference and the target cases is likely to reduce potential bias in the determination of the sloshing impact loads and the structural response and capacity assessment.

The scale factor obtained in this way does not only represent the model scale factor for the experimental setup, but also a calibration of the load and strength assessment procedure. This includes the effects of local wall surface geometry, i.e. raised edges and corrugations, that is normally not considered in the model tests, but that has been shown by research and experience to have a potentially significant influence on the local impact pressure.

Alternative methods to determine the scale factor may be accepted by the Society if proper technical justification is provided.

2.4 Comparative treatment of inner hull structure

The loads that act on the cargo containment system (CCS) shall be carried by the inner hull structure of the vessel. Structural collapse of the inner hull structure is likely to cause loss of containment of the tank. Further, adequate support of the CCS is of high importance to avoid excessive interaction loads and deformations that can lead to damages from extreme impact loads and during long time exposure.

The inner hull structure should be dimensioned to provide adequate safety against excessive permanent deformations, and to provide a sufficient support to avoid unacceptable stress concentrations in the cargo containment system.

Sloshing impact loads are considered to occur on a limited part of the sloshing exposed surfaces of the tank, and due to the proportions of the inner hull structure it is only considered relevant for the dimensioning
of the plates and stiffeners that carry the cargo containment systems. The plates and stiffeners have significantly larger proportions than the structural components of the cargo containment system, and are less sensitive to the highest localised sloshing impact loads that will normally be dimensioning for the containment system. Plate and stiffener dimensions will be determined by the integrated loads over a larger area of the structure.

The strength assessment of the inner hull structure may be carried out using simplified comparative assessment methodology considering only the relative change in linear elastic structural response between the considered target vessel and the reference vessel.

2.5 Sloshing design loads
The determination of the sloshing impact loads for assessment of containment system and hull strength should be based on sloshing model experiments. For the pump tower, the hydrodynamic loads associated with sloshing can be considered to be drag dominated, and may be determined based on numerical simulations.

The ship motions used for the sloshing experiments should be determined using a verified ship motion analysis program.

The sloshing design loads should in general be based on long term load distributions relevant for actual vessel operation at sea. The exceedance probability level for the design loads should be determined based on the consequence associated with the considered mode of failure. If the reference and the target are very similar with respect to geometry, tank size and filling level simplifications might be possible.

The sloshing loads considered for the assessment of the containment system are impact loads acting on small areas, typical box or panel sizes and smaller. Larger areas may be relevant for assessment of the hull structure.

The procedure to determine sloshing design loads is presented in detail in Sec.3.

2.6 Structural response analyses
The structural response analysis methodology needs to be capable of accurately predicting the structural response in the response range up to where damages are likely to occur in the structure. Depending on the response characteristics of the considered structure, it may be required to consider non-linear structural response.

The structural response assessment needs to consider and include all physical effects that affect the relative structural response between locations relevant for the assessment of the individual failure modes, and effects that potentially give different results for the reference and the target cases. The most important effects that shall be included are:

— effects of the temperature variations through the thickness of the insulation system.
— dynamic response effects.

Details on how to handle these effects are given in Sec.5.

2.7 Ultimate capacity models
The ultimate capacity models should be able to predict the limit states associated with damages to the containment system.

All physical effects that significantly affect the relative structural strength between the individual failure modes, and effects that may affect the validity of the load scale factor described in [2.3] should be included in the capacity models.

The most important effects that shall be included are:

— effects of the temperature variations through the thickness of the insulation system.
— strength differences caused by differences in the dynamics of the response.

Details on how to handle these effects are given in Sec.6 for the containment systems.
2.8 Fatigue capacity models

The cumulative damage (D) from repeated sloshing impacts shall be calculated according to the Miner-Palmgren theory in combination with S-N curves, a characteristic stress range, and a long term response distribution curve, as follows:

\[ D = \sum_{i} \frac{n_i}{N_i} \]

where the summation is over a number of \( i \) stress intervals, \( n_i \) is the number of load cycles within each stress interval, and \( N_i \) is the number of cycles to failure for the constant stress range of that interval.

2.9 Design format

2.9.1 Cargo containment system

The criteria for the assessment of the cargo containment system are formulated as follows:

\[ S \cdot (p \cdot DAF) \leq R_c \]

where:

- \( S \) = the structural response, in general a non-linear function of the dynamic load
- \( p \) = the design sloshing impact pressure scaled according to the comparative procedure described in [4]
- \( DAF \) = the dynamic load factor
- \( R_c \) = the capacity in terms of the considered response parameter.

2.9.2 Inner hull structure

The criteria for the assessment of the inner hull structure are formulated as follows:

For comparative strength assessment of inner hull stiffeners:

\[ \sigma_{tar} \leq \sigma_{ref} \]

where:

- \( \sigma_{tar} \) = a stress response at a location considered relevant to describe the resistance of the target vessel structure
- \( \sigma_{ref} \) = a stress response at a location considered relevant to describe the resistance of the reference vessel structure
### 2.9.3 Pump tower assessment

The criteria for the assessment of the pump tower and the pump tower support structures are formulated as follows:

\[ S(F_i \gamma_{Fi}) \leq \frac{R_c}{\gamma_M} \]

Where:
- \( S \) = the structural response
- \( F_i \) = the various load components acting on the structure, e.g. sloshing loads, inertia loads, wave induced loads
- \( R_c \) = the capacity in terms of the considered response parameter
- \( \gamma_{Fi} \) = the partial load factors, defined in [2.10]
- \( \gamma_M \) = the partial resistance factor, defined in [2.10].

### 2.10 Partial safety factors

#### 2.10.1 Load and resistance factors for pump tower and supports

Applicable load and resistance factors for the pump tower and supports are given in Table 1.

<table>
<thead>
<tr>
<th>( \gamma_{Fi} )</th>
<th>ULS</th>
<th>FLS</th>
</tr>
</thead>
<tbody>
<tr>
<td>sloshing loads</td>
<td>1.3</td>
<td>1.0</td>
</tr>
<tr>
<td>wave induced loads (inertia, hull girder, pressure)</td>
<td>1.15</td>
<td>1.0</td>
</tr>
<tr>
<td>other loads (gravity, thermal)</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>( \gamma_M )</td>
<td>1.15</td>
<td>1.0</td>
</tr>
</tbody>
</table>

For FLS assessment, a design fatigue factor shall be used. The calculated damage, multiplied with the design fatigue factor (DFF), shall be less than or equal to 1.0:

\[ D \cdot \text{DFF} \leq 1.0 \]

The design fatigue factor shall be taken as /6/:

- \( \text{DFF} = 1.0 \) for parts accessible for inspection
- \( \text{DFF} = 3.0 \) for parts not accessible for inspection, but not substantial consequence of failure
- \( \text{DFF} = 10.0 \) for parts not accessible for inspection, and substantial consequence of failure
3 Application of principles

This sub-section describes the application of the design principles as outlined in [2] using the design basis as given section Sec.2.

3.1 Limit state applications

This section describes the application of the limit states, as described in [2.1], to specific LNG carrier applications.

Consideration of fatigue limit states (FLS) will normally not be required for the membrane insulation systems. This conclusion is based on technical investigations carried out by the Society that can be generalised as a consequence of both the characteristics of the sloshing phenomenon as well as properties of the comparative strength assessment methodology.

The following characteristics of the sloshing phenomenon are important:

1) Significant liquid sloshing motion requires a certain level of ship motions combined with motion periods in the same range as the natural period for the sloshing motion in the tank (resonance). The number of sloshing impacts during the lifetime of a vessel is therefore low compared to what is common for fatigue exposed structures.

2) Extreme sloshing impact loads do not only require the presence of extreme sloshing motion in the tank, but also requires a certain unfavourable fluid surface geometry at impact to avoid gas cushioning. The few high impact loads of this character occurring during the lifetime of the vessel are significantly higher than the other impact loads. The long term sloshing load distribution will therefore show a rapid decrease in load level with increasing number of cumulative impacts.

3) Variations in wave heading and filling levels will lead to variations in the sloshing exposed locations in the tank. This will further limit the number sloshing impacts per impact location in the LNG tank.

A typical shape of the long term sloshing load distribution is shown in Figure 2.

![Figure 2](image)

**Figure 2 Typical long term distribution of sloshing impact loads**

For load distribution like this, representative for 20 years operation, the Society’s investigations have shown that:

— The main contribution to the cumulative fatigue damage comes from the upper range of the response spectrum (the highest loads).
— Fatigue is not a relevant failure mechanism when the hot spot stress level is governed by the ultimate strength requirements as ensured by the comparative strength assessment. This covers most of the relevant hot-spots for the considered insulation systems.

— The fatigue damage at hot spots not governed by the ultimate strength requirements is acceptable for load levels up to what is considered the limits for strengthening of the insulation systems.

These conclusions can be extended to govern longer operational times for the target case, e.g. 40 years operation by the following argument.

The ULS design loads are taken at exceedance probability levels significantly below the lifetime return period, whether this is 20 or 40 years. The maximum response in response spectrum relevant for fatigue assessment will hence be significantly lower than the ultimate strength of the structure for any practically relevant design life time.

3.2 Comparative design – reference case

Only an ULS condition is considered for this case, i.e. no FLS and ALS conditions are considered. Drifting in beam seas due to an engine black-out is considered to occur in a less severe sea state than the ULS requirements for beam seas.

3.2.1 ULS condition

The ULS design load should be derived from a long term distribution of sloshing impact loads. The ULS condition should be based on the accumulated operational experience for the existing fleet of LNG carriers that has been confirmed to operate without sloshing damages.

Environmental conditions relevant for the actual trade of the LNG fleet should be considered. A trade specific scatter diagram can be developed based on global wave statistics given in the Society's document DNVGL RP C205.

The reference case sloshing impact loads should be defined as the most probable highest sloshing load experienced over the accumulated damage free operation time for the entire fleet. It follows that only the part of the fleet that has been inspected and found without damages caused by sloshing impacts should be considered.

If damages or deformations of the cargo containment system of the reference vessels have been identified, the load level considered not giving damages are relevant to consider as an acceptable reference load level.

The short-term sea states shall be modelled by the two-parameter Pierson-Moskowitz spectrum, see DNVGL RP C205.

Waves shall be modelled as short-crested.

A representative wave heading distribution should be used. If no accurate information is available, an equal probability of all headings assumption is recommended. A minimum step size of 15 degrees is recommended. To account for the voluntary heading changes, a significant wave limit \(H_s\) of 7 meters should be used for beam sea conditions (60-120 degrees). The probabilities related to sea state exceeding this \(H_s\) limit for beam sea conditions should be assigned to headings close to head sea (165 degrees).

For quartering sea conditions (30-60 and 120-150 degrees) the same principle of voluntary heading change applies. The Society recommends that a linear varying limitation from 7 meters at 60 and 120 degrees to 14 meters at 30 and 150 degrees. The probabilities related to sea state exceeding this \(H_s\) limit for quartering sea conditions should be assigned to headings close to head sea (165 degrees).

A ballast loading condition and a loaded condition should be considered. For the latter a fully loaded or partly loaded condition may be used. The loading condition giving the most representative ship motions should be used to determine the ship motions for the sloshing analyses.

The speed of the vessel should reflect typical operating speed. The speed of the vessel used in the analysis has an important effect on the sloshing loads.

In following and stern quartering seas (135 and 180 degrees) no speed loss is anticipated.

For beam sea conditions (45 to 135 degrees) a reduction to 2/3 of the design speed is recommended for waves exceeding 5 meters significant.
For head sea and bow quartering seas a reduction to 2/3 of the design speed is recommended for significant wave heights between 4 – 7 meters, and a reduction to half the design speed for waves exceeding 7 meters significant.

Use of other assumptions than those described above may be accepted if justified based on operational experience.

3.3 Novel vessel designs operated within the standard approved filling ranges

Only an ULS condition is considered for this case.

3.3.1 ULS condition

The ULS design load should be derived from a long term distribution of sloshing impact loads.

For vessels designed for unrestricted operation the ULS condition should be based on North-Atlantic operation. Wave scatter diagrams for the North-Atlantic Ocean can be found in IACS Rec. No. 34, see [2], or the Society’s document DNVGL RP C205.

Vessels could also be designed to operate on specific routes, with actual wave climate conditions. Representative wave conditions are given in the Society’s document DNVGL RP C205.

The short-term sea states shall be modelled by the two-parameter Pierson-Moskowitz spectrum, see DNVGL RP C205.

Waves shall be modelled as short crested waves.

Wave headings from following (0 degrees) to head waves (180 degrees) shall be considered. A minimum step size of 15 degrees is recommended.

As captains normally voluntarily will avoid beam sea conditions in the most severe sea states, a significant wave height ($H_s$) limit may be introduced for sea conditions (60-120 degrees). The Society recommend a limitation of 7 meters, but other limitations may be accepted if more precise information of operation of the vessels is available. The probabilities related to sea state exceeding this $H_s$ limit for beam sea conditions should be assigned to headings close to head sea (165 degrees).

For quartering sea conditions (30–60 and 120–150 degrees) the same principle of voluntary heading change applies. The Society recommend that a linear varying limitation from 7 meters $H_s$ at 60 and 120 degrees to 14 meters at 30 and 150 degrees. The probabilities related to sea state exceeding this $H_s$ limit for quartering sea conditions should be assigned to headings close to head sea (165 degrees).

The speed of the vessel should reflect both the involuntary and voluntary speed loss in different wave conditions and relative wave headings, experienced by the vessel under operation. The speed of the vessel used in the analysis has an important effect on the sloshing loads.

In following and stern quartering seas (135 and 180 degrees) no speed loss is anticipated.

For beam sea conditions (45 to 135 degrees) a reduction to 2/3 of the design speed is recommended for waves over 5 meters.

For head sea and bow quartering seas a reduction to 2/3 of the design speed is recommended for significant wave heights between 4 – 7 meters and a reduction to half the design speed for waves exceeding 7 meters significant.

A ballast loading condition and a loaded condition shall be considered. For the latter, a fully loaded or partly loaded condition may be used. The loading condition giving the most severe ship motions shall be used to determine the ship motions for the sloshing analyses.

Use of other assumptions than those described above may be accepted if justified based on operational experience.

3.4 Operation outside the standard approved filling ranges

Only ULS and ALS conditions are considered for this case.
### 3.4.1 ULS condition

The ULS design load should be derived from a long term distribution of sloshing impact loads.

The ULS design load should be based on wave conditions from a specific site/trading route.

The short-term sea states shall be modelled by a wave spectrum representative for the operational site/trading route. If no specific wave spectral information is available the JONSWAP wave spectrum may be used with a peakness parameter as described in the Society’s document DNVGL RP-C205, [3.5.5.5].

Waves should be modelled as short-crested or long crested depending on what is considered most representative for the operation site/trading route.

Special care shall be taken if multi directional waves are expected on the site.

For a bow-turret moored LNG carrier wave headings from beam seas (90 degrees) to head waves (180 degrees) should be considered. A minimum step size of 15 degrees is recommended.

A weather vaning bow-turret moored LNG carrier will experience predominantly head sea waves. A low-frequency drift study may be used to determine the probability distribution for headings from beam to head sea waves. Such a study needs careful modelling or assessment of at least the following aspects:

- draft and trim variations
- joint modelling of wave, current and wind
- the effect of swell
- mooring characteristics
- shallow water effects (if present) (wave and ship dynamics).

For vessels where mooring limits weather vaning, wave headings from following (0 degrees) to head waves (180 degrees) should be considered. A minimum step size of 15 degrees is recommended.

For vessels using dynamic positioning (DP) to control wave headings during operation it is recommended to contact the Society in order to discuss a relevant set of wave headings that need to be considered in the sloshing experiments.

For an LNG carrier that will operate inside the barred filling range on a specific route, wave headings from following (0 degrees) to head waves (180 degrees) should be considered.

The actual operation of the vessel needs to be reflected in the analysis. This includes e.g. the tank filling and/or discharge procedure, cargo transfer procedures that affects the filling level probability distribution for the tanks.

The number of tanks simultaneously operated in partial filling should be considered in the analysis.

Ballast, fully loaded, and intermediate loading conditions should be considered. The loading condition giving the most severe ship motions should be used to determine the ship motions for the sloshing analyses.

While on-site no forward speed is normally to be considered.

### 3.4.2 ALS conditions

Accidental situations having a low but not negligible probability of occurrence, and where the resulting operation scenario cannot be considered governed by the ultimate limit state assessment, will have to be specially addressed as accidental limit state conditions.

ALS conditions will be relevant in situations where the ULS assessment is based on a set of operational restrictions, and where accidental or emergency scenarios can be envisaged where the specified operational limits may be exceeded.

Examples of emergency and accidental events that may potentially lead to situations that require consideration of accidental conditions are:

- a vessel is forced to leave sheltered discharge locations with partially filled tanks due to an emergency situation.
- loss of passive or active wave heading control during partial fill operation due to disconnection from mooring or loss of thruster power.

Rare and extreme weather events such as hurricanes/typhoons may represent an ALS condition in cases where the environmental conditions created by these events are not reflected in the design basis (wind, wave, current, etc.) for the ULS.
The relevance of accidental conditions are dependent on the probability for coincident exposure to other operational or environmental parameters outside the specified operational limits, as well as the existence of emergency plans to mitigate risks during accidents or emergencies. In the examples above this would typically be:

— the probability to encounter sloshing critical sea states and/or wave heading during emergency departure
— the probability to encounter wave environments outside the operational limits for beam sea operation
— the ability and preparedness to avoid unfavourable wave heading exposure by alternative operational measures such as navigation away from site
— the capability to redistribute cargo between tanks to limit exposure to hazardous conditions.

Relevant accidental scenarios should be identified in each particular case by operational risk analysis like HAZOP studies or similar.

The expected frequency and duration of the scenarios should be estimated to the extent possible. This is important for determining relevant environmental conditions and acceptance criteria for the analyses.

Emergency operation procedures should be considered in the analyses.

The short-term sea states should be modelled by a wave spectrum which is representative for the specific site/route.

4 Strength assessment methodology

The overall methodology for the strength assessment of the membrane type insulation systems and its supporting hull structure are summarised in Figure 5. In the figure it is assumed that the reference load is established, but in principle this shall be done in a similar way as described for a target vessel.

The comparative aspect of the methodology lies in the assessment of the reference case, which is used to harmonise the load and the capacity to be consistent with the operational experience of membrane type LNG carriers operated to date. In practice this is achieved by scaling the sloshing design loads, but the harmonisation also covers other systematic uncertainties in the procedure. The sloshing impact load is considered to represent the largest uncertainty. This is related to scaling of experimentally determined impact loads from model scale to prototype scale, and the effect of membrane surface geometry. See [2.3] for a discussion of this.

The methodology can be summarised as follows:

Reference case (see Figure 3):

1) Establish a curve relating sloshing impact pressure and sloshing exposed area based on experimental results as described in Sec.3. Load factors should be disregarded in this step.
2) Establish a curve relating the impact load capacity of the insulation system and the sloshing exposed surface area of the structure as described in Sec.6. Resistance factors should be disregarded in this step.
3) Establish the ratio between the load and the capacity for the entire range of load areas, and identify the maximum ratio between load and capacity. Denote this ratio by $\alpha_{comp}$.
4) Scale the load uniformly for all load areas using the maximum identified ratio between the load and the capacity. The scaled load will now for any load area size be lower than the ultimate capacity of the insulation panels. This step is motivated by the damage free operational experience with the membrane type LNG carriers.

The resulting load curve should be the basis for the strength assessment of the insulation system and its supporting hull structure.
Figure 3 Scaling of loads for reference case to the ULS capacity

Target case (see Figure 4):
1) establish a curve relating sloshing impact pressure and sloshing exposed area based on experimental results as described in more detail in Sec.3.
2) scale the load using the maximum ratio, $\alpha_{comp}$, between load and response determined from the reference case.
3) carry out a strength assessment of the insulation system according to the procedure specified in Sec.6.
4) carry out the necessary reinforcement of the insulation system so that the load for any load area is lower than the ultimate capacity of the insulation panels.
5) carry out a strength assessment of the supporting hull structure according to the procedure specified in Sec.6.

The strength assessment should be carried out for all insulation structure elements that will experience sloshing impact loads in the cargo tank. In practice this means the dedicated transverse and longitudinal corner/knuckle structure and standard flat wall structure adjacent to the corner/knuckles. The specific locations are determined by the applicable tank filling limitations and the operation of the vessel.

Figure 4 Scaling of loads for target case using the same scale factor as for the reference case. Strengthen containment system according to the scaled load curve

A single comparative load scaling factor, $\alpha_{comp}$, representative for the weakest element of the considered insulation structures should be applied in the assessment of all relevant insulation structures of the target
vessel. This means that potential strength margins determined for the reference case can be utilised in the target case.

