PROCEDURAL SPECIFICATION

DESIGN OF STEEL SUBSTRUCTURES FOR FIXED OFFSHORE PLATFORMS (AMENDMENTS/SUPPLEMENTS TO API RP 2A-LRFD)

DEP 37.19.00.30-Gen.

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DESIGN AND ENGINEERING PRACTICE

USED BY

COMPANIES OF THE ROYAL DUTCH/SHELL GROUP

PREFACE

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PART I INTRODUCTION

1.1 SCOPE

This new DEP provides guidance on engineering principles and practices relating to the design of steel substructures for fixed offshore platforms. Guidance is also provided on issues relating to planning and construction to the extent that these are relevant to design.

This DEP is based on API RP 2A-LRFD. Part II of this DEP amends, supplements and replaces various clauses/sections of API RP 2A-LRFD. Part III provides corrections to typographical errors in API RP 2A-LRFD.

This DEP does not cover design of the topside structure. This DEP is also limited to structures whose extreme response is predominantly quasi-static.

1.2 DISTRIBUTION, APPLICABILITY AND REGULATORY CONSIDERATIONS

Unless otherwise authorised by SIPM, the distribution of this DEP is confined to companies forming part of the Royal Dutch/Shell Group or managed by a Group company, and to Contractors nominated by them (i.e. the distribution code is "C", as described in DEP 00.00.05.05 Gen.).

This DEP is intended for use by Group companies involved in the design, construction and operation of fixed offshore platforms, and by Contractors working on their behalf.

If national and/or local regulations exist in which some of the requirements may be more stringent than in this DEP the Principal and/or the Contractor shall determine by careful scrutiny which of the requirements are the more stringent and which combination of requirements will be acceptable as regards safety, environmental, economic and legal aspects. In all cases the Contractor shall inform the Principal of any deviation from the requirements of this DEP which is considered to be necessary in order to comply with national and/or local regulations. The Principal may then negotiate with the Authorities concerned with the object of obtaining agreement to follow this DEP as closely as possible.

1.3 DEFINITIONS

1.3.1 General definitions

The **Contractor** is the party which carries out all or part of the design, engineering, procurement, construction, commissioning or management of a project or operation of a facility. The Principal may undertake all or part of the duties of the Contractor.

The **Principal** is the party which initiates the project and ultimately pays for its design and construction. The Principal will generally specify the technical requirements. The Principal may also include an agent or consultant authorised to act for, and on behalf of, the Principal.

The word **shall** indicates a requirement.

The word **should** indicates a recommendation.

1.3.2 Specific definitions

A **fixed platform** is a bottom founded installation which supports facilities above the water surface and is not normally re-located during its service life.

Owner is a title used in API RP 2A-LRFD and in Part II of this DEP, which has the same meaning as Principal.

The **substructure** consists of a welded tubular spaceframe and foundation piles. The piles permanently anchor the platform to the sea floor and carry both lateral and vertical loads. The substructure normally supports a topside structure.

The **topside structure** consists of trusses and decks which support operational and other loads.

1.4 CROSS-REFERENCES

The clause/section numbering used in Part II and III of this DEP corresponds with that used in API RP 2A-LRFD. Other documents referenced by this DEP are listed in Part IV.

PART II AMENDMENTS/SUPPLEMENTS TO API RP 2A-LRFD

SECTION A PLANNING

A.2 Platform Types

A.2.1 Fixed Platforms

Replace clauses A.2.1.1 and A.2.1.2 with:

A.2.1.1 Jacket Structures

A jacket is a spaceframe structure in which the piles are inserted through the legs and are connected to the legs at the top.

A.2.1.2 Tower Structures

A tower is a spaceframe structure in which the piles are inserted through and attached to sleeves at the base of the structure.

A.3 OPERATIONAL CONSIDERATIONS

A.3.5 Access and Auxiliary Systems

Add to this clause:

Installation, maintenance and inspection requirements should be considered in determining access platform and walkway locations at the top of the substructure. Access should be provided to the water level for sea escape and to facilitate self rescue in man-overboard situations.

