

◎ Technical paper

Analysis of Long Term Stress and Fatigue of Semi-Submersible Drilling Platform

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반잠수식 시추선의 피로 해석 연구

유 병 건

Key Words : Semi-submersible Drilling Platform(반잠수식 시추선), Wave Loading(파랑 하중), Short-Term Response(단기 응답), Long Term Response(장기 응답), Fatigue Analysis(피로 해석)

초 록

반잠수식 시추선의 설계에 있어서 구조의 피로수명을 정확하게 추정하는 것은 매우 중요하며, 또한 어려운 일이다. 본 논문에서는 이미 실용화된 Aker H-3 모델에 대하여 Palmgren-Miner 가정에 의하여 시추선의 hot-point인 수평 및 수직 K-brace 멤버에 대하여 피로수명을 로이드 선급의 커브에 따라 계산한 결과 북해의 해상환경 조건에서는 16년 동안 계속 작업을 할 수 있다는 결론을 얻었으나 응력 집중계수(SCF)를 선정하는데 불확실성으로 인하여 사용하는데 주의를 요한다.

1. Introduction

This paper presents results of a study of the wave induced motions, loads and structural response of semi-submersible drilling platform.

The principal objective of the study was to obtain estimates of the long term stress maxima and of the fatigue life of the critical parts of the structure when operating in severe sea conditions. For this purpose, typical sea spectra of the North Sea were used. For reason sea comparison, long term stress computations are performed and stress results presented also for the sea conditions to be expected for the Gulf of Mexico and the Caribbean. A secondary purpose of the study was to obtain

short term predictions of the levels of wave induced motions and anchor cable forces in similarly severe sea conditions. However these results are not shown in this paper.

The general procedures which were utilized in performing the study comprise the following four parts:

(1) The wave induced motions and dynamic force distributions on the structure are computed for unit regular waves.

(2) A space frame analysis of the response of the structure to these unit regular wave forces is carried out.

(3) The principle of linear superposition is utilized in combining the regular wave structural

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response with appropriate random wave spectra to obtain the short term structural response to realistic wave conditions. The short term motions and anchor forces are predicted also in this manner.

(4) These short term structural responses are then combined with long term wave statistics to obtain predictions of fatigue life of the structure and long term probability distributions of maximum stress values. Each of these computations together with the result is described in more detail in the following sections of the paper.

2. Computation of Wave Induced Forces and Motions

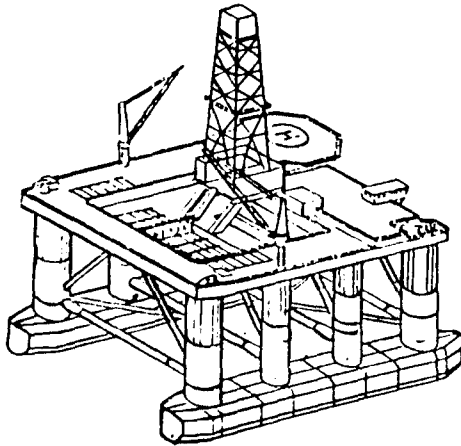


Fig.1 Aker H-3 drilling semi-submersible

For this study, Aker H-3 semi-submersible drilling platform which has two pontoons and eight columns was adopted because this design was most popular and proven. The sketch of the platform is shown in Fig. 1. The summary data are given in Table 1.

Linearized procedures are employed throughout for computing the response of the structure to waves. By making an assumption of linearity, we are able to employ linear superposition in obtaining responses to irregular seas.

Detailed description of the procedures used in predicting the wave forces and structural response are given in [1,2]*.