**Figure 5 Overview of the application of the strength assessment methodology**
SECTION 3 SLOSHING IMPACT DESIGN LOADS

1 Introduction

1.1 Background

Use of risk based strength acceptance criteria requires prediction of the probability that the sloshing impact load will exceed the structural resistance of the cargo containment system during the intended operation of the vessel. Such predictions generally require knowledge of the statistical distribution of sloshing impact loads for the intended operation of the vessel over its service life considering all expected sea state encounters, wave heading encounters, and tank filling encounters. Such statistical distributions are normally referred to as long term distributions.

Long term distributions are derived by combination of a number of so called short term statistical distributions. Short term distributions represent the statistical distribution of the sloshing impact load for a specific sea state, wave heading, tank filling, or any other environmental or operational parameter identified to be relevant for the assessment. It is recommended that the short term distributions are derived by statistical analyses of impact pressures recorded during sloshing model experiments. Signal processing and statistical analysis methods are described in [2].

Model experiments are conducted by exposing a scale model of the tank to the predicted motions of the vessel for the considered sea state, heading, and filling conditions. Guidance for environmental and operational modelling required both for ship motion analysis and for the long term statistical analyses of the measurements are given in [3]. Ship motion analyses are described in [4].

Guidance and recommendations for execution of sloshing model tests are given in [5]. Model testing is considered to be the only method currently available to reliably predict sloshing induced impact pressures. The model test tank shall be equipped with suitable impact pressure measurement system that shall be connected to a data acquisition system that can record the impact pressures occurring during the tests.

It follows that model experiments will have to be conducted for a range of different sea state, wave heading, and filling level combinations to be able to determine the long term distribution. Guidance on how to efficiently execute the model test program to achieve the best possible accuracy of the long term distribution is given in [6].

Sloshing design loads are derived from the long term distribution such that compliance with the strength acceptance criteria will ensure a suitably low probability that the load will exceed the structural resistance. Guidance on design load assessment is given in [7].

[8] gives recommendations on how experience, physical insight, together with data analysis to evaluate and improve the quality of the statistical analyses.

2 Statistical analysis

2.1 General

The purpose of the statistical analyses described in this section is to derive the long term distribution of sloshing impact loads for the various structural elements in the tank. The main steps in this process are:

— Processing of impact pressure time series to develop discrete short term statistical distributions.
— Processing of measured time signals to produce modified signals representing combinations of the original signals.
— Processing of measured and combined time series to determine the dynamic characteristics of the measured impacts.
— Fit of continuous theoretical statistical distributions to the discrete statistical distributions obtained from model tests.
— Compile long term statistical distributions for the impact pressure considering operational environment and operational parameters such as wave heading encounter, tank filling encounter, etc.

In addition to the main steps mentioned above, the section provides guidance and recommendations on how to make the most efficient use of the available test data, and outlines how to improve the quality of the prediction by efficient use of the information contained in the available data.

### 2.2 Long term distribution

The long term exceedance probability function for sloshing impact loads can be determined as follows:

\[
Q_{\text{long-term}}(p_a) = \sum_{i=1}^{\text{Sea States}} \sum_{j=1}^{\text{Headings}} \sum_{k=1}^{\text{Fillings}} p_k \cdot p_j \cdot p_i \cdot w_{ijk} \cdot Q_{ijk}(p_a)
\]

where:

- \(Q_{ijk}(p_a)\) = the exceedance probability function of sloshing impact peak pressure values given sea state \(i\), heading \(j\), and tank filling \(k\)
- \(w_{ijk}\) = the weighing factor to account for relative number of impacts in condition \(i, j, k\)
- \(E_{ijk}\) = the impact event rate for short term condition \(i, j, k\)
- \(p_i\) = probability for sea state range \(i\)
- \(p_j\) = probability for relative wave heading \(j\)
- \(p_k\) = probability for filling range \(k\)

The probabilities to encounter sea state \(i\), heading \(j\), and filling range \(k\), \(p_i, p_j, p_k\), should be assessed based on relevant environmental conditions and the intended operation of the vessel. This is discussed in [3.1] and [3.2].

A sloshing event is defined as an impact pressure that exceeds the specified pressure threshold. In the equation defining the long term load distribution, the relative difference in impact frequency between the different short term conditions is taken into account in \(w_{ijk}\). This is worth mentioning as the load frequency varies much more between different short term conditions for sloshing induced impacts than for e.g. ship sectional forces due to wave loads.

In structural reliability and risk analyses it is common practice to formulate the failure acceptance criteria in terms of annual probability of failure. It is hence convenient to formulate the sloshing impact design load in terms of annual probability of exceedance. The annual probability of exceedance distribution can be established as follows:

\[
Q_{\text{annual}}(p_a) = 1 - (1 - Q_{\text{long-term}}(p_a))^N
\]

Where:
\( Q_{\text{long-term}}(p_d) \) = the long term exceedance probability function

\( N \) = the average annual number of impacts.

This formulation requires that the recorded sloshing impacts can be considered as statistically independent. This is considered to be a reasonable assumption.

The load exceedance probability level to consider as design load will depend on the consequence of the related failure mode, and should be agreed with the Society for each particular case.

2.3 Identification of pressure peaks

A peak-over-threshold method is recommended to identify the impact peaks and separate them from noise in the signal and hydrostatic and hydrodynamic pressures.

Maxima shall be identified as illustrated in Figure 1. A moving time window may be used to identify only these global maxima within the window settings. Alternatively a minimum required time-step between global impact peaks may be set.

The threshold should be set well above the noise level in the pressure signal. The methodology used in the long term load assessment should not be sensitive to the selected threshold, since the focus is on the low probability high pressure peaks.

A variation of the moving window size and the threshold is recommended to identify parameters for these such that reliable and converging results are obtained.

This process should be combined with the Maximum per event method described in [2.4].

2.4 Maximum per event method

The aim of the sloshing strength assessment is to ensure that the risk of cargo tank damages caused by sloshing is at an acceptable level. As the cargo containment system is similar for large parts of the tanks, the sloshing design loads shall represent an acceptable probability of exceeding the containment system strength capacity at any location in all tanks on board the ship. To achieve this, the statistical distributions of sloshing impact loads from which the design loads are derived should represent the envelope of impact loads that occur on any location in any tank of the ship, and not at one specific position in one tank. (This can most conveniently be derived from short term distributions that represent the highest impact loads on a given surface area per sloshing impact on board the ship.)

Sloshing model tests cannot be carried out for all tanks on board the vessel. Neither is it feasible to instrument all locations in the tested tank. It is, however, expected that good estimates can be achieved.
if the most sloshing exposed regions of the most sloshing exposed tank of the vessel is covered by the instrumentation. In that case the maximum sloshing impact pressure during an impact event is likely to occur somewhere inside the region covered by the instrumented region. Each sensor or group of adjacent sensors in the instrumented region represents a certain surface area of the tank, and it is assumed that the structural configuration of the insulation is similar across the region. The short term distribution for the maximum pressure on a certain surface area during an impact event can therefore be obtained by sampling only the maximum peak pressure recorded among the sensors (or relevant configurations of groups of adjacent sensors) in the instrumented region each time an impact occurs.

In practice this can be done by establishing new channels/time series by combining the different sensors/sensor combinations to obtain the maximum average pressures for a given loading area for each impact event. The method will be referred to as “maximum per event – method” in this class guideline, and is illustrated in Figure 2. Here three different signal time series \( S_1, S_2, S_3 \) are combined into one new “Max per event” signal. Different colours are used to illustrate which peaks are included from the different channels.

![Figure 2 Illustration of “Maximum per event” – method](image)

The Max per event channels are then used in the statistical processing described in [2.6]. Besides providing more relevant information about the loading of the tank walls, the maximum per event – method also has the benefit of giving a larger data set and hence allows for more reliable fits to theoretical statistical distributions and thereby prediction of extreme values.

### 2.5 Load area processing

Model tanks are normally instrumented with clusters of pressure sensors. As indicated above this is done to improve the likelihood of capturing the maximum impact pressure occurring during each sloshing event, but also to allow for evaluation of the average pressure over larger surface areas of the tanks than what is described by the individual sensors. Such information is important to be able to assess the strength of all relevant components of the containment system.

Signals representing the average pressure over various surface areas of the tank can be obtained by summing up the time signals for adjoining sensors. Signals should be developed for all possible combinations of adjacent sensors within the sensor clusters that represent the same continuous loaded area. These combinations can conceptually be viewed as virtual sensors with larger reference area.

If the pressure over the sensor cluster area has been approximated by a continuous pressure field by interpolation of the discrete measurements, virtual sensors can in principle be developed for any size area within the tank.
2.6 Statistical processing of pressure peaks

The identified pressure peaks shall be statistically processed to develop short term statistical distributions of impact pressure.

A histogram of the identified sloshing impacts shall be determined. A large number of bins are defined from zero to the largest impact pressure measured. All the identified peaks are sorted in these bins in order to establish the histogram.

When normalising the area under the histogram the discrete probability density function (pdf) is obtained. Integration of the PDF gives the cumulative distribution function (CDF). Given the occurrence of an impact peak, the CDF gives the probability that this peak is lower than a given pressure value. Since the main interest is in the extreme values the CDF is preferably presented in the form of the Exceedance Probability Function (EPF) on a logarithmic scale. Figure 3 illustrates the various probability functions.

Reference is made to textbooks on statistics for further details about the theory of probability functions.
Figure 3 Schematic procedure to determine sloshing impact peak pressure probability functions

From the peak identification process the total number of identified sloshing impact peaks is obtained. The total time duration in which these peaks occurred is known as well, hence the average time between successive sloshing impact peaks can be calculated. This average time is often called the *response period* and the average number of impact recordings per hour is called the event rate as defined in [6].

Fitting of the discrete statistical data may be done using a mathematical function, like the Weibull or the Pareto distribution. It should be remembered that none of those mathematical functions have a fundamental relation to the physics dominating the randomness of the impacts. A mathematical fit should be plotted together with the discrete data. A mathematical fit is convenient to use for further mathematical processing and necessary for extrapolation beyond the probability levels measured in the model test campaign.
When conducting a series of irregular motion sloshing experiments for an identical condition, the results should be considered as a single long test, and one probability function should be established based on the entire test duration. Longer duration tests give more reliable statistical distributions.

The statistical post-processing procedures as outlined in this paragraph apply to signals obtained from single pressure sensor as well as signals resulting from a summation/integration over a group of pressure sensors as described in [2.5].

2.7 The Weibull statistical distribution

The 3 parameter Weibull distribution is most commonly used to describe the probability density of the sloshing impact pressures. This function is given by

\[ f(p; k, \theta, \lambda) = \frac{k}{\lambda} \left( \frac{p - \theta}{\lambda} \right)^{k-1} e^{-\left( \frac{p - \theta}{\lambda} \right)^k} \]

where:

\( k \) = the shape parameter  \\
\( \theta \) = the location parameter  \\
\( \lambda \) = the scale parameter.

The cumulative distribution of the probability density function is given by.

\[ F(p) = \int_0^p f(p) dp = 1 - e^{-\left( \frac{p - \theta}{\lambda} \right)^k} \]

The probability of exceedance function,

\( Q(p) \)

is then found by

\[ Q(p) = 1 - F(p) = e^{-\left( \frac{p - \theta}{\lambda} \right)^k} \]

Figure 4 illustrates how the three-parameter Weibull distribution is fitted to sloshing induced impact pressure peaks from a tested short term condition. The fit seems reasonable, but any use around and at lower probability levels than where experimental data exists is associated with uncertainty. This is further discussed in [7]. There is no theoretical reason behind the choice of a three-parameter Weibull function to describe the distribution of impact pressures, but experience has shown that this distribution in most cases provide reasonably good fits to the data in the tail of the distribution.
2.8 Dynamic characterisation of impacts

The dynamic response of the cargo containment system depends on the time variation of the pressure signals, and it is therefore necessary to characterise the dynamic nature of the impact events.

Studies have shown that the time from the start of the impact to the time of the maximum peak, normally denoted the rise time of the pressure signal, is the key parameter for describing dynamic amplification of structural response. The shape of the decay part of the signal may have some influence on the response, but this is considered to be of secondary importance.

For impacts with duration significantly shorter than the natural period of the structure, the structural response will be uniquely determined by the impulse of the load. The response to such impulsive loads will, however, generally be smaller than the quasi-static response to the peak impact loads, and are hence of less importance from a safety point of view.

Figure 5 illustrates the characteristics to be determined. For some impacts the definition of rise time is not straightforward, as the slope of the impact may vary significantly. To handle such cases it may be more appropriate to define the rise time as twice the time from half the peak value to the peak value, see Figure 6.

The impulse is defined as the pressure integration of the impact.
The use of the rise time in assessing the dynamic response of the containment system requires a transformation of the time scale from what is measured in model scale to what will occur in prototype scale. It is normal practice to use Froude scaling to get a first estimate of the prototype time scale. This means that the prototype time is obtained by multiplying the measured time in model experiments by the square root of the geometric scale factor between the scales, i.e.:

\[ t_{\text{prototype}} = t_{\text{model}} \cdot \sqrt{\alpha} \]

Where:

\( \alpha \) = the geometric scale factor between prototype and model
Because of lack of full physical similarity of impacts between model test and prototype, as discussed in Sec. 2 [2.3], Froude scaling of the rise times measured in model scale may not be fully accurate. Rise times are hence associated with some uncertainty, and some conservativeness is recommended when using the model test results to assess structural dynamic effects.

It is recommended to treat the rise times in a statistical manner. Ideally, joint probability distribution of rise times and impact pressures should be made to identify the rise time for the most likely largest impact pressure. Due to the previously discussed uncertainties in the model scale rise times it is however acceptable to do it in a more simplified manner, with main focus on the sensitivity. The procedure is outlined in the following.

Rise times from all the tests are sorted in bins and plotted as a histogram. Figure 7 illustrates the results of such processing for three different situations depending on the choice of pressure threshold used in the signal processing. The first figure illustrates the probability density function if all the measured impacts are included in the probability distribution. This gives a broad distribution, with a large scatter in the observed rise times. As the pressure threshold is increased the probability density function gets narrower and more peaked as illustrated in the two next figures. The variability in rise time among the highest recorded events is lower than for the entire population.

The high pressure design sloshing event has a low probability of occurrence, and is not likely to be represented among the events recorded in the experiments. It is nevertheless reasonable to assume that the characteristics of the design event are better represented by the events with the highest recorded pressures than with the entire population. It is therefore recommended that the “design” rise time is determined from the peak value of a distribution determined using a high threshold. A reasonable threshold value should be determined based on a parametric study. It is important to ensure that the number of impact events included in the sample remains sufficient to get a reasonable estimate of the distribution.

It is important to look at the sensitivity in the choice of rise time and the related dynamic response of the containment system. This is both related to the uncertainty in the scaling of the rise time as discussed above, but also related to the variability in the measured rise time. The dynamic structural response should therefore be calculated at least one standard deviation lower and higher than the peak value. The highest DAF should be chosen for design.
The above procedure is incapable of providing a statistically consistent dynamic amplification factor since it does not consider the correlation between the rise time and the measured peak pressure value.

An alternative way is to include the dynamic response factor in the statistical post processing of the model test pressure signals. The DAF is then calculated for each impact event based on its rise time and the procedure in Sec.5 [6]. The equivalent static pressure is calculated for each event, and a statistical distribution for this equivalent static pressure is calculated. A statistical consistent DAF can be determined from the curves as indicated in Figure 8. In this way load statistics including the dynamic response amplification factor can be derived on relevant probability levels. The uncertainty related to scaling of rise times becomes very important for this method, and the simplified method described above might be just as precise.
2.9 Fluid structure interaction

Fluid structure interaction (FSI) describes a situation where the hydrodynamic and the structural responses are coupled. This means that the hydrodynamic forces depend on the response of the structure and vice versa.

FSI effects are typically not reflected in model testing as the test tanks are rigid. Based on available documentation it appears that FSI effects represent a minor uncertainty compared to for instance the overall statistical uncertainty, and that they may be disregarded in the analysis.

3 Environmental and operational modelling

3.1 Environmental modelling

The wave environment shall be described by a wave scatter diagram. The scatter diagram gives the probability of occurrence of short-term sea states. The basis for the scatter diagram, i.e. measurements and/or observation period, is to represent the long-term wave climate. The number of occurrences in the scatter diagram should therefore be sufficiently large; for example 100 000 occurrences considering a duration of 3 hours per occurrence, i.e. a total duration of ~34 years.

For partial filling operation there are requirements for site specific environmental data. This data is often based on a limited number of measurements. Simulations and/or extrapolations may then be needed to define sea state probabilities at sufficiently low probabilities.

The basis for the sloshing impact design loads is a long term distribution of the sloshing loads. This is obtained by combining sloshing model test results from a wide range of sea states representative for the entire scatter diagram. This means that return period contours for $H_s$ and $T_z$ and the related short term responses are irrelevant. It is the actual probability for each sea state that goes into the assessment.

For the reference case and for increased sizes LNG carriers operating on given trades, adequate environmental conditions for specific routes and areas around the world are defined in the Society’s document DNVGL RP C205. Examples of two specific sets of parameters are given in Table 1.
Table 1 Standard sea state scatter diagrams for North-Atlantic and for world-wide operation

<table>
<thead>
<tr>
<th></th>
<th>North Atlantic IACS Rec.34, /2/</th>
<th>DNVGL RP C205 World-Wide</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha$</td>
<td>3.041</td>
<td>1.798</td>
</tr>
<tr>
<td>$\beta$</td>
<td>1.484</td>
<td>1.214</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>0.661</td>
<td>0.856</td>
</tr>
<tr>
<td>$a_0$</td>
<td>0.70</td>
<td>-1.010</td>
</tr>
<tr>
<td>$a_1$</td>
<td>1.27</td>
<td>2.847</td>
</tr>
<tr>
<td>$a_2$</td>
<td>0.131</td>
<td>0.075</td>
</tr>
<tr>
<td>$b_0$</td>
<td>0.1334</td>
<td>0.161</td>
</tr>
<tr>
<td>$b_1$</td>
<td>0.0264</td>
<td>0.146</td>
</tr>
<tr>
<td>$b_2$</td>
<td>-0.1906</td>
<td>-0.683</td>
</tr>
</tbody>
</table>

The parameters given in Table 1 can be used in the following Weibull conditional log-normal distributions to establish the probability distribution of the scatter diagram as a function of $T_z$ and $H_s$. Parameters for different areas and routes can be found in DNVGL RP C205 and used if it considered more relevant. Scatter diagrams for the parameter sets in Table 1 are also found in DNVGL RP C205 and these are recommended to be used to verify that the implementation of the below equations is correct.

\[
F(H_s, T_z) = F(H_s) \cdot F(T_z | H_s)
\]

\[
F(H_s) = 1 - e^{-\left(\frac{H_s - \gamma}{\alpha}\right)^\beta}
\]

\[
f(T_z | H_s) = \frac{1}{\sigma T_z \sqrt{2\pi}} e^{-\frac{1}{2} \left(\frac{\ln T_z - \mu}{\sigma}\right)^2}
\]

\[
f(H_s) = \frac{\beta}{\alpha} \left(\frac{H_s - \gamma}{\alpha}\right)^{\beta - 1} e \left(\frac{H_s - \gamma}{\alpha}\right)^\beta
\]

\[
\mu = a_0 + a_1 H_s^{\alpha_2}
\]

\[
\sigma = b_0 + b_1 H_s^{b_2}
\]
The short-term sea states are typically described by a parametrized wave spectrum, which is a function of e.g. the mean wave period, a significant wave height and a peakedness factor. Different wave spectra may be used depending on the condition that is investigated, see Sec.2 [3.1]. Recommendations for wave spectral formulation are given in DNVGL RP C205.

Wave spreading should be considered based on environmental information from the actual site/route, and some wave spreading should always be included if the operation involves partial filling outside the standard approved filling ranges.

A typical site specific wave scatter diagram for an LNG FPSO site is illustrated in Figure 9.

3.2 Operational modelling

Operational aspects for the vessel directly influence the long term sloshing impact load distribution and hence the sloshing design loads for the cargo containment system.

A heading probability distribution needs to be established. This should be based on a heading analysis, taking into account the different environmental load actions at the specific site/route. Operational measures, such as for instance DP, could be used to reduce the relative wave angles for higher wave heights.

The number of headings to be considered in the testing should in general be sufficient to ensure a good representation of the actual operation of the vessel. Steps of 15 degrees are recommended but a coarser discretization may be accepted if this can be justified to be conservative. This will however affect the final long term distribution as the need for conservative assumptions will add relatively more weight to the more severe conditions. A typical heading probability distribution for a LNG FPSO is illustrated in Figure 9.

The filling probability distribution should be based on the operational procedure for the vessel. An example of a filling probability distribution for an LNG FPSO is illustrated in Figure 9.