A.3.8 Wells and Risers

Add to this clause:

Wells and risers should be positioned to minimise exposure to accidental damage due to vessel collision or other causes.

Add new clause:

A.3.13 Accidental Events

The design should account for the possibility of accidental events including vessel collision, accidental flooding, dropped objects, fire and explosion. Criteria for designing against accidental events should take account of the operational conditions, and the type, function, criticality, service life and location of the platform.

Add new clause:

A.3.14 Abandonment

Consideration should be given to the requirements for abandonment of the platform at the end of its operating life. Specific design or analysis for abandonment is not generally required; however, if removal or dismantling will be necessary it is desirable that the design should make provision for this. For example, consideration should be given to designing installation aids and lifting points so that they can be re-used when the platform is removed or dismantled.

A.4 ENVIRONMENTAL CONSIDERATIONS

Add new clause:

A.4.9.7 Seabed Subsidence

The nature of the soil conditions, reservoir and extraction processes should be investigated to establish whether subsidence of the seabed is likely to occur during the field life. Where subsidence is a possibility it should be accounted for in design.

A.7 REGULATIONS

Replace this section with:

A country will typically have its own set of regulations concerning offshore activities. These should be considered when designing offshore platforms for the waters of that country.

Add new section:

A.8 QUALITY ASSURANCE

A.8.1 Application of Quality Assurance

Quality systems conforming to ISO 9000 or an equivalent standard should be established and used for the design by the Contractor. Independent audits of the quality systems should be commissioned by the Contractor and performed prior to and during the design to ensure adherence to the stipulated requirements.

A.8.2 Responsibilities

Responsibilities should be clearly defined and documented for both the Contractor's and the Owner's organisations, covering all activities and their interfaces during all phases of development. Scopes of individual responsibility should be made known to all concerned.

A.8.3 Documentation

The full extent of all drawings, specifications, reports and documents to be prepared during design and the responsibilities for their preparation and approval should be established. The responsibilities for preparing and maintaining as-built drawings, and for maintaining up-to-date design documentation during construction and during the service life, should be established by the Owner.

A.8.4 Software

For design and analyses, only validated software should be used.

Add new section:

A.9 DIMENSIONAL SYSTEM

A.9.1 Units

All calculations, dimensions and weights should be in Systeme Internationale (SI) units (see DEP 00.00.20.10-Gen.).

A.9.2 Reference System

A consistent dimensional reference system including orientation of Platform North and angle to True North should be established for the platform and used for all calculations, drawings, and weight control computations. The horizontal reference axes on drawings should be the transverse and longitudinal centrelines at the level of lowest astronomical tide (LAT). All vertical elevations on drawings should be relative to LAT.

SECTION B DESIGN REQUIREMENTS

B.2 DESIGN FOR IN-PLACE CONDITIONS

Add to this section:

Combinations of the loads described in Sections C.2 to C.6 should be selected in a rational manner. These combinations should be such that the most onerous conditions for all structural components are covered.

B.3 DESIGN FOR CONSTRUCTION CONDITIONS

Add to this section:

Methods for fabrication, assembly and load-out of the structure should be considered during design in order to identify potential difficulties and limitations for available equipment and facilities. Load conditions for which partially fabricated components of the structure and the assembled structure as a whole need to be checked should be established. Fabrication and assembly tolerances assumed in design should be compatible with those specified in the construction specifications and should be achievable during construction.

B.6 REDUNDANCY

Replace second paragraph with:

The resistance and load factors in API RP 2A-LRFD have been verified for conventionally configured pile founded jacket or tower spaceframe structures. These configurations generally have three or more straight legs, several levels of triangulated plan framing, with all panels in the main 'vertical' frames being braced with diagonal members between plan levels. The factors have not been verified for different types of structural configurations such as mono-columns, braced mono-columns or other novel configurations. Modifications of these factors, or development of new factors suitable for a different type of structural framing should be based on a rational system reliability analysis. See also Section Q.