Table 1 Summary data for an Aker H-3 drilling semi-submersible

Operating displacement	20630 tonnes
Pontoon displacement	14060 tonnes
Length over all(Pontoons)	108.2m
Breadth moulded	67.4m
Elevation upper peck	39.6m
Elevation lower deck	36.9m
Large column diameter	7.9m
Small column diameter	5.8m
Pontoon cross section	11 × 6.7m ²
Deck area(length × breadth)	62.9 × 61m ²
Max. operating draft	21.3m
Storm draft	18.3m
Transit draft	6.4m
Storm air gap	18.3m
Heave period	21.9 s
Transit payload	1500 tonnes
Operating payload(column and deck)	1900 tonnes
Max. water depth	200m

2.1 Wave Force Computation

For the wave force computation, the immersed part of the structure is assumed to consist of a space frame assemblage of slender, cylindrical members. The separation distances between adjacent members are assumed to be sufficiently great that hydrodynamic flow interference effects between members are negligible. Thus, the forces on an individual cylindrical member are computed as though that member alone is exposed to the fluid flow field of the waves and platform motion. Linearization further permits us to separate the forces resulting from the wave motion from those resulting from the platform motion. The total wave force is made up of three components: The pressure or Froude-Krylov force, the added mass force due to water acceleration and drag force due to velocity relative to the member. An added mass coefficient of 1.0 is theoretical value for circular cylinders and the drag coefficient is appropriate for the diameters and flow velocities encountered. For noncircular members representing the pontoons values appropriate for the cross-sectional shape were used.

* Numbers in bracket designate references at end of paper.

2.2 Motion-dependent Forces

There will be additional hydrodynamic forces due to the motion of the platform and, as noted above, linearization permits us to separate these from the forces due to wave motion. Since the platform motion is an unknown in the problem, these forces appear on the opposite side of the equations of motion from the wave forces. They are computed on the basis of similar assumptions regarding the sparse geometry of the structure.

2.3 Hydrostatic Forces

For a member which projects through the water surface, there will be an additional force due to the wave and motion induced variations in immersed volume of the member.

2.4 Anchor Line Forces

The platform is held in position by means of an array of eight anchors, two at each corner. The anchor cables exert forces on the platform as a result of the platform motion which are, in general, nonlinear functions of the motion. For the present computations, equivalent linear springs were used to simulate the effects of the anchors.

The anchor cable force depends on the geometry of the anchor line (length, point of attachment, water depth), the weight and elasticity of the line, and the initial tension. A single nominal value of pretension was used for the computations of structural response since the effects of changes in anchoring characteristics are expected to be secondary in this case.

2.5 Solution of Equation of Motion

The force components just described are added together for all members which make up the structure, and equated to the mass times acceleration of the structure by Newton's second law of motion. The resulting equations of motion for the six degree of freedom rigid-body motion of the platform comprise a set of six, second order linear differential equations in which the unknowns are

the motion amplitudes of the structure's center of gravity. The exciting forces are computed for regular waves of unit amplitude and, in general, depend upon wave frequency. The resulting motions obtained by solving this set of equations may, therefore, be thought of as frequency-dependent motion transfer functions.

2.6 Nodal Equivalent Forces

A principal objective of this part of the computation is to obtain a system of lumped nodal forces for input of a finite element analysis of the structural response. In addition to the wave and motion induced hydrodynamic forces, the hydrostatic, and anchor cable forces described above, we must include the inertia forces due to the mass of the structure itself. These forces may all be computed once the equations of motion have been solved.

In general, the equivalent lumped nodal forces are computed for each member by determining the concentrated forces at each end of the member which would have the same resultant force and moment as the actual distributed forces. These forces are then treated as external loads applied at the structural nodes to which the ends of the member attached. For non-structural masses such as the platform's payload equivalent lumped masses are placed at appropriate nodes and the resultant acceleration forces computed. The inertia forces computed in this way are consistent with the mass moments of inertia used in the equations of motion so that a self-equilibrating force system results.

3. Finite Element Analysis

3.1 Node and Member Mesh

For structural purposes, the platform is represented as space frame assembled from beam members joined at nodes. In the previous section we described the calculation of the nodal force distribution which serves as input to the structural analysis of this space frame. The node mesh used for this purpose was chosen to provide

accurate modelling of the structural behavior, especially in the vicinity of the pontoons, columns, and transverse trusses. All of these components were represented by beam elements having structural properties computed from construction drawing of the members.

The deck structure of the H-3 platform consists of stiffened plate fields supported by two longitudinal box girders at the sides, and four deep transverse girders in way of the main columns. Care was taken to properly model the stiffness of the longitudinal and transverse girders since they provide the upper end support for the columns and trusses. The stiffened plate deck was represented as an equivalent grid of beam elements. The node mesh and member nomenclature at frame 30 is shown in Fig. 2. Node designations are given in parenthesis and member designations are circled. Members of finite cross sectional dimensions are represented here by line beam elements. Thus, in some cases, the length of the theoretical member will differ from the real member.