The number of filling levels to consider in the testing depends on the intended operation. For operation with unrestricted filling, tests covering low tank filling, intermediate tank filling, and high tank filling need to be included. These three filling ranges cover typical breaking wave impacts on vertical walls, impacts on the upper chamfers due to standing waves, and impacts on the upper chamfers and tank roof, respectively. For an LNG carrier it will normally be sufficient to consider a representative high filling level and a representative low filling level. At least one filling should represent each filling range. A screening should be carried out to determine the most critical filling for the different filling ranges.

In cases where no single tank can be identified to be more critical than others with respect to sloshing, and the vessel is operated with more than a single tank simultaneously exposed to sloshing, the long term statistical impact pressure distributions obtained from testing of a single tank may underestimate the probability of exceeding a certain load level at any sloshing exposed area for a given type of structural elements of the CCS in the vessel. This can easily be realized by imagining that e.g. four model tanks are exposed to the same motion excitation simultaneously. It is intuitively clear that the expected extreme pressure in a unit time period for all four tanks combined is the same as the expected extreme pressure in 4 unit times for the single tank.

The example illustrates the extreme effect of operating with several tanks exposed. In most cases there will be a bias in criticality towards one of the tanks, either because of tank geometry or the difference of vessel motions at different positions on the vessel, in which case this effect is significantly reduced. In addition, as there is a relatively steep gradient in impact pressure intensity with change in tank filling level around the critical filling levels, a suitably conservative selection of representative tank fillings in the tests and application of the test results in the statistical analysis will be beneficial.

Explicit consideration of this effect is considered necessary only for cases where no single tank can be identified to be more critical than others with respect to sloshing, e.g. in certain partial filling scenarios, and the model test program and the statistical processing of the results have been optimized to remove conservatism. It should nevertheless be noted that the mentioned multi-tank effect tends to increase the risk associated with sloshing. It is therefore generally recommended to design operational procedures to limit the simultaneous exposure of tanks to the critical filling ranges.
3.3 Operational restrictions

In some situations it may be necessary to introduce operational restrictions to achieve compliance with the risk acceptance criteria discussed in Sec. 2 [2.2]. This will normally imply that operation with certain critical tank filling ranges is restricted to occur only within specified limits. This can typically be an upper wave height limit, potentially combined with wave heading limits if heading control systems are in place. Wave periods can also be a relevant operational parameter, provided that the periodicity of the waves is sufficiently far away from the sloshing resonance periods of the tanks. This could e.g. be relevant for short period wind generated sea and very long period swell.
Operational restrictions may be accounted for in the development of the long term sloshing impact load distribution by disregarding the short term operational situations that fall outside the operational limits. The probability of occurrence of the disregarded situations shall generally be reassigned to situations within the operational envelope.

The use of operational restrictions introduces the possibility of accidental situations where the operational restrictions cannot be adhered to. Relevant accidental situations should be identified by risk analyses, and should be taken into account in the sloshing assessment as described in Sec.2 [2.1] and Sec.2 [3.4.2].

4 Ship motion calculation for sloshing tests

The motions for a sloshing test shall be determined by a dedicated ship motion analysis using a verified and validated computer code recognized by the Society. The analyses shall provide linear ship motion transfer functions, which can be used to generate tank motions for specified short-term sea states.

The ship motion calculations shall be conducted for the loading condition and speed specified for the associated sloshing test.

In head and near head waves the surge motion is an important motion or excitation mode for sloshing. The surge motion equation shall be modelled properly in the code. It is therefore recommended to use a ship motion program based on a 3-dimensional boundary element method.

In quartering and beam seas the roll motion is an important motion or excitation mode for sloshing. Roll damping additional to the potential damping shall be accounted for in the roll motion prediction. The effects of bilge keels, fins, skegs, etc. and their possible speed dependence shall be included. This inclusion can be done using linearization techniques. This linearization may be done for varying sea state severities.

The coupled motion effect due to partially filled tanks may be included in the motion predictions. In case of a vessel moored side-by-side with another vessel the hydrodynamic interaction shall be taken into account in the motion predictions. A verification of the implementation of the coupling effect in the ship motion program needs to be documented as the results are highly sensitive to the phasing between the coupling effect and the ordinary ship motions.

Irregular time series of the ship in a seaway shall be calculated by combining a specified wave spectrum with the motion transfer functions. The calculated motions shall be calculated for the motion reference point of the sloshing rig. The motions shall be Froude-scaled for input to the sloshing rig.

Repetition of irregular motion sequences of the tank during the test program shall be avoided. This can be achieved by using an inverse Fourier approach using a very small frequency step size, i.e. a large number of frequency components. Alternatively, the irregular time series can be composed by a linear superposition of harmonic components with un-equidistant frequency step sizes.

5 Sloshing model experiments

5.1 Sloshing experimental lay-out

Sloshing model experiments shall be carried out in a laboratory environment with the necessary safety facilities and complying with the applicable safety regulations.

Sloshing experiments require a motion platform capable or simulating the motions of the tank on-board a ship in a seaway. The motion platform should be able to simulate both regular and irregular motions as specified by time series from a ship motion analysis.

The maximum strokes and angles for the motion platform shall be designed such that the most extreme ship motions anticipated for the intended sloshing tank model size can be simulated. It is recommended not to dimension the rig capabilities only by regular maximum expected motion amplitudes. Especially in beam sea conditions, the combination of large heave, large sway and large roll angles is expected to define the capability envelope for the rig.

The motion control needs to be designed such that the specified ship motions can accurately be simulated. The motion platform, loaded with its maximum payload, shall be equipped with a motion response unit to verify if the generated tank motions correspond with the input motion signals.
The capacity of the motion platform should be sufficient to accurately represent both the motion amplitudes and the phasing between motions of the tank. The tank should be as large as possible and at least at a scale of 1:50.

A six-degree of freedom motion platform shall be used.

The sloshing model tank shall be made sufficiently stiff such that natural frequencies of the tank and its parts do not interfere with the sloshing impact pressures.

It is recommended to manufacture the sloshing model test tank of transparent material such that the behaviour of the fluid motions in the tank can be observed.

The sloshing model tank shall be prepared for the mounting of pressure sensors, single and cluster-wise, at various locations throughout the tank in order to be able to measure sloshing pressures at possible critical areas. The local structure of the tank in areas where pressure transducers are positioned shall be sufficiently stiff to avoid fluid-structure interaction effects.

The sloshing model test tank should be equipped with filling and emptying taps, such that level adjustments are made easily and accurately. At high filling (~95% of tank height) significant differences in impact pressures can be measured for small filling differences. Consequently, the accuracy of the filling level is an important aspect.

In case sloshing tests shall be conducted with varying ullage gas conditions the sloshing model tank structure should be designed for depressurisation. The sloshing model tank should be prepared with taps for gas filling and emptying.

When using different gases or fluids to vary the ullage or fluid conditions inside the tank, preparations shall be made to contain the gases or fluids after testing to comply with environmental regulations if applicable.

5.2 Instrumentation and data acquisition

Pressure transducers shall be mounted to measure the fluid pressures inside the tank. The sensing area should be positioned flush with the inner tank wall.

The shock resistance of pressure sensors should be sufficient not to interfere with the impacts and accelerations expected.

The pressure sensors should be applicable in a wet and/or corrosive environment.

The rated pressures and the maximum measurable pressure should be sufficiently above the expected pressures, which will be measured inside the tank.

The frequency response of the pressure sensor and signal amplifier shall be sufficiently high to capture the pressure fluctuations at least at the intended sampling frequency, i.e. the signal should not be conditioned.

The pressure sensors should preferably be sufficiently insensitive to temperature fluctuations or this should be corrected for in the testing.

The sensing area should be in direct contact with the medium inside the tank. No protective cover or similar caps shall be used, which can affect the dynamics of the measurements.

Pressure transducers should be calibrated before use. In addition to a static calibration, a dynamic calibration is recommended. This can be accomplished by use of drop-tests, where the pressure sensors are mounted in a wedge-shaped section being dropped down onto a flat water surface. The temporal and spatial shapes of the pressure pulse are functions of the impact velocity and wedge dead rise angle. They can easily be accurately calculated for a small dead rise angle, and provide an excellent means of assessing the pressure sensor performance.

The size of pressure transducers should be small to be able to measure local impact pressures and to be able to position several pressure transducers close together. At least 16 sensors should cover a full-scale square area of 1.5m². Common hot-spots for impact loads at high filling are at intersections between the tank roof or chamfers and the tank walls. The sensors should be placed sufficiently close to these intersections so that the corner panels are covered. A 4’4 cluster of sensors (16 in total) should be used in order to facilitate a load versus area estimate based on 9 sensors for both the perimeter zone and the internal area excluding corner boxes.
Hot-spots for impact loads at partial filling are close to the corners, in a range relatively close to the still free liquid surface. High pressures may also occur on the longitudinal and the transverse bulkheads. Generally the impact pressures are less localized in hot spots at partial filling, and the tank should be sufficiently instrumented to assess this.

Studies of symmetry of loading for different corners in the tank should be carried out to assure that the area having the most severe loads are chosen for the final instrumentation.

The data acquisition system should be able to sample the pressure sensors at a sufficiently high sampling rate to capture short impact pressure time histories. Typically sampling rates of ~10 kHz to ~20 kHz are required for sloshing tests at scales of 1:15 to 1:50 respectively.

The pressure time histories and the tank motion histories shall be stored as raw data for further post-processing. Alternatively only data exceeding a pre-defined threshold may be stored. This threshold shall be set low enough to capture all relevant impact peak pressures.

The test setup should have a monitoring system stopping the experiments if for instance the deviation between input and output is too large, or if more than one sensor stops working.

Video capturing of the sloshing tests is recommended in order to study the resulting fluid behaviour inside the tank for specific conditions, like for example the wave heading, the sea state and the filling level.

5.3 Test liquid and ullage gas

As mentioned in Sec.2 [2.3] the choice of liquid and ullage gas for the model experiment will to a large extent define the similarity relationships between the experiment and the prototype.

The model tests can be carried out using water and air at ambient temperature and pressure. This configuration has lower density ratio than LNG and its vapour. This is expected to increase the impact pressure during events where the gas is efficiently displaced by the liquid prior to impact on the tank walls, as it implies reduced relative momentum transfer from the liquid to the gas and hence reduced cushioning of the impact.

Improved similarity in density ratio can be achieved by the choice of a suitable ullage gas for the test model. A mix of sulphur hexafluoride and nitrogen is a frequently used and accepted alternative.

Other test configurations may be accepted by the Society if this can be justified by the designer.

Current research does not yet provide conclusive recommendation regarding test liquid, ullage gas, and scaling. The comparative based scaling factor is expected to reduce the uncertainty related to differences in similarity. The same liquid and ullage gas shall be used in the experiments made to determine sloshing loads for the reference and the target vessel.

5.4 Documentation of a sloshing test facility

The Society requires documentation of test facilities that are used for sloshing model experiments. Generally all details needed to set up the experiment are required in the documentation, and the aspects discussed in [5.1] and [5.2] should be included. A summary of some issues that need to be documented is given in the following. Other requirements may be defined in addition.

— A comparison between input and output motions, with a relevant set of excitation motions needs to be documented. It is important that both the maximum motion amplitudes, both for translations and rotations, and the phasing between the motion modes are reproduced accurately. This should be documented for all frequencies relevant for wave excitation, and for amplitudes representing the most severe sea states. The maximum payload of the test rig should be used in the tests.

— The sensors should be tested in a controlled setup, to demonstrate that the dynamic behaviour of the sensors are acceptable. A wedge drop test setup is recommended, where drop velocities and wedge angles are chosen to reproduce representative impact risetimes and magnitudes from sloshing model tests. Several sensors should be tested, to illustrate the variability between the sensors. It should also be documented that the characteristics of the sensors do not change with wear and tear.

— An overview of the testing setup, with sensors, data acquisition system and data processing should be documented. All relevant information such as technical specifications should be given.
— Documentation of the test facility monitoring system.
— Documentation of the statistical post processing procedure.

The documentation should be on a report format, and should be updated annually or whenever changes are made to the test setup or to procedures related to the testing.

6 Model test program

The model test program shall include a sufficient number of relevant short term probability distributions to accurately describe the long term sloshing load probability distribution. In practical terms this means that it is necessary to test a large number of sea state, wave heading, and tank filling level combinations that cover the vessel operation. The duration of the test needs to be long enough to get converged statistical short term distributions.

Long term distributions for linear ship motion responses are normally calculated from short term distributions with a discretization of 1 second for $T_z$, 1 meter for $H_s$ and 15 degrees for the relative wave heading.

Execution of model tests covering the same range of sea states and wave headings is not considered feasible within an acceptable time frame. Advice on how to establish reliable long term sloshing response values with a limited amount of testing is given in the following.

For each filling level and heading the wave period and wave height range that give sloshing should be identified. The criterion for defining the limit sea state conditions could be that no sloshing impacts above the selected threshold value is recorded in 30 minutes on a load area considered critical.

Further the main contributors to the long term load probability distribution should be monitored during the execution of the testing campaign. It is recommended to generate plots of the relative contribution of the tested condition on the long term probability distribution as illustrated in a simplified way in Figure 10. If the design value is around 15 bars, case 3 and case 4 should be tested to achieve good convergence. In a full model test setup there will be a large number of cases, and it could be difficult to prioritize the cases for long duration tests.

The model test programme shall be composed in a dynamic way where results are continuously analysed to plan the next tests. It is recommended to use the results obtained from the “maximum per incident" method (see [2.4]) to make the decisions for the test matrix development. Results from different load areas should be considered to assure that the critical cases are identified. If the trends vary with area, the critical case selection should reflect any previous experience regarding critical loading area size.

![Figure 10 Probability contribution plot](image-url)
Different sloshing test indicators can be utilized in the screening process. Experience shows that very long model tests are needed to get reliable statistical distribution for pressures. A much faster converging parameter is the event rate, which is defined as the number of incidents per time unit above a certain pressure threshold.

Some linear interpolations can be used between the measured results for the tested conditions, but care should be taken for \( T_z \) values around sloshing resonance. Extrapolation of results outside the tested parameter space is not recommended.

Some cases should be tested with long duration tests to study convergence of parameters of the Weibull distribution which is fit to the measured data. For cases with significant probability contribution at the load range relevant for design, the test duration shall be long enough for the statistical fit to represent the test data adequately. Converged Weibull parameters show however little dependence on wave period, wave height and relative wave heading.

Experience and physical reasoning should be utilized when evaluating the statistical distributions that are found from the testing. As the sloshing impact pressures have a large inherent variability, some model test samples will not accurately represent the underlying (actual) distribution for the given filling, heading and sea state combination. Unreasonable and unphysical conclusions may be reached if the test results are applied without proper consideration around these matters. This is elaborated further in [8]. It is recommended to execute the test program in close dialogue with the Society.

7 Derivation of design loads

The procedure described in [3], [5], [6] and [8] will provide long term distributions of model scale sloshing impacts for the considered loaded areas. Distributions representing the loads at full scale can be obtained by applying the scale factor described in Sec.2 [2.3] and Sec.2 [4].

Annual exceedance probability functions can be derived as described in [1], and the design loads are taken at the probability levels considered relevant for the application. Background for the determination of the design load exceedance probability levels are given in Sec.2 [2.2].

Figure 11 illustrates the annual probability of exceedance distribution (green line) and the long term load distribution per impact (blue line) in the same figure. The annual number of impacts is in this case assumed to be 500. It is further assumed that an annual probability of exceedance of \( 10^{-4} \) is the design target value, indicated with dashed lines. The design load is then given on the x-axis directly.

Figure 11 Extraction of design load from long term load distribution
After conducting a statistical post-processing corresponding to the area of single sensors and clusters of sensors, the loads at the relevant annual probability of exceedance level shall be plotted as a function of load area. Figure 12 gives an example.

![Figure 12 Example curves of expected extreme impact pressures as a function of load area](image)

**Figure 12 Example curves of expected extreme impact pressures as a function of load area**

Depending on the related capacity curve for the cargo containment and the shape of the load curve, different loaded areas may turn out to be critical in the strength assessment.

### 8 Quality and efficiency considerations

#### 8.1 Methods to improve the long term sloshing impact load distribution results

As discussed in the above section a large number of different filling, heading and sea state combinations need to be tested to establish a representative long term load distribution. In addition, due to the statistical variability of the sloshing impact phenomenon, a large number of impact measurements are needed to get converged and reliable short term statistical distributions for the various combinations. In practice both requirements may be very difficult to meet, especially for lower sea states where the number of impacts per hour is low. The challenge is then to interpret the model test results, knowing that many of the short term distributions obtained from the testing is not fully representative. Large errors could be made if the test results are used without interpretation. The example below illustrates this matter by use of a Monte Carlo Simulation technique.

**Guidance note:**

Consider one sensor or sensor array for one short term sloshing condition in model scale and assume that the underlying physical phenomenon can be described by one set of Weibull parameters (see [2.7]), \( k = 0.76, \lambda = 0.04 \) and \( \theta = 0 \). Assume further that this short term condition is tested several times with different test duration and statistical distributions is obtained for each test for the sensor/sensor array we are considering. Assume then that we have 100 test results with a sample size of 50 impacts, 100 test results with a sample size of 200 impacts and 100 test results with a sample size of 2000 impacts.

This experiment has been simulated by a standard Monte Carlo simulation technique and the simulation results have been refitted with a Weibull function. Two different approaches have been followed. In the first approach all three Weibull parameters have been fitted to the data. Resulting exceedance probability distribution functions are shown in Figure 13. In the second approach the shape parameter has been forced into the original value of 0.76, and the other two parameters fit to the simulated data. The resulting exceedance probability distribution functions are shown in Figure 14. The reasoning for doing this is further explained later in this section.
Figure 13 Monte Carlo simulation results for sample size 50 (blue lines), 200 (red lines) and 2000 (green lines)

Figure 14 Monte Carlo simulation with “unique” shape results for sample size 50 (blue lines), 200 (red lines) and 2000 (green lines)

Guidance note:
The example above illustrates that the inherent variability in “flat tale” statistical distributions like the ones for sloshing impacts is large. If for instance a sample size of 50 is obtained from a test any of the blue lines in Figure 13 could be the outcome of the test with the same statistical properties of the input motion time series. Looking at the two limiting representations for instance at $10^{-3}$ probability level in Figure 13, the load varies from 0.3 to 1.2. It is obvious that the contribution to a long term load distribution from these two distributions will be very different.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

An approach to reduce the randomness in the statistical fits as illustrated in the example above shall try to utilize some physical knowledge about the problem as well as trends in the model test results. Consider that a number of tests have been carried out for different heading and sea state combinations for a given filling, and that a 3 – parameter Weibull distribution is fit to the measured data.
Figure 15 Shape parameter as a function of Sample size

Figure 15 illustrates Weibull shape parameters as a function of the number of recorded events during the test (each blue dot is one model test). The trend is very evident, and it looks like the shape parameter converges towards a single value (red line) with increasing sample size. This shape parameter value is then chosen as a representative for all the different heading and sea state combinations for this given filling, irrespective of what the test data for the different short term conditions indicate. We implicitly assume that the reason why all the shape parameters not fall into this single red line is lack of measured impacts in the different conditions – too small samples.

A new Weibull fit is made to all the considered data sets, but this time with the shape parameter fixed to the representative value found in the previous paragraph. This is similar to what is performed in the Mont Carlo simulation presented in the example above. A plot of the new scale parameters found for the different cases as a function of the event rate (number of impacts per hour) is illustrated in Figure 16.

Figure 16 Scale parameter as a function of event rate

As the shape parameter is assumed to be the same for all the cases, the scale parameter solely determines the anticipated severity of one given sloshing impact in a given condition. Higher scale parameter means higher expected impact pressures. Based on the results presented in Figure 16 (blue dots) it seems like impacts in sea states with very low event rate in some cases are more severe than impacts in sea states
with high event rate. Looking more closely at the data, one could identify cases where a low $H_s$ gives a much higher scale parameter than for a higher $H_s$ value for a given heading and $T_z$. This seems unphysical, and a possible explanation is lack of convergence in the cases with low event rate. A means to compensate for this unphysical effect shall assume a relationship between the event rate and the scale parameter, as e.g. the trend curve drawn in red in Figure 16. It is not obvious how to draw this trend curve, but experience indicates that the scatter of scale parameter is reduced with increasing sample size for the different cases. It could look like the scale parameter is converging towards some finite value with increasing event rate.

When using the above approach to improve the basis for the design load assessment, only the event rate is taken directly from the test data results. The event rate is converging much faster than the Weibull parameters, and only a limited number of test hours in normally needed to get a precise estimate. The event rate normally increases for motion periods close to liquid motion resonance and for increasing $H_s$ values.

As demonstrated in this section, there will be large differences between small sample realisations of statistical distributions expected to be relevant for sloshing impact loads, and direct fit to these realisations will not reliably reproduce the original distribution. Small samples are often encountered in sloshing assessments due to practical limitations, and it follows that the method outlined in this paragraph, or a similar method, will give more reliable predictions of short and long term statistical distributions of sloshing loads than a method based entirely on mathematical fits to short term experimental results.