B.7 CORROSION PROTECTION

Replace this section by:

The corrosion protection system should be designed in accordance with the provisions of Section H.7.

Add new section:

B.9 EFFICIENCY

The structure should be designed to provide for efficient and cost effective fabrication, installation, operation and abandonment. The structural design should be optimised in order to minimize total life cycle cost. The requirements for in-service above water and under water inspection and maintenance should be optimised to achieve cost effective operation during the service life.

SECTION C LOADS

C.2 GRAVITY LOADS

C.2.4 Live Load 1, L_1

Replace first sentence with:

Live Load 1 includes the weight of consumable supplies and fluids in pipes, tanks and stores, the weight of transportable vessels and containers used for delivering supplies, and the personnel on the platform. Where appropriate, the weight of ice should be included in L_1. The weight of scaffolding or other temporary access systems used during operations and maintenance of the platform should also be included in L_1.

C.2.5 Live Load 2, L_2

Add to this clause:

The additional weight of liquids used for testing of vessels and pipes is included in Live Load 2.

Add new clause:

C.2.10 Weight Estimates

Weights used in design should be based on a rational weight estimating procedure which takes account of potential changes during design and construction. Allowances should be included for potential future changes which may result from operational or maintenance requirements in-service. These allowances should be developed and agreed with the operations department prior to adoption for design. Contingencies should also be considered for potential changes in platform function or facilities, for example due to unexpected reservoir performance or future changes/extensions to the field development. These contingencies should be agreed with management prior to adoption for design.

The dead and live loads used for design should be reconciled with the weights derived using the weight estimating procedure.

C.3 WIND, WAVE AND CURRENT LOAD

C.3.1.2 Extreme Wind, Wave and Current Load, W_e

Replace this clause with:

W_e is the force applied to the structure due to the combined action of extreme wind, current and waves. Where information on the joint occurrence of wind, current and waves is not available, W_e should be calculated using the 100 year return period wind, the 100 year return period current and the 100 year return period wave conditions; the 100 year extreme magnitude for each phenomenon being determined independently. Where information on joint occurrence is available, it is recommended that it should be used to justify alternative combinations which give more appropriate overall load levels, as described in Comm. C.3.1.2.

C.3.1.3 Direction of Wind, Wave and Current

Add to this clause:

Normally, the directions of the extreme wind, wave and current should be assumed to coincide for determining W_e. However, in some locations, geographic features provide fetch limitations on wave heights from specific directions, or tidal or general circulation currents may be in a predominant direction. In these situations, if reliable data are available to derive different extreme wave and/or current magnitudes for different approach directions, then these may be used in design. Normally, the extreme wave and current magnitudes should be combined with the extreme wind specified irrespective of approach direction. Where directional wave and/or current magnitudes are used the possibility of the installed orientation of the platform deviating from the design orientation, due to installation tolerances, should be accounted for.

C.3.2 Static Wave Analysis

Add after third sentence:

For other areas, wave, current and water depth data for the platform location should be provided by the Owner. These should be determined in accordance with the guidance given in Section A.4.

C.3.2.2 Two - Dimensional Wave Kinematics

Add to this clause:

Breaking waves may occur in storm conditions in shallow water and when the ratio of wave height to water depth is large. The type of breaking wave (e.g. plunging or spilling) and the wave height / water depth limits are dependent on local environmental and topographical conditions such as the presence of current and seabed slope.

C.3.2.12 Local Member Design

Add to this clause:

For the local design of smaller members adjacent to larger members, the kinematics used should take account of the effect of flow enhancement around the larger member.