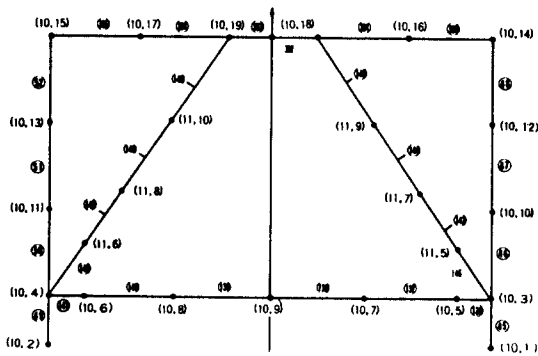


Fig. 2 Member arrangement at frame 30 (looking forward)

3.2 Boundary Conditions

As is the case in all finite element computations it is necessary to insert six artificial boundary conditions to insure rigid body fixity of the entire structure. In the present case, these fixed nodes were located at the upper corners of the main deck in such a way that any slight residual forces would have minimum effect on stresses in the areas of interest. While, in principle, the nodal forces were

self-equilibrating, there was a slight discrepancy in the manner in which hydrostatic forces were dealt with which result in small boundary forces. The magnitudes of these were checked and found to insignificant.

3.3 Load Cases

The force calculations produce one nodal force consisting of in phase and quadrature components for each wave frequency. These were computed for a total of eleven frequencies corresponding to wave periods of four to twenty-four seconds in order to cover the practical range of ocean waves and platform response. These frequencies were repeated for waves approaching the platform from directly ahead, 180°, bow quartering, 135°, and abeam 90°. In addition calculations were made for the static case of the platform afloat in calm water. The weight of the drill string of 750 tonnes was included in the static calculation but not in the dynamic cases. Otherwise, the weight distributions were similar for the two.

5.4 Mass Distribution

The mass distribution was taken from the report by [3]. In general, the mass per unit length is specified for the principal structural members. This was done for all beam members below the main deck, i.e. hulls, columns and trusses. Lumped masses were specified at prescribed nodes on the main deck to represent both structure and payload weights. Additional lumped masses were placed at certain nodes of the main columns and hulls to represent the weight payload located there. The mass distribution of the horizontal truss members included the weight of free flooding water which they contained.

3.5 Output

The output of the finite element calculations, as noted previously, are in phase and quadrature components of member internal reactions. There are six reactions at each end of each beam member, three forces and three moments. There are, therefore, twenty four outputs for each member

for each wave frequency. Recall that these have been computed for unit wave amplitude and may, therefore, be considered as transfer functions. After they have been computed, these reactions are stored in a computer file for later use in the short term and long term stress computations. These are described in the next sections.

4. Determination of Maximum Stresses to be Expected for 15 Years of Service

Short and long term stress results are computed on the basis of the transfer functions for the member internal reaction forces and of seaway statistics for the operating area considered. The computational procedure is only outlined briefly here. A more detailed description is given in Appendix which is based on the reasoning described by [4].

The total service period of the platform is subdivided into subperiods characterized by conditions of the sea environment with a given significant wave height, wave period and predominant wave direction. A short term analysis is performed on the basis of the corresponding sea spectrum and the appropriate transfer function for each such sea state, leading to, among others, the variance as well as the characteristic period of the seaway effect considered. Assuming a Rayleigh probability distribution for the response, the probability may be computed for the seaway effect to reach or exceed a given value.

Taking the probability of the different seastates to occur from wave statistics, the probability for a seaway effect to reach or exceed a specified value may be computed for the total service period considered. Conducting such computations for each structural member for systematically increasing specified values of response a cumulative distribution of response can be determined.

For the present study a period of 15 years of continuous service was considered, and calculations are made for the North Sea as well as for the area of the Gulf of Mexico and the Caribbean. Ocean

wave statistics were taken from [5].