The method is most efficiently utilised if it is taken into account in the planning and execution of the model test program as outlined below:

1) Select a set of key sea state and operational conditions that together can be considered governing for the statistical behaviour in other sea states/operational conditions.
2) Carry out long duration tests for these sea states to obtain reliable estimates of short term statistical behaviour.
3) Carry out shorter duration model tests for the large number of other sea states and operational conditions that require consideration to establish event rates.
4) Assign representative short term distributions from the governing long duration tests. Alternatively, fit distribution function to the test data by constraining e.g. the shape parameter of the distribution to the value obtained for the long duration test.

8.2 Quality control and resolution of the parameter space in the long term load distribution

Choices shall be made in how to discretize the parameter space (wave height, wave period, wave heading, tank filling, etc.) when deriving sloshing impact design loads by use of a long term assessment methodology, as described in [6].

To verify that the actual discretization is good enough it is recommended check the probability contribution at the design load level as a function of the different parameters. If for instance the probability contribution is highest for the largest wave heights and these are not the largest wave height encountered, it indicates that the test matrix has not been extended to high enough sea states. If on the other hand the contribution is largest for the smallest wave heights, it may be an indication that the threshold level is too high.

Another potential finding could be that just one wave heading dominates the design load probability, in which case it could indicate that a refinement of the heading probability distribution around this heading could be beneficial.

This kind of QA work on the derivation of sloshing impact design loads is an absolute necessity, as it has proven to be very easy to make mistakes in the process of deriving them. Small errors in the implementation of different input probabilities, Weibull parameters, or event rates give huge effects on the final results. Great care should hence be taken in all steps of the process.
SECTION 4 PUMP TOWER DESIGN LOADS

1 Motivation
The pump tower structure shall be designed to withstand both extreme loads (ULS) and repetitive loading (FLS). Different load effects will act simultaneously on the tower structure, and the structural response due to the combined load effect needs to be assessed.

It is recommended to use long term statistics to identify both the stress ranges for the fatigue loads and the extreme loading for the ultimate stress analysis.

The different loads to be considered, and a recommended procedure to determine the design loads, are outlined in the following.

2 Identification of relevant loads

The loads relevant for the dimensioning of the pump tower are due to the ship motion, the motion of the LNG in the cargo tanks, and the temperature of the cargo. Several load effects need to be considered and it is recommended to separate them in the following categories:

1) Sloshing load, gravity loads and inertia loads should be treated simultaneously in time domain. All three force components are dynamic loads that depends on the ship motions. The phasing between the loads and the distribution of the loads over the tower will be important.
2) Thermal loads on the tower structure due to the low temperature of the cargo.
3) Hull girder loads (for liquid dome area and for base support).
4) Internal tank pressure and external sea pressure (for base support).

A description of how to treat the different load categories is given in [3] to [6]. Guidelines for combination of load effects for ultimate strength and fatigue analyses are given in [7].

3 Sloshing, gravity and inertia loads

3.1 General
Sloshing loads on the pump tower structure occur due to motion of the liquid inside the cargo tanks. Inertia loads on the pump tower are due to the motion of the vessel. Translational tank accelerations due to the pitch and roll motion of the ship shall be accounted for. Weight of liquid inside the pipes should be included in the calculation of inertia loads.

Gravity loads includes the self-weight of the pump tower and the longitudinal and transverse components of the self-weight due to the roll and pitch angles experienced by the ship. When the gravity loads are calculated, the buoyancy of the pipes should be deducted.

For both inertia loads and gravity loads, the mass of additional elements (structural members and equipment) that are not included in the finite element model used for the stress response analysis should be included as lumped or distributed masses.

It is recommended to process the sloshing loads together with the inertia- and gravity loads to get the phasing between the load effects incorporated in a consistent way.

The sloshing loads shall be determined using model testing or numerical analyses by computational fluid dynamics (CFD). Irregular vessel motions shall be considered.

If the pump tower design is similar in all tanks of the vessel it may, the analysis/tests may be limited to the most highly loaded pump tower. The critical tank(s) should be determined by evaluation of the ship motions. This will usually be either the foremost or the aftmost tank, depending on the phasing between the motion modes. If tanks have different sizes, like for instance on a conventional LNG carrier, both tank no. 1 and tank no. 2 may need to be considered to determine which one is most critical with respect to the combined...
3.2 Filling levels and loading conditions

Depending on the filling level in the tanks and the loading condition the vessel is operating with, either inertia effects or sloshing loads may be dominating the structural response of the pump tower. The sloshing loads are very dependent on the filling level in the tank, while the inertia loads are more dependent on the loading condition of the vessel. It is generally not feasible to optimize the analysis for both load effects. If sloshing loads are dominating, a simplification of the loading condition distribution is recommended.

The sloshing loads should be considered for various filling levels according to the ship’s filling restrictions. For operation with a barred filling range, at least three filling levels need to be considered: one level representing the low filling range, and two levels representing the high filling range. This will normally be the lower filling limit in the case of ballast operation and upper filling limit and full tank (95%H) in the case of full load.

For unrestricted filling, at least four different filling levels should be considered, representing different characteristic tank filling ranges. A screening to identify the worst filling level within each range is recommended. Typically the following fillings should be considered:

- 15% B
- 50% H
- 70% H
- 95% H.

Here B represents the tank breadth and H the tank height.

A more refined filling discretization may be used to reduce the conservatism of the analysis.

Ship loading conditions should be selected in a conservative way, e.g. by using a ballast condition for all filling levels except for 95%H.

If it can be documented that inertia forces are the main contributor to the structural response for high filling levels, and sloshing loads are the main contributor for low filling levels, the FLS assessment may be carried out by considering the two effects separately for some fillings.

Sloshing loads may be disregarded for 95%H filling if it can be documented that the effect is small.

3.3 Load prediction

A sloshing test programme or calculation programme should be composed based on operational and environmental conditions for the considered vessel. Irregular motion time series based on the ship response spectra for the different conditions should be used to excite the tank.

It is normally beneficial to divide the testing or calculation programme into a screening phase and a design phase. The screening phase is used to identify the combinations of sea state, heading and tank filling that give the main contribution to the long term probability distribution. Longer duration tests or analyses should be carried out to achieve better statistical convergence for the cases identified to be most important.

The same load distribution should be used for both the FLS and the ULS analyses.

The load on the tower has a spatial variation that cannot easily be described statistically. From an analysis point of view it is therefore more convenient to describe the loads on the tower in terms of the forces and bending moments at the top and bottom supports. Forces/moments at other potential critical cross sections of the tower can be included as required. Long term statistical distributions should be determined for the selected response parameters, from which design values can be derived by specifying an appropriate return period.
Design load scenarios in terms of a spatial distribution over the tower can be derived by selecting relevant loading histories recorded during the testing/analysis program, and by scaling these to achieve the design values of the selected response parameters. The procedure is outlined in the following.

### 3.3.1 Establish time series for the sectional forces

A response model of the pump tower will shall be developed to calculate sectional forces and moments as function of the applied loads. The model should include the appropriate support stiffness from adjacent supporting structures.

An example of a calculation model is illustrated in Figure 1. \( f(\xi) \) denotes the combined load effect of sloshing, gravity and inertia in a given time instant. Guidelines for how to determine these loads are found in [3.3.2], [3.3.3] and [3.3.4]. In this case the tower is modelled as a beam described by the Young’s modulus \( E \), the cross-section area moment of inertia \( I \), and the mass per unit length \( m \). Tripod masts typically used in membrane LNG tanks may be better described by a truss/frame model.

For the support there will in general be reaction forces for the translation \( (F_A \text{ and } F_B) \) and for the rotations \( (M_A \text{ and } M_B) \). The rotational stiffnesses \( (K_A \text{ and } K_B) \) and the end support conditions shall be determined based on the design of the tower.

Time series for reaction forces and moments are determined by solving the equilibrium equations for the system in all degrees of freedom at all time instants:

\[
\sum F = 0
\]

\[
\sum M = 0
\]

Time series for other sectional force parameters can be found by solving similar equilibrium equations for the appropriate sub-regions of the model.

---

**Figure 1 Calculation model for pump tower**
3.3.2 Establish the distributed sloshing forces

Liquid sloshing loads distributed along the tower are determined by use of Morison’s equation. The sloshing loads on the pump tower segment are calculated using Morison’s equation, as described in DNV GL RP C205. Time series of the velocity- and acceleration are used to calculate the sloshing forces on all exposed structural members. The fluid velocities and accelerations are found from either model tests or CFD simulations as discussed in [3.4]. It is further assumed that the tower is fixed, and that the velocities and accelerations are taken relative to a ship fixed coordinate system. The force \( F \) acting on a part of a member is

\[
F = \rho_L \left( 1 + C_m \right) A a + \frac{1}{2} \rho_L C_D v |v| D
\]

where:

- \( \rho_L \) = liquid density
- \( C_m \) = added mass coefficient
- \( \left( 1 + C_m \right) \) = the mass coefficient
- \( A \) = cross sectional area of the member
- \( a \) = particle acceleration normal to the member axis
- \( C_D \) = drag coefficient
- \( v \) = liquid particle velocity normal to the member axis
- \( D \) = the diameter of the member projected onto a plane normal to the velocity direction

Values of \( C_D \) and \( C_m \) should be determined for each structural member according to recommendations given in DNV GL RP C205. For a cylindrical tube \( C_m \) shall be taken as 1.0. It should be noted that for oscillatory motions, \( C_D \) is a function of the \( K_C \)-number,

\[
K_C = U_M T / D
\]

where:

- \( D \) = diameter of pipe
- \( T \) = sloshing wave period
- \( U_M \) = maximum sloshing velocity

The drag coefficient may be taken from Figure 2, see DNV GL RP C205.
For cylinders that are close together, such as the emergency pipe and the filling pipe, group effects may be taken into account when determining the drag coefficients. If no documentation of the group effect is available, the drag coefficients for the individual cylinders should be used.

### 3.3.3 Calculation of inertia forces

Accelerations at any position of the pump tower can be calculated from the tank motion time history used for sloshing tests or sloshing analyses. Distributed inertia forces are calculated by multiplying the acceleration by the mass per unit length of tower considering also the weight of any liquid inside the pipes.

### 3.3.4 Calculation of the gravity force

Gravity will cause lateral dynamic forces on the pump tower due to rolling and pitching of the vessel. The lateral components of the gravity vector is calculated directly from the rotational angles for pitch and roll, and the distributed gravity force in time domain are calculated by multiplying with the relevant mass per unit length as for the inertia load.

### 3.3.5 Long term distribution of response

When time series for the reaction forces are calculated for different sea state, heading and filling combinations, short term statistical distributions can be established. The procedure described in Sec.3 [2] is recommended for statistical post-processing of the reaction force time histories to get the short term distributions.

The long term exceedance probability distribution of pump tower reaction forces is obtained by combination of short term distributions, and is in principle similar to the procedure outlined in Sec.3 [2.2]:

\[
Q(F) = \sum_{i=1}^{\text{filling states}} \sum_{k=1}^{\text{headings}} r_{ik} \cdot Q_{ik}(F) \cdot p_{ik}
\]

where:
\[ p_{ijk} = \text{the probability of occurrence of a given heading } i \text{ combined with a sea state } j \text{ and a filling } k \]
\[ r_{ijk} = \text{the ratio between the response crossing rates in a given sea state, heading and filling combination and the average crossing rate} \]
\[ = \frac{v_{ijk}}{v_0} \]
\[ v_0 = \sum_{i=1}^{\text{filling}} \sum_{j=1}^{\text{seastates}} \sum_{k=1}^{\text{headings}} p_{ijk} \cdot v_{ijk} \]
\[ v_{ijk} = \text{the response zero-crossing rate in heading } i, \text{ sea state } j \text{ and filling } k. \text{ This could be different from the wave zero crossing period} \]
\[ Q_{ijk}(F) = \text{the exceedance probability function for reaction forces derived from the experiment or CFD calculation in heading } i, \text{ sea state } j \text{ and filling } k \]

The FLS assessment shall be carried out using the same long-term load distribution as applied for the ULS assessment.

3.3.6 Design load conditions

To find the stress at different positions in the tower structure, a spatial distribution of the different load effects need to be applied on a structural response model. This can e.g. be a finite element model of the tower, As only the reaction forces at different probability levels are available, design load conditions needs to be derived based on the information available from the simulations/tests and the statistical values of the considered response parameters.

It is recommended that the design load conditions for the pump tower are defined based on snapshots of the load distribution obtained in simulations at time instants where large loads and reaction forces are recorded. The selected loads should be scaled so that the response at the position of interest, e.g. the reaction forces at the tower supports, matches the 25 year return period value.

It follows that several load cases may shall be considered to cover all relevant locations to be checked, e.g. considering different tank fillings.

3.4 Sloshing experiments and analysis

The sloshing loads on the pump tower resulting from the applied tank motion are a function of the liquid velocities and accelerations. The velocities, and if possible the accelerations, should be determined at several vertical locations along the midpoint of the tower, i.e. along the z-axis indicated in Figure 3.
The fluid forces on the pump tower structure may be assumed to be drag dominated, so that the fluid acceleration is of secondary importance relative to the fluid velocity. The velocities and accelerations may be determined by one of the following methods:

— experimental tests
— analysis with computational fluid dynamics (CFD).

The forces acting on the actual pump tower can then be calculated using Morison’s equation, as explained in [3.3.2].

### 3.4.1 Fluid velocities and accelerations based on experiments

The experiments should facilitate velocity estimates at least at ten points along the vertical axis.

A direct flow velocity measurement can be achieved by use of particle image velocimetry (PIV). The fluid is seeded with tiny reflective particles with density close to that of the fluid. The particles are assumed to follow the flow, and they are illuminated by e.g. a laser sheet. One or more cameras are used to capture two frames within a short instant. With two cameras in a stereoscopic setup, particle displacements along all three axes can be found. The velocity field in the imaged part of the laser sheet is typically found based on particle displacements from a cross-correlation analysis and the time separation between the images. The challenge of light reflection from the free surface can be partly overcome by use of fluorescent particles. PIV can also be used to estimate fluid accelerations.

An alternative to a direct velocity measurement method is given in the following example. The setup is illustrated in Figure 4. The bending moment in a pipe located in the tank is measured with strain gauges positioned to measure vertical strain. Force transducers should be fitted at top and bottom, in order to measure the total reaction forces on the pipe. By fitting for instance a spline function to the measured bending moments, the shear force is found as the derivative of the moment. By requiring force balance for a pipe segment, the difference in shear force at the end of the segment equals the external force on the segment. This external force contains gravity, inertia, buoyancy and hydrodynamic components. The inertia effect may be found by measuring the acceleration by an accelerometer. The gravity and buoyancy shall also be accounted for. Then the fluid force on the segment is obtained. For estimation of the fluid velocities and accelerations, Morison’s equation can be used. Again, drag may be assumed to dominate the fluid forces on the pump tower structure.
3.4.2 Fluid velocities and accelerations from CFD
If analyses are carried out using CFD, the following requirements should be fulfilled:

— The adequacy of the program used shall be documented, especially for low filling heights where breaking waves are expected. The adequacy is strongly related to the treatment of the free surface. To ensure that the software is capable of describing the physical sloshing phenomenon, the CFD software should be validated against model tests.

— The program should be capable of handling irregular tank motions and long simulation duration.

— Requirements related to modelling, mesh, and time step shall be given careful consideration. The requirements are important for numerical stability and an adequate discretisation of the problem which can provide a physical solution.

— For calculations with an Eulerian grid, the discretisation is recommended to be at least 40 elements in lengthwise direction, 40 elements along the tank breadth and 30 elements in the vertical direction.

Velocities and accelerations in the liquid at the location of the pump tower shall be determined. The velocities and accelerations may be calculated for a vertical axis located on the mean distance between the discharge pipes and the emergency pipe. The loads will vary in the vertical direction, but for each vertical position the velocities and accelerations may be taken as equal for all the pipes. The number of positions should be sufficient to describe the flow field along the pump tower length accurately. Typically more than 10 positions should be used.
4 Thermal loads

Thermal loads occur due to differences in thermal shrinkage between various parts of the pump tower and its supports due to temperature gradients or differences in material properties. The temperature effect is most important for the upper and lower supports of the pump tower.

The temperature distribution over the height of the pump tower shall in principle be determined for each filling level considered. For filling levels above 70% H, the temperature can be taken as constant and equal to -163ºC. For filling levels below 10% H, the temperature of the submerged part of the pump tower can be taken as -163ºC, while the temperature of the non-submerged part can be assumed to vary linearly from -163ºC at the liquid surface to -30ºC at the top of the pump tower.

The initial temperature of the steel can be taken as 20ºC. Thermal expansion coefficients relevant for the pump tower material shall be applied.

The temperature field shall be applied to the FE model used for the response analysis to determine the stress field in the structure resulting from the thermal shrinkage.

Low-cycle fatigue due to cyclic variation of the thermal stresses between the empty and loaded condition may need to be included in the fatigue life calculations.

Thermal stresses acting in the base support due to the thermal gradient from the level of the primary membrane to the level of the inner bottom plating shall also be calculated.

5 Hull girder loads

The global bending moment acting on the ship hull girder causes a stress field in the liquid dome area. The stress field resulting from the still water and wave bending moment shall be determined and applied to the FE model used for analysis of the liquid dome area. The maximum bending moment acting along the ship length shall be considered. The global stress shall include the stress concentration due to the liquid dome opening.

Similarly, the longitudinal stress acting in the bottom due to the global bending moment should be determined and applied to the FE model used for analysis of the base support.

Figure 5 Monitoring locations for velocities and accelerations
6 Internal tank pressure and external sea pressure

The double bottom stress resulting from the internal tank pressure and external sea pressure acting on the double bottom should be determined and applied to the FE model used for analysis of the base support.

7 Combination of load effects

Ultimate strength and fatigue assessments of the pump tower and the supports shall be based on the structural response to the combined action of the four load categories described in the previous sections.

For ultimate strength assessment the combination should be representative for the 25 year return period structural response. As sloshing is a resonance phenomenon, it is unlikely that the extreme sloshing response will be encountered on the same sea state as the extreme hull girder bending moment or internal tank pressure/external sea pressure. Further, even if the extreme responses are correlated, extreme values are not likely to coincide in time due to differences in phasing with the wave elevation.

Combined dynamic responses for ultimate strength assessment may therefore be calculated as follows:

\[ R_{\text{dyn}} = \sqrt{R_{\text{tower}}^2 + R_{\text{hull}}^2 + R_{\text{press}}^2} \]

where:

- \( R_{\text{dyn}} \) = a vector or scalar describing the combined 25 year return period combined response
- \( R_{\text{tower}} \) = a vector or scalar describing the 25 year return period response for the combined effects of sloshing, inertia, and gravity loads
- \( R_{\text{hull}} \) = a vector or scalar describing the 25 year return period response for hull girder bending
- \( R_{\text{press}} \) = a vector or scalar describing the 25 year return period response for internal tank pressure and external sea pressure

Combined values of derived values like the von Mises equivalent stress cannot be obtained by this formula, but shall be calculated from the combined component stresses.

Thermal load effects are considered as static, and can be directly combined with the effect of the other loads either by including the thermal effect in the finite element response analyses or by summing the combined dynamic response and the thermal response:

\[ R_{\text{total}} = R_{\text{dyn}} + R_{\text{thermal}} \]

where:

- \( R_{\text{dyn}} \) = a vector or scalar describing the combined 25 year return period response to the combined dynamic loads acting on the tower
- \( R_{\text{thermal}} \) = a vector or scalar describing the thermal structural response

The long-term FLS stress distribution may be described by a two parameter Weibull distribution, characterised by a reference stress range and the Weibull slope parameter. The reference stress range may
be taken as two times the 25 year return period dynamic stress amplitude defined above. The number of load
cycles in the spectrum should be taken as the maximum of the expected cycles for the four considered load
categories.

Other response combination methods can be accepted by the Society if they can be demonstrated to be
conservative.

For ULS assessment and FLS assessment, both tank no. 1 and tank no. 4 should be considered, in order to
determine which one is most critical with respect to the combined effect of sloshing loads and inertia loads.
SECTION 5 STRUCTURAL RESPONSE ANALYSIS OF CONTAINMENT SYSTEMS

1 General
The strength assessment of structures exposed to transient dynamic loads such as those generated by sloshing impacts generally requires the assessment of the dynamic response of the structure. A simplified approach including the use of a dynamic load factor along with quasi-static response analyses should be used to assess the dynamic responses of the membrane type insulation systems. Sloshing impacts may cause deformations of primary membranes of membrane type containment systems, but are not likely to cause significant damage and loss of tightness of membranes that are properly supported by the load carrying insulation of the containment system. The primary purpose of the sloshing impact assessment of the cargo containment system is therefore to ensure that the load carrying insulation maintains its structural integrity, and hence ensure that the membranes remain supported. This chapter is therefore limited to description of structural response analyses of the insulation system.