Add new clause:

C.3.2.13 Recommended Static Wave Analysis

Sections C.3.2.1 to C.3.2.8 address the calculation of wave kinematics and selection of force coefficients for establishing the deterministic static wave load based on a traditional regular periodic wave theory. An alternative wave theory, called NEWWAVE has been developed within the Group to provide wave kinematics which more accurately represent the maximum conditions in a realistic complex random sea. NEWWAVE theory, used in conjunction with realistic hydrodynamic force coefficients, has been validated for calculating extreme wave loads for drag dominant structures for which the maximum waveload occurs with the wave crest positioned at the structure. For such structures it is recommended to use NEWWAVE instead of the provisions of Sections C.3.2.1 to C3.2.8 to calculate the deterministic static wave load for design. NEWWAVE is valid in the range of water depths where the Stokes V order wave theory is valid. EP 94-0161 gives a detailed description of the application of NEWWAVE to spaceframe substructures. See also Commentary C.3.2.13

C.3.3 Extreme Wave Dynamic Analysis

C.3.3.1 Extreme Inertial Load D_n

Replace second paragraph with:

For platforms with sufficiently short natural periods, dynamic amplification is negligible. D_n can normally be neglected if the fundamental natural periods of the structure are less than three seconds and less than one fifth of the peak period of the design wave spectrum.

The provisions of this Section for the calculation and use of the extreme inertial load are only applicable to structures which exhibit moderate levels of dynamic response. A moderate level of dynamic response is defined such that the inertial part of the maximum global response D_n (e.g. base shear or overturning moment) is less than one half of the corresponding component due to W_e.

C.3.3.2.1 Dynamic Analysis Methods

Replace this clause with:

The wave and other time varying loads should be realistic representations of the frequency content of the loading. Random time domain methods allow the frequency content and the nonlinear characteristics of the extreme loading and response to be explicitly modelled. Frequency domain techniques provide a computationally efficient means to represent the frequency content providing appropriate linearisations of the extreme loading (drag wave load and free surface inundation) and response are used. EP 87-0170 provides a review of alternative dynamic analysis methods and gives guidelines on their application. EP 93-2525 describes specific simplified techniques which may be used to estimate extreme response accounting for the non linearity due to drag wave loading.

C.3.3.3 Member Design

Add to the end of this clause:

Guidance on the representation of the distribution of the extreme inertial load is given in EP 93-2525.

C.3.4 Wind Force

C.3.4.2.5 Spatial Coherence

Replace last three sentences with:

The one minute sustained wind is appropriate for calculating total static superstructure windloads applied in conjunction with maximum wave and current loads.

C.3.4.4 Local Wind Force Considerations

Add to this clause:

All slender members exposed to winds should be investigated for the possibility of vibration due to periodic vortex shedding as discussed in Comm C.3.2.12.

C.3.4.5 Shape Coefficients

Add to this clause:

Further values for shape coefficients are given in DNV.

C.3.4.6 Shielding Coefficients

Add to this clause:

Guidance on values for shielding coefficients is given in DNV.

C.3.5.2 Current Associated with Waves

Replace last sentence with:

Where there is adequate and reliable data available on currents and waves, their magnitudes and directions may be determined in combination in accordance with C.3.1.2 and C.3.1.3.

C.3.6 Deck Clearance

Replace this clause with:

C.3.6 Platform Elevation

C.3.6.1 Substructure Elevation

Variations and uncertainties in the water depth and platform elevation should be considered when evaluating wave and current forces on the platform and when designing items sensitive to the final elevation of the structure. Account should be taken of variations in water depth due to astronomical tide and storm surge, and allowances should be made for uncertainties in the methods used to determine the water depth. The effects of seabed subsidence and uncertainties on the penetration of the structure into the seabed should be considered when evaluating the range of final platform elevations. For extreme conditions the range of water depths between the maximum and minimum associated with the 100 year storm should be investigated and the upper and lower limits of platform elevation should be considered when evaluating local and global loading on the platform. For operating conditions the mean sea level should be used.

C.3.6.2 Deck Elevation

Large forces may result when waves strike the platform deck or the equipment thereon. An air gap of at least 1.5 metres should be provided above the maximum wave crest elevation in order to determine the minimum acceptable level to avoid waves striking the deck. The maximum wave crest elevation should include the 100 year return period maximum wave plus the associated storm surge and astronomical tide, in addition to allowances for water depth uncertainty, seabed penetration and subsidence. Components or equipment placed within or below the air gap should be designed for wave impact and loading.