Several characteristic elements of the tubular bracing system of the platform were selected for the analysis and transfer functions for stresses were determined from the transfer functions for member internal reaction forces according to the following reactions:

$$\sigma_x = \frac{A_x}{Q_x} + \sqrt{\left(\frac{M_y}{S_y}\right)^2 + \left(\frac{M_z}{S_z}\right)^2} \quad (1)$$

and

$$\sigma_x = SCF \cdot \left\{ \text{MAX} \left[\frac{Q_x}{A_x} + \frac{M_y}{S_y}, \left(\frac{Q_x}{A_x} + \frac{M_z}{S_z} \right) \right] \right\} \quad (2)$$

where

- Q_x : axial member force
- M_y, M_z : bending moment about y and z axes
- A_x : cross sectional area
- S_y, S_z : section moduli about the y and z axes
- SCF : stress concentration factor for the hot spot stress.

Whereas the computations described in Section 3 were made assuming cylindrical characteristics for each beam element, the cross sectional properties used for the computation of stresses are determined on the basis of the actual geometry of the member at the cross section considered.

Long term stress computations are conducted for the bracing elements shown in Figs. 3 and 4. They are elements in the regions of tubular joints for a vertical bracing system between the heavy columns (truss I), for the vertical bracing system between the thinner columns (truss II) and for the horizontal K-braces connecting the heavy columns of truss I to the transverse of truss II.

As an example, the cumulative distribution of stresses according to equation (1) are shown for an element of the horizontal K-brace, element 119, in Fig. 5.

The long term stress distribution is evaluated with regards to two aspects: the maximum dynamics stresses caused by the seaway, and the fatigue life to be expected for the structural components considered in the proposed operating area.

The maximum stress amplitude is the one to be expected to be reached or exceeded in an element once during the service period considered for the

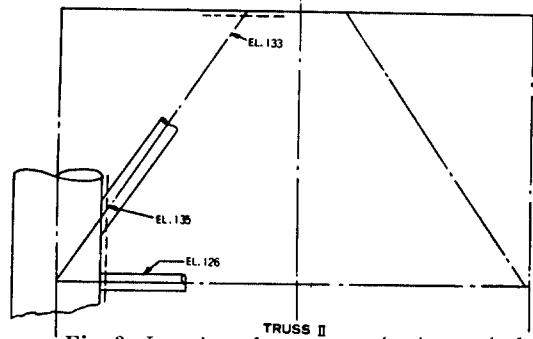
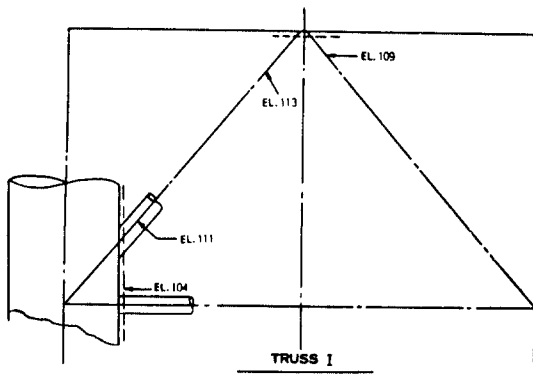