Structural response analyses of primary membranes may be required to ensure that membrane deformations remain within specified limits. In this case designers are encouraged to discuss and agree on analysis methodology with the Society prior to commencement of the analyses. The requirements and guidelines for the quasi-static response analyses are described in [2] for the Mark III system and [3] for the NO96 system. The dynamic factor is defined in [6]. Analysis should be carried out for transverse corner/longitudinal knuckle insulation structures and for the flat wall insulation structure adjacent to corners and knuckles. Note that the response analysis methodology specified for the Mark III system in [2] is also applicable to the CS 1 system, but that the comparative strength assessment methodology cannot be applied to this system. Alternative analyses methods may be accepted by the Society if the designer can demonstrate the relevance of the proposed method. It is recommended that the designer raise such questions with the Society at the startup of a project, and that agreement is reached on what documentation will be needed to get the Society’s acceptance for the method.

2 Mark III system

2.1 General
A schematic representation of the quasi-static load-displacement response of a Mark III insulation panel is shown in Figure 1. The elastic response limit of the structure is reached at point 1 on the curve, where the elastic capacity of the reinforced polyurethane foam (RPUF) is reached at the mastic support locations as illustrated in the upper right hand sketch on Figure 1. This is not considered to represent the capacity of the structure. Loading past Point 1 on the response curve requires that the incremental load is transferred to the mastic supports as shear force in the plate, and given sufficient strength of the bottom plywood plate, the stress in the foam will reach a distribution as shown in the lower right hand sketch of Figure 1. Loading beyond this limit will lead to excessive deformation of the panel, and it consequently represents an upper bound on the capacity of the insulation panel.
Figure 1 Schematic illustration of the load-displacement response and stress states in the polyurethane foam at the foam/plywood interface of a Mark III insulation panel

The actual capacity may be governed either by excessive deformation of the insulation panel at the load level identified at Point 2, or shear or bending failure of the bottom plywood plate at an intermediate load level between point 1 and point 2. It is clear that an assessment of the bottom plate capacity requires a determination of its structural response in the non-linear response range of the insulation panel. The non-linear response assessment may be carried out using linear elastic finite element response analysis to determine the elastic stress and deformation state at point 1 ([2.2]), in combination with a simplified analytical model based on linear elastic beam theory to assess the incremental shear force and bending moment in the bottom plywood plate ([2.3] and [2.4]).

The primary membrane is supported by the insulation system but is not considered to significantly influence the response of the insulation and vice versa. If relevant the structural response of the membrane can be determined using a separate model containing only the membrane.

2.2 Finite element analyses

The finite element response analyses should be carried out using a finite element model covering a sufficiently large portion of the insulation panel to make sure that the structural response is well confined within the interior of the model. The model need not reflect the full length and width of the insulation panel. Symmetry in geometry and/or structural response of the insulation panel may be exploited. For the corner/knuckle panels it is sufficient to model only one side of the corner/knuckle as illustrated in Figure 2. The model should include a sufficiently large portion of the adjacent flat insulation panel to avoid significant effects of boundary conditions for the considered load areas.

Figure 2 Required model extent for corner panel
An example finite element model comprising half the width of a flat wall Mark III panel (495mm) and approximately a quarter length of a panel is shown in Figure 3.

Figure 3 Example finite element model of a Mark III panel

The model should be built up using a mix of continuum, shell, and membrane elements, as follows:

— the polyurethane foam, the mastic strips, and the hardwood key of the corner/knuckle panels should be modelled using eight node 3D continuum elements. Elements using a reduced integration scheme may be used.

— the upper and lower plywood plates as well as the secondary barrier level plywood plate of the corner/knuckle panel can be modelled using four-node shell elements.

— the primary corrugated steel membrane may be modelled using membrane elements with small in-plane stiffness, but with a representative mass per unit area to include its inertia effects on the dynamics of the panel.

— the secondary triplex membrane may be disregarded.

The shell and the continuum components of the model should be constrained to enforce kinematic continuity between the components, taking into account the thickness of the plates.

The membrane elements representing the primary membrane should be constrained to follow the deformation of the 12mm top plywood plate.

The finite element mesh requirements are given in the following. Mesh densities deviating from these requirements will be accepted provided that the adequacy of the mesh can be demonstrated with a mesh convergence test.

In regions of the model where the structural response will be extracted:

— 3 elements should be used across the width of the mastic supports.

— at least 7 elements should be used across the free span of the bottom plywood plate.

— the element edge length in the direction of the mastic supports and in the through thickness direction of the panel should be determined so that the elements are practically square.

In other parts of the model:

— 2 elements should be used across the width of the mastic supports.

— at least 5 elements should be used across the free span of the bottom plywood plate.

— the element edge length in the direction of the mastic supports and in the through thickness direction of the panel can be rectangular, with a maximum aspect ratio of 3.0.

In the case the three planes of symmetry in geometry and response of the panel is exploited, the boundary conditions for the panel should be taken as follows:

— restrained translations of the bottom nodes of the mastic support.
— symmetry conditions on secondary foam, bottom plywood plate and centre mastic at panel symmetry planes (see Figure 4).
— symmetry conditions on primary foam and top plywood plate at cross-panel boundaries (see Figure 4).

The far end plane of the model may be kept free. All patch loads should be applied away from this boundary, and even in the case of uniform load the results are taken at the opposite end of the model.

**Figure 4 Symmetry planes for the example Mark III insulation panel model**

The plywood and the reinforced polyurethane foam materials may be modelled as homogeneous linear orthotropic elastic materials. In this case the bending stiffness moduli should be used to represent the plywood stiffness. Alternatively the plywood may be modelled as a 0-90 laminate of layers of orthotropic elastic materials. The mastic may be modelled as a homogeneous linear elastic material. Material properties are given in [4], and should be selected based on the material directions specified in Figure 5.

**Figure 5 Definition of RPUF and plywood material directions relative to the Mark III insulation panel**
2.3 Average through thickness stress in foam

The average through thickness stress may for a patch load case be interpreted as the equivalent uniform stress applied to the surface of the panel. It follows that for a uniform load situation the stress is directly defined by the applied load, \( p \):

\[
\bar{\sigma} = p
\]

The average through thickness stress for the secondary foam insulation at the interface with the bottom plywood plate should be assessed as follows:

1) For a reference patch load, establish the elastic through thickness stress on top of the mastic support located as far as possible directly below the centre of the loaded area (see Figure 6). If the load patch covers more than one mastic support, the average of the stress at all the mastic supports may be used. Let this be \( \sigma_0^p \).

2) Establish the elastic through thickness stress on top of the same mastic support(s) for the case of a uniform reference load with the same magnitude as above (see Figure 6). Let this be \( \sigma_0^u \).

3) Calculate the ratio

\[
r = \frac{\sigma_0^p}{\sigma_0^u}
\]

4) Calculate the average through thickness stress as \( \bar{\sigma} = r \cdot p \).
Section 5

Figure 6 Definition of the stress quantities used to define the average through thickness stress

The procedure outlined above implies that the average compressive stress is defined based on the linear elastic through thickness load dispersion in the model. This is conservative, since inelastic deformations below the loaded region of the panel will result in a change in the relative stiffness between this area and the adjacent areas, and thus also increase the load distribution to adjacent areas. Another source of conservatism in this calculation is the selection of the peak elastic stress in the patch load case to enter the calculation of the ratio \( r \).

2.4 Shear force and bending moment in bottom plate

In the case of simple load situations where the load on the panel surface extends over an area spanning several mastic support spacings, the structural system for assessment of the incremental shear force at the supports is statically determinate. The additional force applied to the system beyond the elastic response limit (Point 1 in Figure 1) cannot be transferred through the foam into the supports because the elastic limit of the foam has been reached at the mastic support locations. The entire force shall hence be transferred to the supports as shear force in the plywood plate.
Figure 7 Illustration of the concept of combining linear finite element response analyses for the linear elastic response range and simplified analytical models for the elasto-plastic load range of the insulation panel.

\[ \lambda_{el,1} = \frac{\sigma^e}{\sigma^e_{foam}} \]
\[ \lambda_{el,2} = \frac{M_c}{M^e_{1}} \]
\[ \lambda_{el,3} = \frac{Q_c}{Q^e_{1}} \]
\[ \lambda_{el} = \min(\lambda_{el,i}) \]
An assessment of the bending moment in the plate is more complicated, and requires that an assumption is made with respect to the distribution of incremental vertical stress exerted by the foam on the plywood plate. The incremental bending moment will therefore be a more uncertain quantity than the incremental shear force.

For patch load situations the assessment of the shear force will be approximate, since the procedure will not account for the redistribution of forces that will take place in the structure as soon as the stiffness locally beneath the load decrease as a result of inelastic material response.

The approach outlined above is illustrated in Figure 7, where the selected parabolic additional foam stress function $q_{pl}(x)$ is also specified. The amplitude $q_{pl}^0$ is a function of the magnitude of the incremental load, as outlined in more detail in the following.

The response assessment requires the execution of the following analysis steps:

1) Linear elastic structural response analysis using a reference load amplitude, $P_{ref}$.
2) Extract the critical response parameters from the model. This involves (see Figure 7):
   a) the vertical stress in the reinforced polyurethane foam at the centre of the most highly loaded mastic support, $\sigma_{ foam}^{ref}$.
   b) the bending moment per unit plate width of the bottom plywood plate at mastic supports and mid span, $M_1^{ref}$, $M_2^{ref}$, $M_3^{ref}$.
   c) the shear force per unit plate width of the bottom plywood plate at mastic supports, $Q_1^{ref}$, $Q_2^{ref}$.
   d) the rotations of the bottom plywood plate at the mastic supports, $\theta_1^{ref}$, $\theta_2^{ref}$.

The structural response should be output at locations of maximum response centred below the applied load. When the bending moment and the shear force is obtained at the integration points of the shell elements adjacent to the mastic supports the corrections shown in Figure 8 and Figure 9 should be made to make the results representative for the plate cross-section adjacent to the mastic support. $L_{int}$ is the distance from the integration point of the element to the edge of the mastic.

![Figure 8 Correction of FE calculated plate shear force to obtain a better estimate of the plate end value](image-url)
Figure 9 Correction of FE calculated plate bending moment to obtain a better estimate of the plate end value

3) Determine the minimum elastic pressure capacity of the insulation panel (Point 1 in Figure 7) in terms of a proportionality factor $\lambda_{el}$ on the applied reference load. This is the minimum of the load proportionality factors $\lambda_{el,i}$ satisfying the following equations:

a)
$$\lambda_{el,1} \cdot \sigma_{f,\text{foam}} = \sigma_F$$

b)
$$\lambda_{el,2} \cdot M_{i,\text{foam}} = M_c$$

c)
$$\lambda_{el,3} \cdot Q_{i,\text{foam}} = Q_c$$

where $\sigma_F$ is the crushing strength of the polyurethane foam in the through thickness direction of the insulation panel specified in Sec.6 Table 7, and $M_c$ and $Q_c$ is the bending moment capacity and the shear force capacity of the bottom plywood plate in the relevant cross-section direction, specified in Sec.6 Table 6.

If $\min(\lambda_{el,i}) = \lambda_{el,1}$ the next step is to calculate the incremental shear force. If not, the capacity is governed by plywood plate failure in the linear elastic response range of the structure, and can be estimated using either the bending moment or shear force strength criterion. The remaining steps are presented under the assumption that $\min(\lambda_{el,i}) = \lambda_{el,1}$. 
The incremental shear force and bending moment are calculated under the assumption that an incremental pressure \((\lambda - \lambda_{el})p_{\text{ref}}\) is applied to the panel, where \(\lambda\) is the load proportionality factor describing the ratio between the design sloshing impact pressure including dynamic effects \((p \cdot D)\) and the reference pressure used in the finite element analyses.

4) As mentioned earlier in this section, the incremental load shall be transferred to the mastic support as shear force in the plywood plate. It is assumed that the shear force is equally distributed to each end of the plate, and the incremental shear force at the end of the plate is therefore:

\[
Q_{pl} = \frac{1}{2} (\lambda - \lambda_{el}) p_{\text{ref}} L_{\text{ref}}
\]

5) The total plate end shear force per unit plate width in the non-linear response range can be estimated as:

\[
Q_1 = \lambda_{el} Q_1^{\text{ref}} + \frac{1}{2} (\lambda - \lambda_{el}) p_{\text{ref}} L_{\text{ref}}
\]

6) For the assessment of the incremental bending moment per unit plate width, the incremental load specified above is assumed to be distributed according to the parabolic function shown in Figure 7, as follows:

\[
q_{pl}(x) = 6(\lambda - \lambda_{el}) p_{\text{ref}} L_{\text{ref}} L^2 (x^2 - Lx)
\]

The bending moments for the case of \((\lambda - \lambda_{el})=1\) at end 1, mid span and end 2 (see Figure 7) are:

\[
M_{pl,1}^0 = \frac{p_{\text{ref}} L_{\text{ref}} L^2 k_2 (6EI + k_1 L)}{90(3EI)^2 + EIL(k_1 + k_2) + k_1 k_2 L^2}
\]

\[
M_{pl,2}^0 = \frac{p_{\text{ref}} L_{\text{ref}} L^2 k_1 (6EI + k_2 L)}{90(3EI)^2 + EIL(k_1 + k_2) + k_1 k_2 L^2}
\]

\[
M_{pl,3}^0 = \frac{p_{\text{ref}} L_{\text{ref}} L^2 k_2 (6EI + k_1 L) - 5p_{\text{ref}} L_{\text{ref}} L}{32EI}
\]

The elastic modulus \(E\) should represent the bending modulus of the bottom plate in the direction transverse to the mastic supports, and the area moment of inertia \(I\) should be calculated for a unit width plate strip.
The support bending stiffness $k_1$ and $k_2$ are calculated from the plate end bending moment and rotation obtained from the linear finite element analyses, as follows:

$$k_1 = \frac{M_1^{\text{ref}}}{\theta_1^{\text{ref}}}, \quad k_2 = \frac{M_2^{\text{ref}}}{\theta_2^{\text{ref}}}$$

The sign conventions for the bending moment are indicated in the upper left hand sketch in Figure 7.

7) The total plate end and mid-point bending moments per unit plate width in the non-linear response range can be estimated as:

$$M_i = \lambda \phi_i M_i^{\text{ref}} + (\lambda - \lambda \phi_i) M_{\text{pl},i}^{\theta}, \quad i=1, 2, 3$$

2.5 Shear force and bending moment in the secondary barrier plywood plate in corners/knuckles

The plate shear force and bending moments should be output at high stress locations at the edges of the hardwood key (transition between the hardwood key and the softer primary foam).

When the bending moment and the shear force is obtained at the integration points of the shell elements adjacent to the hardwood key, similar corrections as specified for the bottom plate should be made (Figure 8 and Figure 9) to make the results representative for the plate cross-section adjacent to the hardwood key. $\sigma_{\text{Foam}}$ should in this case be taken as the differential stress between the foam on the primary and secondary insulation side of the plate.

3 NO96 system

3.1 Finite element analyses

The primary and the secondary insulation boxes should be modelled as separate entities.

Both the primary box and the secondary box may be modelled as continuous structures, meaning that full translational and rotational continuity may be assumed between the bulkheads and the top and bottom plates. The discontinuous stapled connection between the plates is hence disregarded.

In the case of the reinforced primary box with double cover plates, the upper and lower plates should be modelled as separate entities. The plates should be connected using elastic springs at the positions of the staples, as indicated in Figure 10, and the interaction between the plates should otherwise be treated as contact in the normal direction and free sliding in the tangential direction. The stiffness of the connecting springs should be taken as 10 000 N/mm.

The cover plates should be modelled as three individual parts in order to account for the discontinuity of the plating at the invar tongue slits.

Analysis of the corner boxes should include the adjacent flat wall boxes and the plywood plate that covers the gap between the corner structure and the flat wall structure.
Figure 10 Location of staples in the FE model

The interaction between the primary and the secondary box should be modelled as a contact constraint. In addition the primary box should be connected to the secondary box in each corner using elastic springs to prevent undesired relative displacement of the boxes relative to each other. This is a simplification of the stud bolt arrangement that secures both boxes to the hull structure. The stiffness of the connection springs should be taken as 10 000 N/mm.

The resin ropes may be disregarded, and the support boundary conditions may be directly imposed at the intersection lines between the bulkhead plates and the bottom plate of the secondary box.

The primary and secondary invar membranes may be disregarded.

Figure 11 Cut through the 3D model
Figure 12 Modelling of staples in the FE model

The NO96 insulation boxes should be modelled using 4 node shell elements. The mesh requirements are as follows:

Cover plates:
— at least 7 elements between the vertical bulkheads of the primary box
— the element aspect ratio for the cover plate should not exceed 2.0, and should be determined based on the bulkhead plate mesh requirements given below

Primary box bulkheads:
— the element mesh should be congruent with the mesh in the cover plates
— at least 7 elements should be used across the height of the bulkhead plates
— the elements in the bulkheads should as far as possible be square.

Secondary box:
— at least 9 elements should be used across the height of the secondary bulkheads
— the elements in the bulkhead plates should as far as possible be square.

The following displacement restraints shall be imposed on the model:

— the secondary box shall be fixed in the vertical direction at the intersection lines between the bulkhead plates and the bottom plate of the secondary box. In case the resin ropes are included in the model the same condition should be enforced at the bottom of the resin rope elements
— a minimum restraint against in-plane translation and rigid body rotation of the model should be imposed at two of the bottom corners of the secondary box.

The plywood material may either be modelled as a homogeneous linear orthotropic elastic material, or as a 0-90 laminate of layers of orthotropic elastic materials. In the former case the cover plate stiffness should be represented by the bending stiffness moduli of the plate, whereas the membrane stiffness moduli should be used for all other components. Material properties are given in [4], and should be selected based on the material directions specified in Figure 13 unless other directions are specified on the drawings.
3.2 Shear force and bending moment in the cover plates

The output parameters should be the bending moment and shear force per unit width of the plate cross-section at locations of maximum response centred below the applied load. In case the bending moment and the shear force are obtained at the integration points of the shell elements adjacent to the mastic supports, the corrections shown in Figure 14 and Figure 15 should be made to make the results representative for the plate cross-section adjacent to the primary box bulkheads. $L_{int}$ is the distance from the integration point of the element to the edge of the mastic. Note that the corrections are specified under the assumptions of double cover plates, and that the shear force and bending moment corrections are relevant for each individual plate.

No correction of the bending moment is required if the maximum moment occur at mid span of the unsupported plate field, and the element mesh contains an element row centred on the plate field.
3.3 Reference stress for buckling strength assessment

The in-plane reference stress to be used in the buckling strength assessment of the vertical bulkheads of the primary and the secondary box should be selected at mid height of the bulkheads and in line with the intersection between the bulkheads in the primary and the secondary box as illustrated in Figure 16 for the primary box bulkhead and in Figure 17 for the secondary box bulkhead.
If the element mesh contains an even number of elements across the height of the bulkhead and thus does not allow for selection of an element at the centre of the bulkhead, the element farthest away from the stress concentration at the primary and secondary bulkhead intersections should be selected. This implies the element just above the centreline should be selected for the primary bulkhead and the element just below the centreline should be selected for the secondary bulkheads.

The stress should be selected from the most highly loaded bulkhead for the considered load condition.
3.4 Reference stress for bulkhead intersection crushing assessment

The in-plane reference stress to be used in the strength assessment of the bottom plate of the primary box, the cover plate of the secondary box, and the bulkhead plate edges should be obtained at mid height of the primary box bulkhead and in line with the intersection between the bulkheads in the primary and the secondary box as illustrated in Figure 17. This is the same stress as should be used in the buckling strength assessment of the primary box bulkheads.

3.5 Reference stress for bottom plate crushing assessment

The reference stress for the crushing strength assessment of the bottom plate of the secondary box at the intersection with the vertical bulkheads should be selected at the bottom element row of the secondary bulkhead, as illustrated in Figure 18. This should be the vertical membrane stress component in the bulkhead.

![Figure 18 Selection of vertical in-plane membrane stress for crushing strength assessment of the bottom plate of the secondary box](image-url)
4 Material stiffness parameters

4.1 Plywood

The material stiffness properties for the plywood laminate and the individual wood layers of the laminate are given in Table 1 and Table 2, respectively. Subscript m denotes the in-plane (membrane) stiffness properties of the plates, and subscript b denotes the bending stiffness properties. The properties are given in terms of the material coordinate systems shown in Figure 19.