Reliability studies indicate that the very large forces which occur if the wave crest hits the deck frequently control the overall reliability of the structure. For this reason the crest elevation of the maximum wave with a return period equal to the reciprocal of the target annual reliability of the structure should be examined, and if necessary the air gap increased, to ensure that excessive wave forces on the deck do not result. This crest elevation should be determined using expected values for water depth, tide, surge, settlement and subsidence. Use of the maximum values for these parameters would be conservative beyond the intent of this requirement.

C.4 EARTHQUAKE LOADS

C.4.1.2 Evaluation of Seismic Activity

Add after first paragraph:

The site specific assessment should evaluate the intensity of seismic activity for appropriate return periods. The return period of the ground motions used for the strength level and ductility level events should be selected by the Owner based on an evaluation of the acceptable risks and consequences for the type of operation intended.

C.4.1.3 Evaluation for Zones of Low Seismic Activity

Delete this clause.

C.4.2.4 Response Analysis

Add to the first paragraph:

The response analysis should include ground motions in vertical and horizontal directions.

The methods used for evaluating and combining responses in both horizontal directions should be compatible with the approach used to include directionality in the derivation of the input ground motion spectra.

C.4.2.5 Response Assessment

Add to this clause:

The possibility of soil liquefaction occurring during the strength level earthquake should be evaluated. Where liquefaction is a possibility it should be accounted for when determining the stiffness and capacity of the pile foundation.

C.4.4.2 Deck Appurtenances and Equipment

Delete this clause.

C.5 FABRICATION AND INSTALLATION FORCES

C.5.2 Dynamic Effects

Replace last paragraph with:

Unless special investigations are performed to substantiate lower values the following minimum factors should be applied to account for dynamic effects during lifting:

- 1.30 for offshore lifts,
- 1.15 for lifts onshore or in sheltered waters.

C.5.3 Load Factors

Add after last paragraph:

During installation operations the behaviour of the structure and the magnitude and distribution of the forces in the structure may be sensitive to the position of the centre of gravity. Allowances should be made for uncertainties and potential changes in the centre of gravity position during design.

C.5.5 Lifting Forces

C.5.5.1 General

Add after first paragraph:

Lifting forces depend on the nature of the object being lifted (size, weight, stiffness, etc.), particulars of the lifting equipment (crane stiffness, hook speed, sling arrangement, etc.) and on the conditions and procedures under which the lift is made (e.g. onshore, sheltered inshore or exposed offshore). The general guidance on lifting loads in this Section and Sections C.5.2 and C.5.3 assumes typical lifts in reasonably controlled conditions (see Commentary).

Critical offshore lifting operations such as those involving heavy loads approaching the capacity of the lifting equipment, those involving transfer of the lifted object to or from a transportation barge at an exposed location, or those using dual crane operations should be subject to special investigation. This should investigate the dynamic lifting forces in cranes, rigging, lifting points and in the lifted structure, and the impact forces during lift-off and setting down of the load and should account for the limiting weather conditions in which the operation may proceed. Frequency domain and/or time domain techniques may be used to analyse the relative motions of the lift vessel, the transportation barge and the structure to determine the magnitude of these forces.

Specific lifting criteria used for final design should be determined in conjunction with the installation contractor.

C.5.6.2 Horizontal Movement onto Barge

Add before last sentence:

The effect of the forces on the structure during loadout should be investigated taking account of the tolerances on the levels of supports on the land and the barge. The effect of twisting of the structure due to differential horizontal jacking or winching during loadout should be considered.

C.5.7 Transportation Forces

C.5.7.2 Environmental Criteria

Add to this clause:

The environmental conditions used in determining the motions of the tow should be established by the Owner (in consultation with the marine warranty surveyor if appropriate) taking account of the expected tow route and season. For long ocean tows where the structure and barge are unmanned the extreme environmental conditions are typically selected to have a probability of exceedence during the tow duration in the range 1 to 10 percent. The specific value will depend on an evaluation of acceptable risks and consequences. For short duration tows, the environmental conditions should generally have a return period not less than 1 year for the season in which the tow takes place.