Fig. 3 Location of stress results in vertical K-braces

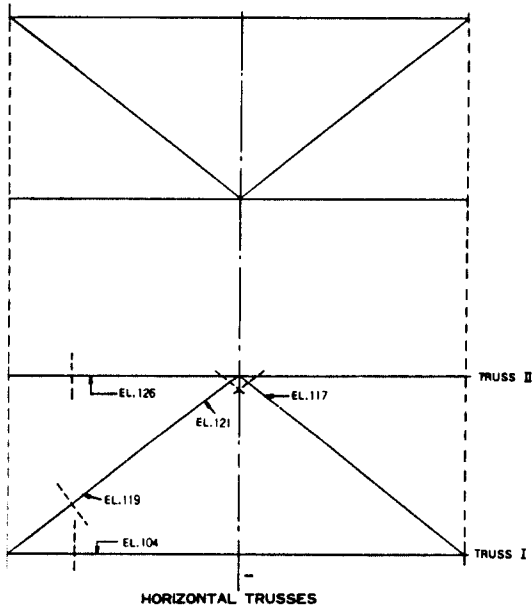


Fig. 4 Location of stress results in horizontal K-braces

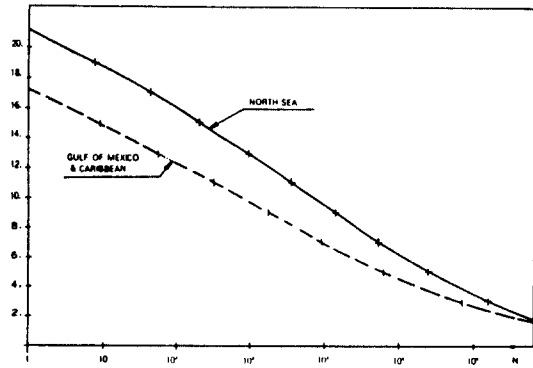


Fig. 5 Cumulative distribution of stresses for member 119 (operation time 15 years)

platform. Results of the calculation of maximum stresses are presented in Tables 2 and 3. The maximum stresses according to reaction (1), i.e. nominal member stresses, are given for 15 years operation in the North Sea and 15 years in the Gulf of Mexico and Caribbean. Together with the maximum stress amplitudes the stresses from the static calculation are also shown in the tables. In order to obtain the maximum total stresses, the maximum dynamic stresses would have to be added to the stresses due to the statics loading.

Table 2 Static and maximum dynamic nominal stresses in vertical K-braces for 15 years of service

Element	Static <i>ksi kp/cm²</i>		North sea <i>ksi kp/cm²</i>		Culf/Mexico <i>ksi kp/cm²</i>	
Truss I						
104	6.71	470.	16.52	1160.	13.35	940.
111	6.97	490.	14.82	1040.	13.47	950.
113	7.10	500.	7.92	555.	6.83	480.
109	7.25	510.	8.16	575.	6.85	480.
Truss II						
126	7.14	500.	13.98	985.	11.41	800.
133	10.38	730.	10.67	750.	9.58	675.
135	9.99	705.	4.98	350.	4.70	330.

Table 3 Static and maximum dynamic nominal stresses in horizontal K-braces for 15 years of service.

Element	Static <i>ksi kp/cm²</i>		North sea <i>ksi kp/cm²</i>		Gulf/Mexico <i>ksi kp/cm²</i>	
119	7.97	560.	21.33	1500.	17.39	1225.
121	.72	50.	17.87	1255.	14.67	1035.
117	.93	65.	19.42	1365.	15.82	1110.

5. The Fatigue Analysis

The fatigue life of the structural components considered in the analysis is determined on the basis of the cumulative stress distribution for the component using the linear Palmgren-Miner cumulative damage hypothesis.

Fatigue failure is assumed to occur in the component when the cumulative damage ratio,

$$D = \sum_i \frac{n_i}{N_i} \tag{3}$$

reaches or exceeds unity.

Here

n_i = number of stress cycles at given stress range,

N_i = Number of cycles at that stress range expected to cause fatigue damage in the material without previous load history. (material fatigue data)

A more detailed description of the calculation procedure is given in Appendix.

For the determination of N_i , two of the proposed fatigue design curves for high cycle loading described by [6] are used.

The "BS 153-F/3" curve is a rather conservative basis for the fatigue analysis, especially so with regards to the high cycle lower stress ranges. Results from the used of this curve for the construction under consideration may be expected to largely under estimate its real fatigue life. They are presented here for reasons of comparison and may serve as guide lines for the selection of inspection intervals.

The "Lloyds improved" design curve for welded joints, the second curve used, is more appropriate for the conditions of platform analysed. For reasons of simplification it is being used here in a form modified slightly towards the conservative side for stress ranges below 5 ksi, as shown in Fig. 6.

The results of an analysis of this kind for the nominal stresses in the bracing elements considered are presented in Table 4. Stresses are determined according to the force-stress relation (1).