![Figure 19 Material coordinate systems for the plywood laminate (left) and the individual layers of wood](image)

The relationship between the individual layer and the integrated laminate properties are:

\[ E_{\alpha\beta} = \frac{1}{l} \int_{-l/2}^{l/2} E_{ij}(x) dx_j , \quad i=1, 2 \]

\[ G_{\alpha\beta} = \frac{1}{l} \int_{-l/2}^{l/2} G_{ij}(x) dx_j , \quad i=1, 2 \quad j=2, 3 \]

\[ E_{ij} = \frac{12}{l^2} \int_{-l/2}^{l/2} E_{ij}(x) \cdot x_i \cdot x_j dx \quad i=1, 2 \]

The local \( E_i \) and \( G_{ij} \) of the individual layers should be evaluated in the material coordinate system of the laminate.
Table 1 Integrated orthotropic material stiffness properties for the plywood plates

<table>
<thead>
<tr>
<th>Parameter</th>
<th>20ºC</th>
<th>-163ºC</th>
</tr>
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<tbody>
<tr>
<td>$E_{m,1}$ [MPa]</td>
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<td>13 200</td>
</tr>
<tr>
<td>$E_{m,2}$ [MPa]</td>
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<td>11 200</td>
</tr>
<tr>
<td>$E_{m,3}$ [MPa]</td>
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<td>1800</td>
</tr>
<tr>
<td>$G_{m,12}$ [MPa]</td>
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</tr>
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<td>$E_{b,1}$ [MPa]</td>
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<td></td>
</tr>
<tr>
<td>9mm [MPa]</td>
<td>10 950</td>
<td>15 350</td>
</tr>
<tr>
<td>12mm [MPa]</td>
<td>10 450</td>
<td>14 650</td>
</tr>
<tr>
<td>Density [ton/mm³]</td>
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<td>6.8e-10</td>
</tr>
</tbody>
</table>

Table 2 Material properties for the individual wood layers

<table>
<thead>
<tr>
<th>Parameter</th>
<th>20ºC</th>
<th>-163ºC</th>
</tr>
</thead>
<tbody>
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<td>$E_1$ [MPa]</td>
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<tr>
<td>$E_2$ [MPa]</td>
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<td>4950</td>
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</tr>
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<td>340</td>
</tr>
<tr>
<td>$v_{12}$ [-]</td>
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<td>$v_{13}$ [-]</td>
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<td>0.1</td>
</tr>
<tr>
<td>$v_{23}$ [-]</td>
<td>0.1</td>
<td>0.1</td>
</tr>
<tr>
<td>Density [ton/mm³]</td>
<td>6.8e-10</td>
<td>6.8e-10</td>
</tr>
</tbody>
</table>

4.2 Reinforced polyurethane foam

The orthotropic linear elastic material properties for the reinforced polyurethane foam are summarised in Table 3.
### Table 3 Material stiffness properties for the reinforced polyurethane foam

<table>
<thead>
<tr>
<th>Parameter</th>
<th>20ºC</th>
<th>-163ºC</th>
</tr>
</thead>
<tbody>
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<td>$E_1$ [MPa]</td>
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<td>$E_2$ [MPa]</td>
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<td>$E_3$ [MPa]</td>
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<td>$G_{12}$ [MPa]</td>
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<td>11</td>
</tr>
<tr>
<td>$G_{13}$ [MPa]</td>
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<td>11</td>
</tr>
<tr>
<td>$G_{23}$ [MPa]</td>
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<td>11</td>
</tr>
<tr>
<td>$\nu_{12}$ [-]</td>
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<td>0.4</td>
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<tr>
<td>$\nu_{13}$ [-]</td>
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</tr>
<tr>
<td>$\nu_{23}$ [-]</td>
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<td>0.2</td>
</tr>
<tr>
<td>Density [ton/mm³]</td>
<td>1.25e-10</td>
<td>1.25e-10</td>
</tr>
</tbody>
</table>

### 5 Mastic

The material stiffness properties for mastic is summarised in Table 4.

### Table 4 Material stiffness properties for mastic

<table>
<thead>
<tr>
<th>Parameter</th>
<th>20ºC</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E$ [MPa]</td>
<td>2900</td>
</tr>
<tr>
<td>$\nu$ [-]</td>
<td>0.3</td>
</tr>
<tr>
<td>Density [ton/mm³]</td>
<td>1.6E-9</td>
</tr>
</tbody>
</table>

### 6 Simplified assessment of dynamic response

#### 6.1 General

The dynamic response is determined from the quasi-static response described above through the application of a dynamic amplification factor, denoted DAF. The factor is a function of the ratio between the pressure pulse rise time $t_r$ defined in Sec.3 Figure 5 in Sec.3 [2.8] and the natural period $T_n$ of the insulation system.

Natural periods to be used in the selection of the dynamic load factor are specified in [6.2] and [6.3] for the Mark III and the NO96 systems, respectively. Note that the natural period in general varies with the loaded area. However, the consequences of the expected variations are considered to be within the uncertainty limits inherent in the present approach to determine the dynamic load factor.

A piece-wise linear curve is used because of the uncertainties involved in determining the rise time of the pressure pulse. Due to this uncertainty, the design curve is specified to envelope the maximum-points on the actual calculated DAF-curves.

The design DAF curve is illustrated in Figure 20, and is defined as follows:

$$\frac{t_r}{T_n} < 0.5; \text{DAF} = f_1$$
\[
\frac{T_r}{T_n} \in (0.5, 1.0): \text{DAF} = f_1 - 2 \cdot \left( \frac{T_r}{T_n} - 0.5 \right) (f_1 - f_2)
\]

\[
\frac{T_r}{T_n} > 1.0: \text{DAF} = \max(1.0, f_2 - 0.33 \cdot \left( \frac{T_r}{T_n} - 1.0 \right) (f_2 - 1.0))
\]

![Figure 20 Illustration of the curve to be used in the simplified assessment of the dynamic amplification factor](image)

**Figure 20** Illustration of the curve to be used in the simplified assessment of the dynamic amplification factor

The factors \(f_1\) and \(f_2\) relevant for the Mark III and the NO96 systems are given below.
6.2 Mark III system

The factors $f_1$ and $f_2$ are given as a function of the vertical position of the considered response variable:
At the bottom plywood plate (furthest away from the load) the following values apply:
\[ f_1 = 1.65 \]
\[ f_2 = 1.25 \]
At the upper plywood plate (closest to the load) the following values apply:
\[ f_1 = 1.25 \]
\[ f_2 = 1.1 \]
Linear interpolation may be applied when response variables in between the top and bottom are considered.

For moderate design modifications of the system including changes of the mastic support spacing and the bottom plate thickness the natural period to be used in the assessment of the dynamic load factor can be taken as 2.0 milliseconds.

More radical design modifications require that the natural period is established by dynamic finite element analyses.

6.3 NO96 system

The factors $f_1$ and $f_2$ are functions of the vertical position of the considered response variable, as given in the following.
At the mid part of the secondary box bulkhead:
\[ f_1 = 1.6 \]
\[ f_2 = 1.2 \]
At the mid part of the primary box bulkhead:
\[ f_1 = 1.2 \]
\[ f_2 = 1.05 \]
At the cover plywood plates (closest to the load):
\[ f_1 = 1.4 \]
\[ f_2 = 1.1 \]
Linear interpolation may be applied when response variables in between the top and bottom are considered.

For moderate design modifications of the system including changes of the internal bulkhead plate thickness the natural period to be used in the assessment of the dynamic load factor can be taken as 1.3 milliseconds.

More radical design modifications require that the natural period is established by dynamic finite element analyses.
SECTION 6 ULTIMATE STRENGTH OF CONTAINMENT SYSTEMS

1 General
Acceptance criteria for the ultimate strength assessment of the Mark III and the NO96 insulation systems are given in [3] and [4], respectively.
Note that the acceptance criteria specified for the Mark III system in [3] area also applicable to the CS 1 system, but that the comparative strength assessment methodology cannot be applied to this system.
[2] gives guidance on how to use the acceptance criteria to calculate the load capacity of the insulation system as needed to determine the load level for the reference case in the comparative assessment (see Sec.2 [4]).

2 Capacity assessment
The load capacity of the insulation systems in terms of peak impact pressure for a given impact area size should be calculated based on the strength acceptance criteria given in this section, as follows:
1) Calculate the design capacity of each relevant response parameter including dynamic effects by solving the limit state equations for the maximum allowable dynamic response parameter.
2) Invert the quasi-static response analysis to determine the surface pressures corresponding to the critical response. This will be the maximum effective dynamic pressure the insulation system can sustain.
3) Calculate the peak pressure capacity for each considered impact area by dividing the effective dynamic pressure capacity by the relevant dynamic factor.

3 Mark III system
3.1 Impact areas
The strength of the Mark III insulation system should be checked for the impact load foot prints identified in Figure 1 and Figure 2, and defined in Table 1.

Table 1 Definition of load foot prints for the flat wall panels

<table>
<thead>
<tr>
<th>Foot Print</th>
<th>Width (mm)</th>
<th>Length (mm)</th>
<th>Area (mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FP 1</td>
<td>150</td>
<td>150</td>
<td>22 500</td>
</tr>
<tr>
<td>FP 2</td>
<td>400</td>
<td>150</td>
<td>60 000</td>
</tr>
<tr>
<td>FP 3</td>
<td>800</td>
<td>150</td>
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<td>FP 4</td>
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<tr>
<td>FP 10</td>
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<td>855.5</td>
<td>43 6305</td>
</tr>
</tbody>
</table>
If the statistical processing of the load data does not distinguish between foot print orientations 2 and 4, 3 and 5, and 7 and 8, it is sufficient to consider only one from each pair for the flat wall panels.

**Table 2 Definition of load foot prints for the corner panels**

<table>
<thead>
<tr>
<th>Foot Print</th>
<th>Width (mm)</th>
<th>Length (mm)</th>
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</tr>
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<td>FP 4</td>
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<td>680</td>
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<td>142 800</td>
</tr>
<tr>
<td>FP 6</td>
<td>420</td>
<td>680</td>
<td>285 600</td>
</tr>
<tr>
<td>FP 7</td>
<td>&gt;600</td>
<td>&gt;680</td>
<td>&gt;40 8000</td>
</tr>
</tbody>
</table>
Figure 1 Load footprints for strength assessment of the flat wall Mark III insulation panel
Figure 2 Load footprints for strength assessment of the corner/knuckle Mark III insulation panels
3.2 Failure modes for insulation panel

The ultimate strength assessment of the Mark III insulation panel should consider the following set of critical failure modes (see Figure 3 and Figure 4):

Figure 3 Visual identification of locations of critical failure modes for the flat wall Mark III insulation panel

1) crushing of primary foam.
2) crushing of secondary foam at plywood interface.
3) shear failure of plywood plate at support.
4) bending failure of plywood plate.
5) shear failure of the secondary barrier level plywood plate (corner/knuckle) at the edge of the hardwood key.
6) bending failure of the secondary barrier level plywood plate (corner/knuckle) at the edge of the hardwood key.

Figure 4 Visual identification of locations of critical failure modes for the corner/knuckle Mark III insulation panel
3.3 Crushing of primary foam
The strength criterion for this failure mode is a stress control criterion for the average through thickness compressive stress in the foam on the form:

\[ p \cdot \text{DAF} \leq \sigma_F \]

where:
- \( p \) = the peak impact pressure acting on the panel
- \( \text{DAF} \) = the dynamic amplification factor as defined in Sec.5 [6.1]
- \( \sigma_F \) = the crushing strength of the reinforced polyurethane for the relevant temperature and load rate conditions, as defined in [6].

3.4 Crushing of secondary foam
The strength criterion for this failure mode is a stress control criterion for the average through thickness compressive stress in the foam on the form:

\[ \bar{\sigma} \cdot \text{DAF} \leq \sigma_F \]

where:
- \( \bar{\sigma} \) = the average through thickness stress in the panel.

The foam crushing criterion will ensure that the loads do not reach levels where the structure will experience total collapse or excessive deformations. A main issue with this kind of strength criteria is that it is not unique in terms of the local state of stress and strain in the structure for different mastic support conditions or other relevant design modifications. This is not believed to be a big issue for the ultimate strength, since the foam appears to be robust and able to sustain rather large ranges of inelastic deformations without any observable differences in strength and damage.

3.5 Shear failure of bottom plywood plate
The maximum shear force in the bottom plywood plate in the most highly loaded cross section adjacent to the mastic support should fulfil the following requirement:

\[ Q(p \cdot \text{DAF}) \leq Q_c \]

where:
- \( Q \) = the calculated dynamic shear force per unit plate width at the most highly loaded cross-section of the bottom plywood plate for a quasi-static pressure \( p \cdot \text{DAF} \). The calculation of this quantity is described in more detail in Sec.5 [2.4]
- \( Q_c \) = the shear force capacity per unit plate width of the plywood plate for the considered material orientation, as defined in [6]. Unless a specific orientation of the plywood plate with respect to the mastic support can be guaranteed the value should be representative for the weakest cross-sectional direction of the laminate.

3.6 Bending failure of bottom plywood plate
The maximum bending moment in the bottom plywood plate in the most highly loaded cross-section should fulfil the following requirement:

\[ M(p \cdot DAF) \leq M_c \]

where:

\[ M = \text{the calculated dynamic bending moment per unit plate width at the most highly loaded cross-section of the bottom plywood plate for a quasi-static pressure } p \cdot DAF. \text{ This is most likely at a cross-section adjacent to the mastic support. The calculation of this quantity is described in more detail in Sec.5 [2.4].} \]

\[ M_c = \text{the bending moment capacity of the plywood plate for the considered material orientation, as defined in [6]. Unless a specific orientation of the plywood plate with respect to the mastic support can be guaranteed the value should be representative for the weakest cross-sectional direction of the laminate.} \]

### 3.7 Shear failure of secondary barrier level plywood plate (corner/knuckle)

The maximum shear force in the bottom plywood plate in the most highly loaded cross-section adjacent to the mastic support should fulfil the following requirement:

\[ Q \cdot DAF \leq Q_c \]

where:

\[ Q = \text{the calculated static shear force per unit plate width at the most highly loaded cross-section of the plywood plate for a quasi-static pressure } p. \text{ The calculation of this quantity is described in more detail in Sec.5 [2.5].} \]

\[ Q_c = \text{the shear force capacity per unit plate width of the plywood plate for the considered material orientation, as defined in [6]. Unless a specific orientation of the plywood plate with respect to the considered cross-section of the plate can be guaranteed, the value should be representative for the weakest cross-sectional direction of the laminate.} \]
3.8 Bending failure of secondary barrier level plywood plate (corner/knuckle)

The maximum bending moment in the bottom plywood plate in the most highly loaded cross-section should fulfil the following requirement:

\[ M \cdot DAF \leq M_c \]

where:

- \( M \) = the calculated static bending moment per unit plate width at the most highly loaded cross-section of the plywood plate for a static pressure \( p \). The calculation of this quantity is described in more detail in Sec.5 [2.5]
- \( M_c \) = the bending moment capacity of the plywood plate for the considered material orientation, as defined in [6]. Unless a specific orientation of the plywood plate with respect to the considered cross-section of the plate can be guaranteed, the value should be representative for the weakest cross-sectional direction of the laminate.

3.9 Failure modes for primary membrane

As mentioned in Sec.5 [2.1] sloshing impacts may cause permanent deformations of primary membrane corrugations, but are not likely to cause damages which can affect the tightness of the membrane if the insulation system remains intact and provides adequate support for the membrane. This is ensured by the strength assessment of the insulation panel.

Service experience indicates that deformation of primary membrane corrugations is likely to occur in the corners of the tank ceiling, and may occasionally occur on the lower part of longitudinal and transverse bulkheads if tanks are regularly operated with liquid heel up to the lower filling limit for the tanks. Deformations below certain limits are not considered to represent any degradation of the performance of the membrane, and will not necessarily require repair when discovered during survey. Deformations are categorised as follows:

- symmetric permanent deformation of the large corrugation.
- asymmetric permanent deformation of the large corrugation.
- symmetric permanent deformation of the small corrugation.
- asymmetric permanent deformation of the small corrugation.

The resistance for these deformation modes are measured as the pressure required to reach a limit value of the maximum membrane deformation measured in the direction normal to the corrugation, as illustrated in Figure 5.

Limits for acceptable permanent deformations of the membrane corrugations can be obtained upon request to the Society.
For tank newbuildings design measures should be taken to limit the extent of membrane deformations. It is suggested that designers consult the Society to agree on failure modes and acceptance criteria for modified membrane/membrane support designs.

### 4 NO96 system

#### 4.1 Impact areas

The strength of the NO96 insulation system should be checked for the impact load foot prints identified in Figure 6 and Figure 7 and defined in Table 3. Foot print 1 covers the entire insulation box surface and is not shown in the figures.

The foot prints have been carefully selected to obtain lower bound strength values for all relevant components of the primary and the secondary insulation box. Foot prints 2-8 are selected to primarily cover the strength of the secondary box. Of these, foot prints 2-5 cover the strength of the external members of the insulation box, whereas foot prints 6-8 cover the internal members.

Foot prints 9-13 are selected to primarily cover the strength of the primary box. Of these, foot prints 9-11 cover the strength of the external members of the insulation box, whereas foot prints 12 and 13 cover the internal members.

**Table 3 Definition of load foot prints**

<table>
<thead>
<tr>
<th>Foot Print</th>
<th>Width (mm)</th>
<th>Length (mm)</th>
<th>Area (cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FP1 (full box)</td>
<td>1141</td>
<td>990</td>
<td>11296</td>
</tr>
<tr>
<td>FP2</td>
<td>188</td>
<td>249</td>
<td>466</td>
</tr>
<tr>
<td>FP3</td>
<td>188</td>
<td>488</td>
<td>915</td>
</tr>
<tr>
<td>FP4</td>
<td>187</td>
<td>990</td>
<td>1849</td>
</tr>
<tr>
<td>FP5</td>
<td>249</td>
<td>990</td>
<td>2466</td>
</tr>
</tbody>
</table>
### Figure 6 Load foot prints primarily intended for assessment of secondary box members

<table>
<thead>
<tr>
<th>Foot Print</th>
<th>Width (mm)</th>
<th>Length (mm)</th>
<th>Area (cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FP6</td>
<td>353</td>
<td>249</td>
<td>876</td>
</tr>
<tr>
<td>FP7</td>
<td>353</td>
<td>368</td>
<td>1298</td>
</tr>
<tr>
<td>FP8</td>
<td>481</td>
<td>990</td>
<td>4759</td>
</tr>
<tr>
<td>FP9</td>
<td>188</td>
<td>118</td>
<td>221</td>
</tr>
<tr>
<td>FP10</td>
<td>530</td>
<td>118</td>
<td>625</td>
</tr>
<tr>
<td>FP11</td>
<td>1140</td>
<td>118</td>
<td>1346</td>
</tr>
<tr>
<td>FP12</td>
<td>530</td>
<td>249</td>
<td>1320</td>
</tr>
<tr>
<td>FP13</td>
<td>1141</td>
<td>251</td>
<td>2864</td>
</tr>
</tbody>
</table>
4.2 Failure modes

The strength assessment of the NO96 insulation boxes is based on the following set of critical failure modes (see Figure 8):

1) shear failure of the cover plate(s) of the primary insulation box.
2) bending failure of the cover plate(s) of the primary insulation box.
3) buckling of internal and external bulkheads of the primary insulation box.
4) buckling of internal and external bulkheads of the secondary insulation box.
5) through thickness crushing of the bottom/cover plates of the primary/secondary boxes at the intersection between the bulkheads of the primary and secondary boxes. The crushing deformation will lead to failure of the same plates in bending or shear.
4.3 Shear failure of cover plate(s)

The strength criterion for this failure mode is a stress control criterion for the through thickness shear force in the cover plates in the most highly loaded cross-section adjacent to the vertical bulkheads:

\[ Q \cdot DAF \leq Q_c \]

where:

- \( Q \) = the calculated section shear force per unit plate width at the most highly loaded cross-section of the cover plates. The calculation of this quantity is described in more detail in Sec.5 [3.1]
- \( Q_c \) = the section shear force capacity of the plywood plate for the considered material orientation.
4.4 Bending failure of cover plate(s)

The strength criterion for this failure mode is a stress control criterion for the bending stress in the cover plates in the most highly loaded cross-section adjacent to the vertical bulkheads:

\[ M \cdot \text{DAF} \leq M_c \]

where:

\[ M \] = the calculated bending moment per unit plate width at the most highly loaded cross-section of the cover plates, most likely to be at a cross-section adjacent to vertical box bulkheads. The calculation of this quantity is described in more detail in Sec.5 [3.1]

\[ M_c \] = the bending moment capacity of the plywood plate for the considered material orientation.

4.5 Buckling of plywood bulkheads

4.5.1 General

This section presents the formulas and procedures to be used for buckling strength assessment of the internal and external bulkheads of both the primary and secondary insulation box. The section is organised into three subsections describing the assessment of the static buckling capacity of the plywood bulkheads, how to account for the additional edge bending moment load potentially experienced by the external primary bulkheads, and last how to utilise the bulkheads in the dynamic and unstable load regime during rapid load events.