C.5.7.3 Determination of Forces

Add to first paragraph:

These should use spectral analysis techniques to account for the spread of wave energy in tow storm conditions. Seastates with peak periods corresponding to the natural periods of the barge and peak periods corresponding to wave lengths which are critical in relation to the length of the barge should be considered as they may represent governing conditions. The analysis should correctly account for relative phasing between motion components (heave, pitch, roll, etc.) when combining responses.

Change first sentence in second paragraph to:

Beam, head and quartering wind and seas should be considered to determine maximum transportation forces in the structure, the barge and the seafastenings.

C.5.7.4 Other Considerations

Change second sentence to:

Submerged members should be investigated for slamming, buoyancy and collapse forces. These forces should be included in the local and overall design of the structure and seafastenings.

C.5.8 Launching Forces and Uprighting Forces

C.5.8.1 Launched Structures

Add to first paragraph:

The environmental conditions considered should be consistent with the limiting environmental conditions specified in the launch procedures.

C.5.9 Installation Foundation Forces

C.5.9.1 General

Add to this clause:

The loads determined following the provisions of this section should be used to check the foundation capacity of footings or mudmats against sliding or bearing failure, for checking the strength of mudmat components and for checking against overturning of the structure.

The structure should also be checked in a piled condition without topsides loads if placement of the deck does not follow shortly after substructure installation.

C.5.9.2 Environmental Conditions

Replace this clause with:

The wind, wave and current conditions used to determine the environmental loads should be established for the installation season and are typically selected to have a probability of exceedence during the exposure period in the range 1 to 10 percent. The specific value will depend on an evaluation of the acceptable risks and consequences. The exposure period used should take account of the installation sequence and the structure configuration for the various phases, considering relevant operations such as buoyancy tank removal, pile stabbing, driving and securing, conductor installation and deck placement. For short exposure durations, the seasonal environmental conditions should generally have a return period not less than 1 year unless restrictions are imposed in installation procedures to limit the conditions in which operations may proceed.

Add new clause:

C.5.11 Fabrication Forces

The effects on long slender members of vortex induced vibrations due to wind during fabrication should be investigated as discussed in Commentary C.3.2.12.

C.6 ACCIDENTAL loads

Replace this section with:

C.6.1 Vessel Collisions

C.6.1.1 General

The risk of vessel collisions with the structure should be evaluated, taking account of the nature of all vessel operations in the platform vicinity. Depending on the risk of collision and the consequences on the overall operability and structural integrity of the platform, a rigorous analysis of vessel impact conditions may be required following the provisions of clauses C.6.1.2 to C.6.1.4. Irrespective of whether a rigorous analysis is required, vessel collisions should be provided for in the design by indirect means such as providing redundancy, avoiding weak elements in the structure (particularly at joints), selecting materials with sufficient toughness, and ensuring that critical components are not placed in vulnerable locations. Further guidance on the assessment of the risk of collisions and on methods of design and analysis for vessel impact is provided in EP 89-0230.

C.6.1.2 Collision Events

The energy level used for the impact analysis should account for the vessel mass and added mass and for its velocity of approach. Two energy levels should be considered for a rigorous impact analysis:

1. Low energy level, representing a serviceability condition, based on the type of vessel which would routinely approach alongside the platform (e.g. a supply boat) with a velocity representing normal manoeuvring of the vessel approaching, leaving or standing alongside the platform;

2. High energy level, representing an ultimate condition, based on the type of vessel which would operate in the platform vicinity, drifting out of control in the worst seastate in which it may operate close to the platform.

The impact zone should be established based on the dimensions and geometry of the structure and the vessel and should account for tidal ranges, operational seastate restrictions and motions of the vessel. Impact scenarios should be established representing bow, stern and beam-on impacts on the platform legs and braces as appropriate. Operational restrictions on vessel approach sectors may limit the exposure of some areas of the structure to impacts.