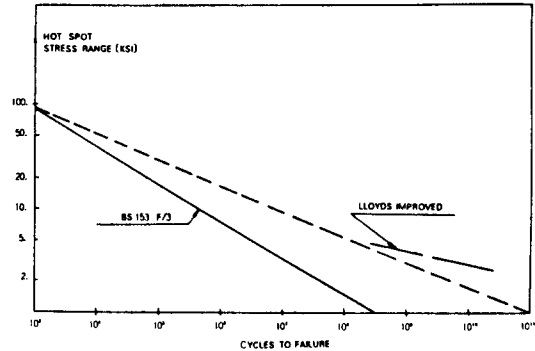


Fig. 6 Design curves for fatigue analysis of welded joints [Ref. 6]

Table 4 Accumulated damage for 15 years of service for nominal stresses according to eq. (1).

Element	North Sea		Gulf of Mexico		
	BS 153 F/3	LLOYDS	BS 153 F/3	LLOYDS	
Truss I	104	.282	.0043	.162	.0017
	111	.110	.0013	.057	.0005
	113	.032	.0002	.014	.0001
	109	.038	.0003	.018	.0001
Truss II	126	.164	.0022	.089	.0008
	133	.040	.0004	.018	.0001
	135	.003	.00002	.001	.00001
Hor. brace	119	.587	.0131	.328	.0047
	121	.345	.0062	.189	.0022
	117	.436	.0086	.241	.0031

Both design curves used are defined for hot spot stress or strain ranges. Some knowledge about the stress concentration factors for the hot spots of the joints would be necessary, in order to be able to conclude from the nominal stresses in the bracing structure about the hot spot stresses to go with them. The stress concentration factor can be determined either by full scale or model measurement in the joint region under consideration or by the way of a detailed joint analysis by the finite element method. In the absence of such data about the actual conditions a stress concentration factor of SCF=3.0 is being used for all joints in the present analysis in connection with the force-stress relation (2). The results are shown in Table 5.

The results on the basis of the more conservative

Table 5 Accumulated fatigue damage for 15 years of service (hot spot stress concentration factor SCF=3.0)

Element	North Sea		Gulf of Mexico	
	BS 153 F/3	LLOYDS	BS 153 F/3	LLOYDS
Truss I	104	4.69 .255	2.79 .097	
	111	2.20 .093	1.24 .034	
	113	0.68 .015	0.39 .006	
	109	0.83 0.21	0.48 .008	
Truss II	126	3.25 .156	1.84 .057	
	133	0.87 .025	0.48 .009	
	135	0.14 .001	0.08 .001	
Hor. brace	119	10.84 .930	6.08 .326	
	121	7.07 .510	3.92 .176	
	117	8.78 .691	4.91 .241	

“BS 153 F/3” curve may, as was stated previously, be used for the determination of inspection intervals for the joints.

6. Conclusions

The results for accumulated damage and fatigue computations are shown in Table 4 and 5 respectively. According to these results a life free of fatigue failure of a little more than 16 years of continuous service in the North Sea may be expected for the most highly stressed location, the connection of the horizontal K-braces to the vertical columns. on the basis of the “Lloyds improved” design curve.

It is stressed once again, that a stress concentration factor of three has been chosen more or

less arbitrarily for the computation of hot spot stresses. The results may therefore, as well as for other reasons of uncertainty in a linear cumulative damage analysis as described, be understood only as approximations of the expected life history of the structure. They should be used mainly for comparisons as well as indications of relative conditions of stressing in the different components and regions of the structure.

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Appendix

Determination of Extreme Stresses and Fatigue Life

(1) Short Term Response

For a given seastate characterized by the significant wave height H_v , the period T_v and the predominant wave direction μ_{vk} , the probability of any response h to exceed an arbitrarily given magnitude h_y can (assuming Rayleigh distribution for the response) be expressed by

$$P(h \geq h_y) = \exp\left(-\frac{h_y^2}{2m_{y0}}\right)$$

where

$m_{y0} = \int_0^\infty \int_0^{2\pi} E_y(\omega, \mu) d\omega d\mu$ is the variance of the seaway effect(response)