The buckling strength check should be based on the following strength equation:

\[ \sigma \cdot \text{DAF} \leq \sigma_c^D \]

where:

\[ \sigma \] = the nominal in-plane stress in the bulkhead plate, as described in Sec.5 [3.2]

\[ \sigma_c^D \] = the nominal in-plane buckling capacity of the bulkhead considering both dynamic and temperature effects.

The temperature dependent material stiffness and strength parameters should be evaluated at the mid-height of the primary and secondary bulkheads according to the procedure described in Sec.5 [4] and [5].
4.5.2 Static capacity assessment

The proposed buckling strength design curve is shown in the figure below, including results from finite element analyses for room temperature and uniform load for different slenderness.

The buckling strength design curve is given as:

\[
\frac{\sigma_c}{\sigma_f} = \frac{1.05 + \lambda^2 - \sqrt{(1.05 + \lambda^2)^2 - 4\lambda^2}}{2\lambda^2}
\]

where:

\(\sigma_c\) = the static buckling strength at room temperature
\(\lambda\) = the reduced slenderness defined as
\[\lambda = \sqrt{\frac{\sigma_f}{\sigma_E}}\]

\(\sigma_f\) = the material compressive strength (measured as average stress over the plate cross-section
\(\sigma_E\) = the elastic buckling strength of the plate

A graphical presentation of the buckling strength design curve is shown in Figure 9.

![Graphical representation of the buckling strength design curve](image)

**Figure 9 Graphical representation of the buckling strength design curve**

The elastic buckling stress for a simply supported bulkhead subjected to loads over parts or all of its edge can be expressed as:

\[
\sigma_E = k_1\sigma_{E,1} + k_2\sigma_f
\]
where:

\[ \sigma_{E,1} = \text{the elastic buckling stress for a simply supported bulkhead subjected to uniform load} \]

\[ k_1 = \text{factor depending on the size of the load exposed region of the bulkhead} \]

\[ k_2 = \text{factor to account for the increased buckling strength of slender bulkheads caused by the rotational restraint resulting from the finite thickness of the bulkheads} \]

\[ \sigma_F = \text{the material compressive strength as defined above} \]

The factors \( k_1, k_2, \) and \( \sigma_{E,1} \) are described in more detail below.

The elastic buckling stress for a simply supported bulkhead subjected to uniform load, \( \sigma_{E,1} \) may be taken as:

\[
\sigma_{E,1} = \frac{\pi^2 D (t/h)^2}{12}
\]

where:

\( t \) = the thickness

\( h \) = the height of the bulkhead plate

\( D \) = the generalised bending modulus of the bulkhead plate, defined as:

\[
D = h^3 \left( \frac{E_{b,1}}{h^4} + \frac{E_{b,2}}{b^4} + \frac{2}{h^2 b^2} \left( v_{12} E_{b,2} + 2 G_{12} \right) \right)
\]

\( E_{b,1}, E_{b,2} \) = bending stiffness properties of the laminate

\( v_{12} \) = the in-plane Poisson’s ratio

\( G_{12} \) = the in-plane shear modulus of the plate,

all defined in Sec.5 Table 1.

When the bulkheads are subjected to compressive loading over a limited area only, the slenderness is modified using the factor \( k_1 \) defined as:

\[
k_1 = \frac{0.007}{b/(h \cdot B) + 0.0042}, \text{ minimum 1.0}
\]

where:

\( h \) = the height of the bulkhead

\( B \) = the width of the bulkhead

\( b \) = the width of the loaded area (all given in mm).

The \( k_1 \)-factor as a function of load width \( b \) is shown in Figure 10.
The width $b$ may be taken as the width of the zone where the vertical stress is larger than half of the maximum stress at mid height of the bulkhead, as illustrated in Figure 11.

**Figure 10** The part load elastic buckling stress correction factor $k_1$

**Figure 11** Illustration of the criterion for selection of the load width $b$
The case of a part load located towards the end of the bulkhead has been considered, but found not to be critical. In this case, the redistribution of the load will be smaller, since the stress can redistribute in one direction only. However, the restraining effect of the end bulkhead will be larger when the load is close to the restraint, and some of the loading will be taken up by the end bulkhead.

If $\sigma_E$ is less than $0.5\sigma_F$, the elastic buckling stress may be increased to $k_2 \cdot \sigma_F$.

The factor $k_2$ is defined to account for the increased buckling strength of slender bulkheads caused by the rotational restraint resulting from the finite thickness of the bulkheads, as illustrated in Figure 12.

![Illustration of the end restraint effect experienced by slender bulkheads during buckling deformations](image)

**Figure 12 Illustration of the end restraint effect experienced by slender bulkheads during buckling deformations**

The rotational restraint is a function of the through-thickness stiffness of the top and bottom plywood plates connected to the bulkheads, and are taken to be linearly decreasing with the applied stress, as follows:

\[
\begin{align*}
\left( \frac{k_1\sigma_{E,1}}{\sigma_F} \right) &> 0.5 : k_2 = 0 \\
\left( \frac{k_1\sigma_{E,3}}{\sigma_F} \right) &< 0.18 : k_2 = 0.112 \\
else & : k_2 = 0.35 \cdot \left( 0.5 - \left( \frac{k_1\sigma_{E,1}}{\sigma_F} \right) \right)
\end{align*}
\]

The $k_2$-factor as a function of the normalised elastic buckling strength is illustrated in Figure 13.
In the case that dynamic strengthening is accounted for, as defined in [4.5.4], the rotational restraint effect shall be re-evaluated after calculation of the dynamic strength. The $k_2$-factor shall then be re-calculated, using the dynamic strength $\sigma_d$ instead of $\sigma_{E,1}$ in the formula above. Using the modified $k_2$-factor, the modified static and dynamic strength is calculated. Further iterations are not necessary.

![Graphical presentation of the end restraint correction factor $k_2$ as a function of the normalised elastic buckling strength](image)

**Figure 13** Graphical presentation of the end restraint correction factor $k_2$ as a function of the normalised elastic buckling strength

### 4.5.3 Correction of static capacity due to bending moment

The outer bulkheads in the primary box will be subjected to bending moment due to the deformation of the top plate, while the outer bulkheads in the secondary box will be subjected to bending moment due to deformation of the entire primary box. The bending stress in the bulkheads is calculated by linear finite element analysis. If bending is significant, the critical buckling stress shall be reduced as a result of the bending moment using a buckling-bending interaction equation. In this case, the rotational restraint correction should not be applied, i.e. $k_2=0$.

The bending stress resulting from linear analyses indicates that the bending moment in the outer bulkhead in the secondary box is relatively small. For uniform load, the bending stress in the outer bulkhead is approximately 10% of the total stress, which means that it may be neglected. This is also supported by the capacity tests carried out by GTT, which gives a capacity in excess of the one calculated by the design formulas proposed, without considering the bending moment effect.

For the end bulkheads in the primary box, the bending effect is much larger. For a strip load on the edge of the box, the bending stress is approximately 37% of the total stress, which means that the bending moment shall be accounted for.

The following interaction equation is used for the external bulkheads in the primary box:

$$\frac{\sigma}{\sigma_c} + \frac{\sigma_b}{\sigma_F(1 - \frac{\sigma}{\sigma_E})} = 1$$

where $\sigma_b$ is the bending stress in the bulkhead, $\sigma_F$ is the yield stress of the outer veneer, and $\sigma$ is the critical average stress under combined axial load and bending. The values to be used for the bending stress and the
yield stress in the above equation depends on whether the plywood plates are modelled as laminate plates or equivalent homogeneous orthotropic plates, as summarised in Table 4.

**Table 4 Definition of bending and yield stress values depending on plate modelling approach**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Plate modelling approach</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Laminate plate</td>
</tr>
<tr>
<td>$\sigma_b$</td>
<td>$\sigma_b = \sigma_{\text{max}} - \sigma_{av} \frac{E_{\text{veneer}}}{E_{\text{average}}}$</td>
</tr>
<tr>
<td>$\sigma_F$</td>
<td>$\sigma_F = \sigma_F^{\text{veneer}} = \sigma_F \frac{E_{\text{veneer}}}{E_{\text{average}}}$</td>
</tr>
</tbody>
</table>

The parameters referred to in the table are defined as follows:

- $E_{\text{veneer}}$ and $E_{\text{average}}$ are the elastic moduli of the surface veneer and the entire laminate, respectively, in the material direction coinciding with the load.
- $\sigma_{\text{max}}$ is the maximum calculated stress at the extreme fibre of the plate.
- $\sigma_{av}$ is the average stress over the plate cross-section, calculated as
  
  $$\sigma_{av} = \frac{1}{h} \int_0^h \sigma dz$$

  in the case that the z axis is oriented in the through thickness direction of the plate.

- $\sigma_F^{\text{marg}}$ is the material strength of the laminate plate specified in Table 6.

Since the bending moment is proportional to the load, $\sigma_b$ may be replaced by $\sigma_b = C_b \sigma$, where $C_b$ is the ratio between bending stress and average stress in the bulkhead, as determined from finite element analysis. The solution to the interaction equation is then:

$$\sigma = \frac{1}{2} (\sigma_e + \sigma_c + C_b \frac{\sigma_e}{\sigma_F} \sigma_c) + \frac{1}{2 \sigma_F} \sqrt{K}$$

where

$$K = \sqrt{K_A + K_R}$$

$$K_A = \sigma_F^2 \sigma_E^2 + 2C_b \sigma_F \sigma_c \sigma_E \sigma_c^2 - 2 \sigma_c \sigma_c^2$$

$$K_R = C_b \sigma_F^2 \sigma_c^2 + 2C_b \sigma_F \sigma_c \sigma_E^2 + \sigma_c^2 \sigma_F^2$$
The bending moment applied to the external bulkheads of the primary box due to bending of the cover plate need not to be taken greater than the value implied by the rotational restraint limit as defined by the above. This means that the maximum moment is defined for the load level giving a nominal compressive stress in the bulkhead plate equal to 0.5\(\sigma_f\).

### 4.5.4 Dynamic strength correction

Buckling failure is a dynamic event, and it is clear that inertia will allow a buckling exposed structure to sustain in-plane loads beyond the static buckling capacity without suffering damage given that the load event is sufficiently rapid. This effect is accounted for by multiplying the critical buckling stress by a dynamic strength factor (\(DSF = \sigma_f^D / \sigma_f\)) defined as a function of the slenderness of the bulkhead and the ratio of the rise time of the sloshing impact stress response (\(t_r\)) and the natural period relevant for lateral oscillation of the bulkhead plate (\(T_e\)), as follows:

\[
DSF = f_1(\lambda) - \frac{f_2(\lambda)}{0.55} \cdot \frac{t_r}{T_e} \leq 0.55
\]

\[
DSF = f_2(\lambda) - \left( \frac{f_2(\lambda) - 1}{0.95} \right) \left( \frac{t_r}{T_e} - 0.55 \right) \cdot 0.55 < \frac{t_r}{T_e} \leq 1.50
\]

\[DSF = 1.0, \frac{t_r}{T_e} > 1.50\]

\(f_1(\lambda)\) and \(f_2(\lambda)\) are given as:

\[f_1(\lambda) = \frac{f_2 \sigma_f}{\sigma_f}, \text{ maximum 4 (see Figure 15)}\]

\[f_3 = \text{safety factor } = 0.95\]

\[f_2(\lambda) = \min(f_1(\lambda), 1.5)\]

A graphical representation of the dynamic strength factor (\(DSF\)) as a function of the ratio between rise time and natural period, \(t_r / T_e\), is shown in Figure 14.

The natural period relevant for lateral oscillation of the bulkhead plate can for all load cases be calculated as:

\[T_e = \frac{2h^2}{t \pi} \sqrt{\frac{12\rho}{E_b}}\]

Where:

- \(t\) = the thickness of the bulkhead plate
- \(h\) = the height of the bulkhead
- \(\rho\) = the density of the plywood
- \(E_b\) = the bending stiffness of the plate about its stiffest axis, see Sec.5 Table 1.
The relationship between the rise time of the impact pressure and the rise time of the stress response in the bulkheads of the primary and the secondary insulation boxes are summarised in Table 5. Response rise times for impact pressure rise times between the values given in the table should be obtained by linear interpolation. The response rise time can be taken as equal to the impact pressure rise time for impact pressure rise times exceeding 1.0ms.

**Table 5 Relationship between impact pressure rise time and vertical stress response rise times for the primary and secondary box bulkheads**

<table>
<thead>
<tr>
<th>$t_{\text{load}} \ [\text{ms}]$</th>
<th>$t_{\text{response}} \ [\text{ms}]$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td><strong>Primary box</strong></td>
</tr>
<tr>
<td>0.5</td>
<td>0.7</td>
</tr>
<tr>
<td>1.0</td>
<td>0.75</td>
</tr>
</tbody>
</table>

**Figure 14 Definition of the DSF as a function of the ratio between the rise time of the sloshing stress response and the natural period of the bulkhead plates in the lateral bending deformation mode**

The dynamic buckling strength is then found as:

$$\sigma_{\text{c}}^{D} = \sigma_{\text{c}} \cdot \text{DSF}$$

The dynamic low temperature buckling strength is limited by the material compression strength:

$$\text{Max}(\sigma_{\text{c}}^{D}) = \sigma_{F}$$
Figure 15 Graphical representation of the slenderness dependent factor $f_1$

4.6 Crushing of plates at bulkhead intersections

The crushing strength is measured in terms of the nominal stress in the vertical bulkhead of the secondary insulation box, $\sigma_{cv}$, measured against a critical value of this parameter, $\sigma_{cv}^c$, associated with through thickness crushing of the insulation box cover plates and more importantly the associated bending failure of these cover plates. The capacity has been determined from laboratory tests and numerical simulations. The strength check should be carried out using the following equation:

$$\sigma_{cv} \cdot \text{DAF} \cdot \gamma_k \leq \frac{\sigma_{cv}^c}{\gamma_M}$$

where:

$\sigma_{cv}$ = the nominal stress in the bulkhead of the primary insulation box in the region of the bulkhead intersections

$\sigma_{cv}^c$ = the through thickness compressive strength of the plywood plate measured in terms of the nominal stress in the secondary box bulkhead.

5 Plywood strength data

Strength data for 9mm and 12mm thick plywood plates are summarised in Table 6. The first subscript $c$ identifies that this is a limit value (critical), and the second and potentially third alphanumeric subscript refers to the material directions of the laminate, as defined in Sec.5 Figure 19. For bending moments the subscript refers to the vector direction of the moment, i.e. $M_1$ denotes bending about the 2-axis and $M_2$ denotes bending about the 1-axis.
### Table 6 Mean strength properties for the 9mm and 12 mm plywood laminates

<table>
<thead>
<tr>
<th></th>
<th>9 mm</th>
<th>12 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>20°C</td>
<td>-16°C</td>
</tr>
<tr>
<td>(\sigma_{c,1t}), tension (MPa)</td>
<td>70.0</td>
<td>60.0</td>
</tr>
<tr>
<td>(\sigma_{c,1c}), compr. (MPa)</td>
<td>43.0</td>
<td>65.0</td>
</tr>
<tr>
<td>(\sigma_{c,2t}), tension (MPa)</td>
<td>54.0</td>
<td>46.0</td>
</tr>
<tr>
<td>(\sigma_{c,2c}), compr. (MPa)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>(\sigma_{c,\alpha}), compr. (MPa)</td>
<td>32</td>
<td>32</td>
</tr>
<tr>
<td>(M_{c,1}) (Nmm/mm)</td>
<td>1100</td>
<td>935</td>
</tr>
<tr>
<td>(M_{c,2}) (Nmm/mm)</td>
<td>760</td>
<td>650</td>
</tr>
<tr>
<td>(Q_{c,13}) (N/mm)</td>
<td>59</td>
<td>59</td>
</tr>
<tr>
<td>(Q_{c,23}) (N/mm)</td>
<td>43</td>
<td>43</td>
</tr>
</tbody>
</table>

* Calculated based on the veneer tensile strength.

### 6 RPUF strength data

The temperature and strain rate dependent through thickness crushing strength of the reinforced polyurethane foam is given in Table 7.

### Table 7 Temperature and strain rate dependent strength for through thickness compression of the RPUF

<table>
<thead>
<tr>
<th>Temperature</th>
<th>0*</th>
<th>0.1/s</th>
<th>4/s</th>
<th>100/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>20°C</td>
<td>1.2</td>
<td>1.35</td>
<td>1.6</td>
<td>1.65</td>
</tr>
<tr>
<td>-163°C</td>
<td>2.0</td>
<td>2.2</td>
<td>2.3</td>
<td>2.3</td>
</tr>
</tbody>
</table>

* Quasi-static loading.

### 6.1 Evaluation of temperature dependent strength

It should be assumed that the temperature at the hull structure side of the containment system is 20°C, with a linear variation down to -163°C at the primary barrier of the system. The material strength can be taken as a linear function of the temperature between the limits given in Table 6 and Table 7. The material strength for temperatures between the limits given in should thus be obtained by linear interpolation between these limits based on the through thickness coordinate at the location being investigated.
6.2 Evaluation of strain rate dependent strength

The strain rate dependent material strength should be determined based on linear interpolation between the values given in Table 7 using a strain rate calculated as follows:

\[
\dot{\varepsilon} = \frac{\bar{\sigma} \cdot \text{DAF}}{2 \cdot E_3 \cdot t_r^{\text{response}}}
\]

where:

\(\bar{\sigma}\) = the average through thickness stress in the foam as defined in Sec.5 [2.3]

\(\text{DAF}\) = the dynamic amplification factor as defined Sec.5 [6.1]

\(E_3\) = the relevant temperature dependent through thickness elastic modulus of the reinforced polyurethane foam as defined in Sec.5 [4]

\(t_r^{\text{response}}\) = the estimated rise time of the average through thickness stress, and should be calculated as follows:

— for the upper part of the foam the response is considered to follow the load, and the response of the rise time should consequently be taken equal as the load rise time.

— for the foam adjacent to the bottom the response rise time should be taken equal to the load rise time for load rise times larger than 1.5ms. For shorter load rise times the response rise time should be obtained by interpolation between the values given in Table 8.

**Table 8 Relationship between load and response rise times for bottom foam for load rise times below 1.5 ms**

<table>
<thead>
<tr>
<th>(t_r^{\text{load}} [\text{ms}])</th>
<th>(t_r^{\text{response}} [\text{ms}])</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.250</td>
<td>0.500</td>
</tr>
<tr>
<td>1.500</td>
<td>1.500</td>
</tr>
</tbody>
</table>
SECTION 7 STRENGTH OF INNER HULL STRUCTURE

1 General

The sloshing impact loads acting on the containment system inside the cargo tanks shall be transferred into the supporting hull structure. It is therefore necessary to ensure that the hull structure has strength to carry the sloshing loads. The complex interaction between the response of the cargo containment and the hull structure as well as the challenge in selecting appropriate load scenarios based on the experimentally determined sloshing impact loads makes it difficult to accurately assess the structural response of the inner hull plates. A simplified comparative assessment is therefore recommended to ensure that the inner hull is properly dimensioned to carry impact loads. The reference case to be considered in the comparative assessment is defined in the next section.

Alternative assessment methods may be accepted by the Society if it can be justified by the designer.

The inner hull strength assessment should be carried out for inner hull stiffeners and plates located in the most sloshing exposed regions of the tank. For normal filling operations, particular attention should be paid to the following areas, see Figure 1:

— the inner deck and chamfer area at locations close to the transverse bulkheads
— the transverse bulkheads at locations close to the inner deck
— the inner deck and chamfer area close to the upper chamfer knuckle along the entire length of the tank
— the chamfer area close to the lower chamfer knuckle along the entire length of the tank.

Figure 1 Part of inner shell which should be specially considered with respect to sloshing (top view)

For conventional LNG carriers the extension of one web frame spacing is normally applied in longitudinal/transverse/vertical directions from transverse bulkhead and upper and lower chamfer knuckles. The extent of the areas shall be determined based on results from sloshing tests for the actual tank configuration.

For operation with partially filled tanks the most severe locations will be the lower part of the longitudinal bulkheads and parts of the transverse bulkheads adjacent to the tank corners and above the level of the hopper knuckle,
2 Comparative basis (reference case)

The basis for the comparative assessment of the hull structure of the new LNG carrier should be taken as the reference case vessel specified in Sec.2 [1.2]. Since tank fillings between 90% and 98.5% of the tank height are assumed for the reference case, a typical deck panel is considered as the basis for the comparative assessment.