C.6.1.3 Collision Process

The energy absorbing mechanisms during the collision should be evaluated accounting for local member denting, elastic and plastic deflection of the impacted member, global elastic response of the structure, buckling and plastic deformation of any other members and denting of the ship. The forces generated during the collision should be evaluated ensuring that the energy dissipated by these mechanisms plus the kinetic energy gained by the structure (manifested as dynamic response) is equal to the kinetic energy lost by the vessel. Global non-linear analyses may be required in order to avoid over estimation of impact loads, particularly for high energy impact conditions. When the duration of the collision is large compared with the periods governing the motion, dynamic effects are not significant and the collision process can be analysed using quasi-static methods. If the duration of the collision is of the same order as, or shorter than the periods governing the motion, then dynamic effects may be significant and should be considered in the design.

C.6.1.4 Structural Performance

The failure sequence of the structure should be investigated considering the various deformation mechanisms involved to evaluate the extent of damage caused by the collision.

The design should be such that the damage caused by a low energy impact can be readily repaired or would not require repair. In the damaged condition following a low energy impact, the structure should be able to withstand environmental conditions which can be expected to occur during the period required for planning and implementing a repair, using loads and resistances calculated in accordance with Sections C, D, E, G and H.

The structure should be designed such that progressive collapse does not occur as a result of a high energy collision. In the damaged condition following a high energy impact the structure should be able to withstand environmental conditions which can be expected to occur in the period required to shut-down and evacuate the platform.

C.6.2 Dropped Objects

The possibility of dropped objects impacting on the structure should be considered in design. Resistance to dropped objects should be provided for by indirect means such as using redundant framing patterns and materials with sufficient toughness in vulnerable areas. Design of the structure against vessel impact will provide resistance to dropped object impact in the more vulnerable external members at the top of the structure. With conventionally configured steel structures, impacts with main legs are likely to be oblique and therefore less damaging. Normal impacts on bracing members are more likely and should be designed against by appropriate means. Further guidance on methods for design against dropped objects is provided in EP 89-0230.

C.6.3 Explosions

Conventional steel framed substructures which do not have enclosed compartments containing flammable fluids or ignition sources do not generally require direct design against explosion. The consequential effects of explosions in the topsides may require consideration in the substructure design. If explosion studies performed in the topsides design indicate unusual loading conditions on the substructure, or indicate specific support requirements to ensure that topsides integrity is maintained, then these should be provided for in the substructure design.

C.6.4 Fire

The possibility of the structure above water level being subjected to fire should be considered during design. Hydrocarbon pool fires on the sea surface can cause heating and degradation in the properties of structural elements. Guidance on methods for assessing the effects of fires, and on protecting against them is provided in EP 89-0230 and SCI-P-112.

SECTION E CONNECTIONS

E.3 TUBULAR JOINTS

E.3.1.2 Design Practice

Add to this clause:

If the offset between outermost centreline intersections at the joint exceeds D/4, the moments due to loads in the eccentric braces should be accounted for.

The arrangement of members at joints should allow adequate access for welding and inspection. Angles between primary structural members should be greater than 30 degrees. Wherever possible, simple joints should be used without overlap of braces with minimum clear distances between braces as specified above. The 51 mm minimum dimension should be measured between the weld toes of the braces. Greater spacing may be required to allow for welding access. Where overlap cannot be avoided, the joint should be detailed to avoid interference of welds and to allow access for welding and inspection. Stiffened joints should be avoided in favour of joints with greater wall thickness, unless the equivalent stiffened joint is demonstrably cheaper to fabricate or if weight constraints dictate their use.

E.3.5 Other Complex Joints

Add to this clause:

Guidance on methods for design of cross joints with diaphragms and overlapping joints is provided in EP 92-0595.

SECTION F FATIGUE

Replace this section with:

F.1 FATIGUE DESIGN

Fatigue design should be performed in accordance with the guidelines in EP 93-2005.