$E_y(\omega, \mu) = Y^2(\omega, \mu) \cdot E(\omega, \mu)$ response spectrum

$E(\omega, \mu)$ = directional wave spectrum

$Y(\omega, \mu)$ = transfer function

ω = circular frequency

μ = direction of encounter

If the platform experiences n_{ijk} cycles during short term time at the given seastate, it'll experience

$$\bar{n}_{hijk} = P(h \geq h_y) \cdot n_{ijk}$$

cycles with a response amplitude larger than or equal to h_y . The ISSC directional sea spectrum is used for the computations presented in this paper:

$$E(\omega, \mu) = 173 H_v^2 / T_v^4 \cdot \omega^{-5} \exp\left[-\frac{5}{4} \frac{553}{\omega^4 T_v^4}\right] \cdot \frac{2}{\pi} \cos^2(\mu - \mu_v); |\mu - \mu_v| < \frac{\pi}{2}$$

(2) Long Term Response

The probability for a seastate, specified by H_{vi} , T_{vi} , and μ_{vk} , to occur in a given area P_{ijk} , is taken from sea observations: e. g. cumulative distribution of wave heights at different wave periods, here taken from tabular representation in [5].

Probability for different wave directions to occur is taken here as the same for all directions: i. e.

$$\text{for 8 directions } P_k = \frac{1}{8}$$

$$\text{hence: } P_{ijk} = \frac{1}{8} P_{ij}$$

with P_{ij} taken from wave data as stated above.

During a given seastate the response will exceed the amplitude h_y a number of times

$$N_{hijk} = P_{ijk} \cdot n_{hijk} = P_{ijk} \cdot \exp\left(-\frac{h_y^2}{2m_{y0}}\right) \cdot n_\Lambda$$

where n_Λ is the total number over all intervals into which the service period was subdivided we get

$$N_h = \sum_i \sum_j \sum_k N_{hijk}$$

or, expressing n_Λ in terms of service period T_s and characteristic periods of the seaway effect T_{yijk} ,

$$N_h = \sum_i \sum_j \sum_k P_{ijk} \exp\left(-\frac{h_y^2}{2m_{y0}}\right) \cdot T_s / T_{yijk}$$

Here T_{yijk} , P_{ijk} , and m_{y0} are functions of the seastate regarded. Varying h_y , the amplitude to be reached or exceeded, over a wide enough range we get

$$N_h = N_h(h_y)$$

the cumulative distribution of the response i. e. stress.

(3) Fatigue Analysis

Using the cumulative distributions for several critical locations in the structure we can estimate

(i) The maximum value to be expected to be reached or exceeded once in the service period of time and service area

(ii) The fatigue life.

A. The extremes:

The maximum values must be lower than or equal to allowable stresses specified for the material, considering possible stress concentration effects and taking account of the extent of danger to the total structure caused by local failure of the structural component considered.

B. The fatigue life

The cumulative distribution curve of the response is subdivided into several regions.

The response amplitude (stress) may be expected to lie between h_{y_i} and $h_{x_{i+1}}$.

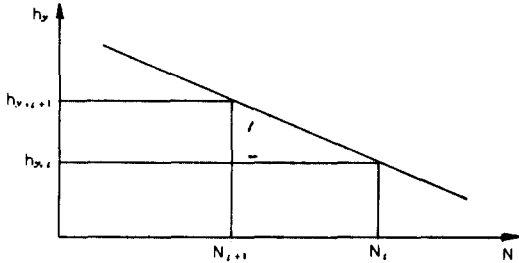


Fig. 7

The fatigue life of the undamaged material at this stress level, $\frac{h_{y,i} + h_{y,i+1}}{2}$, is taken from material data, and is \bar{N}_i cycles. In the present study fatigue data described by [6] for tubular joints is used.

To take account of the influence of non-zero mean stress on the fatigue life of a structural component, appropriate fatigue data for the material

would have to be used instead of the S-N curves described above, to determine the values \bar{N} .

The contribution to damage of stresses at the given stress level is

$$d_i = \frac{\Delta N_i}{N_i} = \frac{N_i - N_{i+1}}{N_i}$$

The total damage experienced during the service period considered will be, according to the linearly accumulative damage theory

$$D = \sum_i d_i = \sum_j \frac{\Delta N_j}{N_j}$$

D may be smaller, equal or larger than 1.

The expected fail safe life will then be

$$\text{Life} = \frac{(\text{service period considered})}{D}$$

If $D > 1$ the structure may not be considered fail safe for the total service period considered and closer spaced inspection intervals should be recommended.

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