A stiffened panel with the dimensions listed in the following is taken as representative for the reference case:

**Vessel with Mark III containment system:**

<table>
<thead>
<tr>
<th>t (mm)</th>
<th>S (mm)</th>
<th>L (mm)</th>
<th>Profile (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>13</td>
<td>880</td>
<td>2800</td>
<td>250×90×12/16</td>
</tr>
</tbody>
</table>

**Vessel with NO96 containment system:**

<table>
<thead>
<tr>
<th>t (mm)</th>
<th>S (mm)</th>
<th>L (mm)</th>
<th>Profile (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>14</td>
<td>810</td>
<td>2800</td>
<td>200×90×9/14</td>
</tr>
</tbody>
</table>

where \( t \) is plate thickness, \( S \) is stiffener spacing, \( L \) is stiffener span, and Profile is stiffener scantlings. The yield stress for the hull structure material for the reference case is 235 MPa.

3 Strength of hull plating

The key assumptions for the simplified assessment are:

— the strength utilisation of the target case plating should not exceed the strength utilisation of the reference case plating
— the spatial distribution of forces onto the inner hull plates are similar for the reference and target cases
— the length of the plate is significantly longer than the width of the plate for both reference and target case, and hence can be disregarded in the assessment
— the increase of the force applied to the plating is proportional to the increase of impact pressure acting on the primary barrier level.

Under these conditions the bending moment carried by the plate is proportional to the pressure and to the square of the span of the plate (distance between stiffeners). The required inner hull plate thickness can then be calculated from the following formula:

\[
t_{\text{tar}} = \sqrt{\frac{p_{\text{tar}} \cdot \sigma_{\text{av.tar}}}{p_{\text{ref}} \cdot \sigma_{\text{av.ref}} \cdot s_{\text{tar}} \cdot t_{\text{ref}}}}
\]

where:

\( p_{\text{tar}} \) = the design impact pressure for the target case
\( p_{\text{ref}} \) = the design impact pressure for the reference case
\( \sigma_{\text{av.tar}} \) = the available strength of the inner hull steel of the target vessel remaining for sloshing loads
4 Strength of hull stiffeners

A simplified comparative assessment method can be used to determine the stiffener strength requirements as described in the following. The assessment covers both the shear capacity and the bending capacity of the inner hull stiffeners.

It should be noted that the following method is applicable in the case where the stiffeners span in the target case is increased as compared to the reference case. Reference case stiffeners strength remains the minimum requirement for shorter spans than in the reference vessel.

The key assumptions for the simplified assessment are:

— the strength utilisation of the target case stiffeners should not exceed the strength utilisation of the reference case stiffeners
— the axial stress acting in the stiffener is similar for the reference and the target cases
— the spatial distribution of forces onto the inner hull stiffeners are similar for the reference and target cases
— the increase of the force applied to the stiffeners is proportional to the increase of impact pressure acting on the primary barrier level.

Under these conditions the maximum bending moment carried by the stiffeners can be conservatively considered proportional to the pressure and to the span of the stiffener elevated to the power of 1.5. The required inner hull stiffener section modulus can then be calculated from the following formula:

\[
Z_{tar} = \frac{p_{tar}}{p_{ref}} \cdot \frac{\sigma_{av,tar}}{\sigma_{av,ref}} \cdot \left( \frac{l_{tar}}{l_{ref}} \right)^{1.5} \cdot \frac{s_{tar}}{s_{ref}} \cdot \frac{t_{ref}}{Z_{ref}}
\]

where:

\( p_{tar} \) = the design impact pressure for the target case
\( p_{ref} \) = the design impact pressure for the reference case
\( \sigma_{av,tar} \) = the available strength of the inner hull steel of the target vessel remaining for sloshing loads
\( \sigma_{av,ref} \) = the available strength of the inner hull steel of the reference vessel remaining for sloshing loads
\( s_{tar} \) = the stiffener spacing for the considered inner hull structure in the target vessel
\( s_{ref} \) = the stiffener spacing for the considered inner hull structure in the reference vessel as defined in [2]
\( t_{ref} \) = the inner hull plate thickness of the reference vessel

\( \sigma_{av,ref} \) = the available strength of the inner hull steel of the reference vessel remaining for sloshing loads
\( s_{ref} \) = the stiffener spacing for the considered inner hull structure in the reference vessel as defined in [2]
\( t_{ref} \) = the inner hull plate thickness of the reference vessel
The maximum shear force carried by the stiffeners is proportional to the pressure span of the stiffeners, and the required inner hull stiffener shear area can be calculated from the following formula:

\[
A_{S_{sw}} = \frac{p_{\text{tar}}}{p_{\text{ref}}} \cdot \frac{\sigma_{av_{r_{e f f}}}}{\sigma_{av_{t_{a r}}}} \cdot \frac{L_{\text{tar}}}{L_{\text{ref}}} \cdot \frac{S_{\text{tar}}}{S_{\text{ref}}} \cdot A_{s_{r_{e f f}}}
\]

where:

- \( p_{\text{tar}} \) = the design impact pressure for the target case
- \( p_{\text{ref}} \) = the design impact pressure for the reference case
- \( \sigma_{av_{t_{a r}}} \) = the available strength of the inner hull steel of the target vessel remaining for sloshing loads
- \( \sigma_{av_{r_{e f f}}} \) = the available strength of the inner hull steel of the reference vessel remaining for sloshing loads
- \( S_{\text{tar}} \) = the stiffener spacing for the considered inner hull structure in the target vessel
- \( S_{\text{ref}} \) = the stiffener spacing for the considered inner hull structure in the reference vessel as defined in [2]
- \( t_{\text{ref}} \) = the inner hull plate thickness of the reference vessel
- \( L_{\text{tar}} \) = the stiffener span for the considered inner hull structure in the target vessel
- \( L_{\text{ref}} \) = the stiffener span for the considered inner hull structure in the reference vessel as defined in [2]
- \( A_{s_{r_{e f f}}} \) = the shear area of the inner hull stiffeners of the reference vessel as defined in [2]
5 Effect of differences in vapour pressure

Two types of loading are considered in the analysis; a static vapour pressure $p_{vapour}$ and a dynamic $p_{sloshing}$. As the static pressure utilises some of the hull capacity, a limited steel strength is available for carrying the additional dynamic sloshing impact pressure. The remaining strength available to carry the sloshing impact pressure in the target and reference cases are denoted $\sigma_{av\_tar}$ and $\sigma_{av\_ref}$ in the definitions above.

The acceptance criterion is given on the total stress experienced by the structure by a combination of both static and dynamic loads:

$$\sigma = \sigma_{vapour} + \sigma_{sloshing} \leq \sigma_{allowable}$$

Where:

- $\sigma_{allowable}$ = $\sigma_f$ for stiffeners
  = $2\sigma_f$ for plates (plastic mechanism)

- $\sigma_{vapour}$ = $M_{vapour} / Z$

- $\sigma_f$ = the yield strength of the material
- $M_{vapour}$ = he bending moment caused by vapour pressure in the structural element to be checked
- $Z$ = the section modulus

It follows that the available strength to sustain sloshing loads in the reference case and the minimum requirement for the remaining strength in the target case are:

$$\sigma_{av\_ref} = \sigma_{allowable\_ref} - \frac{M_{vapour\_ref}}{Z_{ref}} \quad \text{and} \quad \sigma_{av\_tar} = \sigma_{allowable\_tar} - \frac{M_{vapour\_tar}}{Z_{tar}}$$

As a result, the solutions become implicit for the target plate thickness in [3] and for the target stiffener section modulus in [4]. An iterative approach is therefore necessary to determine the plate thickness and stiffener section modulus requirements. In cases where the first iteration satisfies requirements, there is no need for iteration. Otherwise, since the section modulus is affected by the plate thickness, iteration on the plate thickness should be performed before the iteration on the section modulus.
SECTION 8 RESPONSE AND STRENGTH OF PUMP TOWER AND SUPPORTS

1 General
The strength of the pump tower main structure, the base support and the liquid dome area should be checked with respect to ultimate strength (ULS) and fatigue (FLS). In addition, a vibration check should be carried out of the main structure.

Finite element analyses of the pump tower structure should be carried out to determine the response in the structure due to the loads relevant for the ULS and the FLS assessments. Separate analyses should be carried out for the main structure, for the base support, and for the liquid dome area, as described in the following sections.

2 Response analysis of main structure

2.1 Finite element model
The finite element analysis of the main structure of the pump tower should be carried out as a 3D analysis. The analysis may be linear elastic. A principle sketch of the tower structure is shown in Figure 1.

![Figure 1 Illustration of pump tower main structure](image-url)
The main strength members of the pump tower are the emergency pipes and the two discharge pipes. The columns are connected by intermediate braces, and are fitted to the base plate at the bottom of the tower. The filling pipe and the float level gauge are connected to the tower by supports.

The members that should be included in the finite element model are:

— emergency pipe
— discharge pipes (port and starboard)
— filling pipe
— float level gauge
— supports for filling pipe and float level gauge
— braces
— base plate.

The columns and braces may be modelled with beam elements, while the base plate should be modelled with shell elements. The beam elements should be positioned in the centre of each column/brace, as indicated by the dashed lines in Figure 2, with the finite element nodes at the intersection between the elements. The restraining effect of the joint on the braces may be accounted for by specifying rigid ends of the elements. Loads should only be applied to the free span part of the brace elements.

Figure 2 Illustration of beam element modelling

2.2 Load application

Sloshing loads should be applied to the columns and braces located below the liquid dome.

Gravity, inertia loads and thermal loads should be applied to all elements. The mass and load/drag force effect of additional elements should be included (such as pumps, platforms, ladders and valves), by distributing the masses and modifying the drag coefficients to the main members of the tower.

The mass of liquid inside the columns shall also be included for calculation of inertia forces. The mass may be applied to the columns as an additional distributed mass. However, the amount of liquid will depend on the filling level, and a separate model will therefore be required for each filling level considered. Therefore, the force due to the inertia acting on the additional mass may alternatively be included directly by a distributed load.
2.3 Boundary conditions

At the top, the emergency pipe and the discharge pipes are connected to the liquid dome cover plate. The boundary conditions at the top of each column may be taken as:

\[ T_x = T_y = T_z = R_z = \text{fixed} \]
\[ R_x = R_y = \text{free} \]

where \( T \) is translation, \( R \) is rotation, \( x \) is longitudinal direction, \( y \) is transverse direction, and \( z \) is vertical direction, ref. Sec.4 Figure 3.

Alternatively, translational \( (k_x \text{ and } k_y) \) and rotational \( (k_{Rx}, k_{Ry}, k_{Rz}) \) springs may be used. The spring stiffness should then be documented from the finite element analysis of the liquid dome area.

The sliding joints of the three upper horizontal struts should be modelled so that axial displacement and rotation about the axial direction of the strut are allowed:

\[ T_x = R_x = \text{free} \]

where the \( x \)-direction is in the length direction of the strut.

At the bottom, the emergency pipe and the discharge pipes are connected to the base plate. In order to account for the sliding pads between the base plate and the base support, the boundary conditions of the base plate are taken as:

\[ T_x = T_y = R_z = \text{fixed} \]
\[ T_z = R_x = R_y = \text{free} \]

where \( x \) is longitudinal direction, \( y \) is transverse direction, and \( z \) is vertical direction.

Alternatively, translational \( (k_x \text{ and } k_y) \) and rotational \( (k_{Rz}) \) springs may be used. The spring stiffness should then be documented from the finite element analysis of the base support.

The filling pipe and the float level gauge should be connected to the emergency pipe and the horizontal back struts, respectively. The connection should be modelled so that vertical displacement and rotation of the pipes are allowed.

2.4 Material properties

Characteristic values of the material properties shall be applied in the response analysis. The pump tower is usually constructed by stainless steel, and typically of qualities in the 300-series. Properties for the 304L stainless steel are given in Table 1.

**Table 1 Material properties for 304L stainless steel**

<table>
<thead>
<tr>
<th>( E ) (MPa)</th>
<th>200 000</th>
<th>Modulus of elasticity</th>
</tr>
</thead>
<tbody>
<tr>
<td>( v ) (-)</td>
<td>0.3</td>
<td>Poisson’s ratio</td>
</tr>
<tr>
<td>( \rho ) (kg/m(^3))</td>
<td>7970</td>
<td>Density</td>
</tr>
<tr>
<td>( \sigma_{y,20} ) (MPa)</td>
<td>170</td>
<td>Yield limit at 20(^{\circ})C</td>
</tr>
<tr>
<td>( \sigma_{y,-78} ) (MPa)</td>
<td>180</td>
<td>Yield limit at -78(^{\circ})C</td>
</tr>
<tr>
<td>( \sigma_{y,-163} ) (MPa)</td>
<td>220</td>
<td>Yield limit at -163(^{\circ})C</td>
</tr>
<tr>
<td>( \alpha_{20} ) (10(^{-6})/(^{\circ})C)</td>
<td>16.3</td>
<td>Thermal exp. coefficient at 20(^{\circ})C</td>
</tr>
<tr>
<td>( \alpha_{-163} ) (10(^{-6})/(^{\circ})C)</td>
<td>13.5</td>
<td>Thermal exp. coefficient at -163(^{\circ})C</td>
</tr>
</tbody>
</table>

For intermediate temperatures, the yield limit and the thermal expansion coefficient may be interpolated linearly between the specified values. The reference temperature for the thermal expansion is 20\(^{\circ}\)C.
2.5 Response parameters

For tubular members, the following result parameters should be determined for critical sections along the length of the member:
- axial force (or tension/compression stress)
- vertical bending moment (or bending stress)
- horizontal bending moment (or bending stress)
- torsional moment (or torsional stress).

For joints, the following result parameters should be determined:
- axial load
- horizontal bending moment
- vertical bending moment.

In addition, total reaction forces at the top and at the bottom of the tower shall be determined.

3 Response analysis of base support

The finite element analysis of the base support of the pump tower should be carried out as a 3D analysis using shell elements. The analysis may be linear elastic.

The finite element model should include the base support itself, and a portion of the double bottom. The portion of the double bottom should be large enough to provide realistic restraint conditions to the base support.

Mesh density may be chosen according to recommendations given in DNVGL CG 0129, Fatigue assessment of ship structures and DNVGL RP 0005.

The reaction forces determined from the analysis of the pump tower main structure should be applied to the model, and combined with hull girder loads due to global bending moment, double bottom stresses due to external sea pressure, and thermal stresses due to the thermal gradient in the base support. The temperature variation through the height of the base support should be determined by a separate analysis.

The resulting stress field in the base support should be determined, and applied for the ULS and FLS strength assessment.

Areas of special concern for the base support are the connection between the base support and the inner bottom, the connection to the primary membrane, the connection to the secondary membrane, and the tower guide structure.

In addition, the reinforcements in the inner bottom underneath the base support should be considered.

The complete set of hot spots to be considered should be agreed with the Society.

4 Response analysis of liquid dome area

The finite element analysis of the liquid dome area should be carried out as a 3D analysis using shell elements, as illustrated in Figure 3. The top cover plate and the connection to the pump tower should be included in the analysis. The analysis may be linear elastic.

Mesh density may be chosen according to recommendations given in DNVGL CG 0129, Fatigue assessment of ship structures and DNVGL RP 0005.

The reaction forces determined from the analysis of the pump tower main structure should be applied to the model, and combined with hull girder loads due to global bending moment.

The resulting stress field in the liquid dome area should be determined, and applied for the ULS and FLS strength assessment.

Areas of special concern are the connection between the pump tower structure and the liquid dome cover plate, and the corner areas of the liquid dome.
5 ULS assessment

The main structure, the base support and the liquid dome area should be checked with respect to yielding, using the von Mises yield criterion:

\[ S_{\gamma F} \leq \frac{R_{\gamma}}{\gamma_M} \]

where:

- \( S = \sigma_e \)
- \( R = \sigma_F \)
- \( \sigma_e = \) the von Mises equivalent stress:

\[ \sigma_e = \sqrt{\sigma_x^2 + \sigma_y^2 - \sigma_x \sigma_y + 3\tau} \]

- \( \sigma_F = \) the yield stress of the material

In addition, the tubular members of the main structure should be checked with respect to buckling and combinations of bending and buckling. The tubular joints should be checked with respect to shear capacity.

The buckling/bending check of the tubular members may be carried out according to DNVGL CG 0128, Buckling or API-RP-2A /4/. Strength checks should be carried out for several sections along the length of each brace, in order to make sure that the most critical section is checked, i.e. the worst combination of axial and bending stress.

The shear capacity of the tubular joints should be checked according to API RP 2A /4/.

For each failure mode, it should be checked that the strength of the structure is acceptable according to the requirements given in Sec.2 [2].
6 FLS assessment

The fatigue damage should be calculated for relevant parts of the structure.

The damage may be calculated based on the SN fatigue approach under the assumption of linear cumulative damage (Miner-Palmgren rule):

\[
D = \sum_{i=1}^{k} \frac{n_i}{N_i}
\]

Where:
- \( D \) = accumulated fatigue damage
- \( n_i \) = number of stress cycles within stress range \( i \)
- \( N_i \) = number of cycles to failure at constant stress range \( \Delta \sigma_i \) according to the appropriate SN-curve

The total damage is found by summing the damage for each

- sea state (according to the North Atlantic scatter diagram)
- for each sea state: all load conditions (full load/ballast)
- for each sea state and load conditions: all headings (0-180°).

The cumulative damage for a design life of 40 years should not exceed the acceptable damage as defined in Sec.2 [2].

Basic design SN-curves are generally given as

\[
\log N = \log \bar{a} - m \log \Delta \sigma
\]

Where:
- \( N \) = number of cycles to failure at the stress range \( \Delta \sigma \)
- \( \bar{a}, m \) = constants

SN-curves for specific materials and details are obtained from fatigue tests, and are typically based on the mean-minus-two-standard-deviation curves for relevant experimental data, meaning a 97.6% probability of survival.

For the tubular joints of the main structure of the pump tower, the SN-curve should be taken as the two-slope T-curve for air given in DNVGL RP 0005:

- \( N \leq 10^7 \): \( \log \bar{a} = 12.164 \), \( m = 3 \)
- \( N > 10^7 \): \( \log \bar{a} = 15.606 \), \( m = 5 \)

This curve is based on a reference thickness of 32mm. For other thicknesses, a thickness modification should be applied, as specified in DNVGL RP 0005. It should be noted that the T-curve assumes full penetration welding.
For simplicity, a one-slope curve may conservatively be used, with

$$\log a = 12.164$$

and $m = 3$ for the entire stress range. The fatigue damage for each load condition can then be expressed as:

$$D = \frac{\nu_0 T_d}{\bar{\sigma}} q^m \Gamma(1 + \frac{m}{h})$$

where:

- $\nu_0$ = the long-term average response zero-crossing frequency
- $T_d$ = the design life of the ship in seconds ($= 1.26 \times 10^8$ for 40 years design life)
- $\Gamma(1+m/h)$ = the gamma function (equal to 6.0 for $m=3.0$ and $h=1.0$).

Stress concentration factors (SCFs) should be applied according to recommendations given in DNVGL RP 0005, Sec. 2.[8] and App. 2. The hot spot stress to be used in combination with the T-curve is calculated as

$$\sigma_{hot, spot} = SCF \sigma_{nom}$$

The combined hot spot stress due to axial force and bending moments should be calculated for several points around the circumference of the intersection. Alternatively, the SCFs may be calculated using influence functions as proposed to Efthymiou /5/.

For the base support and the liquid dome area, the appropriate SN-curve to be used for each detail should be taken according to the recommendations given in DNVGL RP 0005 App.1. Alternatively, the hot spot stress may be calculated by finite element analysis and combined with the D-curve, as explained in section 2.13 of the RP. The procedure given in DNVGL CG 0129, Fatigue assessment of ship structures may also be followed. Special attention should be given to the connection between the base support and the double bottom, and to the connection between the base support and the primary/secondary membranes.

### 7 Vibration check

Modal analysis of the pump tower should be carried out to check that the natural frequencies of the pump tower are outside the resonance intervals of excitations coming from the propeller blade and the engine room. The same model as used for the response analysis may be applied.

For LNG carriers operating with tank filling restrictions the vibration check should be carried out for ballast condition, 10%H filling, 70%H filling and 98%H filling. Additional tank fillings may have to be considered for vessels or units designed to operate with unrestricted filling. Mass of liquid inside pipes and added mass effects shall be considered in the analysis.

The margin between the excitation frequencies and the resonance frequencies should be at least 10%.

If the vessel is intended for ice-going operation, the excitation frequencies and forces from breaking the ice should also be considered.
SECTION 9 REFERENCES

1 General


/6/ NORSOK, N-004 *Design of Steel Structures*.


/10/ Det Norske Veritas (DNV), DNV-RP-A203 *Technology Qualification*. 
CHANGES – HISTORIC

There are currently no historical changes for this document.
Driven by our purpose of safeguarding life, property and the environment, DNV GL enables organizations to advance the safety and sustainability of their business. We provide classification and technical assurance along with software and independent expert advisory services to the maritime, oil and gas, and energy industries. We also provide certification services to customers across a wide range of industries. Operating in more than 100 countries, our 16 000 professionals are dedicated to helping our customers make the world safer, smarter and greener.