SECTION G FOUNDATION DESIGN

Replace second sentence with:

Design considerations and recommendations for the foundation design of mudmats are given in Sections G.12 to G.15.

G.1 GENERAL

Replace last sentence with:

The possibility of active geologic processes, discussed in Section A.4.9, in particular earthquake, sea floor instability, scour, shallow gas and seabed subsidence should be investigated, and, if anticipated, these effects should be considered in the foundation design.

G.2 PILE FOUNDATIONS

G.2.1 Driven Piles

Add to the first paragraph of this section:

It is not recommended to use vibratory hammers for installing axially loaded foundation piles, due to lack of data with respect to the effect of the installation method on the pile axial bearing capacity. Vibratory hammers may be considered for piles which are subject to horizontal loads alone. Conductor piles may be installed using vibratory hammers.

Replace second and subsequent sentences below ' 1. Plug Removal ' with:

If plug removal results in inadequate pile capacities, the removed soil plug should be replaced by a grout or concrete plug. The minimum load carrying capacity of the plug should be equal to the pile end bearing capacity in a plugged condition.

Replace last sentence below ' 2. Soil Removal Below Pile Tip' with:

Considering the uncertainties with respect to the pile bearing capacity, it is not recommended to remove the soil below the pile tip in order to reduce the soil resistance during driving in uncemented soils. Under special circumstances, in the case of an intermediate layer of strong cemented material, undersized drilling may be applied to remove the hard layer. The depth of drilling should be restricted to the thickness of the hard layer. Where soil removal below the pile tip during placement has been undertaken by undersized drilling, the contribution of the relevant zone of soil to the pile capacity should be ignored.

G.3 PILE DESIGN

G.3.4 Foundation Capacity

Replace first sentence with:

1. Pile Strength: The pile strength should be verified using the steel tubular strength checking equations given in Section D.3. for conditions of combined axial and lateral load and bending.

Add after last sentence:

The above pile resistance factors apply for design of foundations comprising isolated piles and for groups of closely spaced piles. For pile groups Φ_{PE} and Φ_{PO} apply to the overall capacity of the group, determined in accordance with Section G.9. For individual piles within a group Φ_{PE} and Φ_{PO} may be taken as 1.0, providing it can be shown that loads can be redistributed to other piles within the cluster in the event that the individual pile is overloaded.

G.4 PILE CAPACITY FOR AXIAL BEARING LOADS

G.4.1 Ultimate Bearing Capacity

Replace first sentence below symbols, "Total end bearing, Q_p..." with:

For open-ended tubular piles, the total end bearing capacity, Q_p, should not exceed the sum of the capacity of the internal plug and the end bearing on the pile wall.

Replace last sentence of last paragraph with:

The effect of a pilot hole should be considered when computing the total end bearing capacity of the bell.

G.4.2 Skin Friction and End Bearing in Cohesive Soils

Add to second paragraph:

For the unit shaft friction, an upper limit of f = 250 kPa is recommended. Justification of higher values will require verified supporting evidence.

Replace penultimate sentence with:

When the piles penetrate less than two to three diameters into the stronger layer some reduction in end bearing resistance is required. In order to preclude the risk of punch-through the piles should not be installed to a depth within three diameters from the top of the weaker layer below.

Add to this clause:

The equations G.4-2 and G.4-3 are applicable for flush tubular piles. If an internal driving shoe is applied to reduce the driving resistance, the effect of the internal driving shoe on the ultimate internal skin friction should be considered.

G.4.3 Shaft Friction and End Bearing in Cohesionless Soils

Replace this clause, except Table G.4.3.-1, with:

For pipe piles in cohesionless soils, the unit shaft friction, f, may be calculated by the equation:

$$f = K\, p'_o \tan \delta \qquad \text{(G.4-5)}$$

where:

K = coefficient of lateral earth pressure
p'_o = effective overburden pressure at the point in question
δ = friction angle between the soil and pile wall.

For open-ended pipe piles driven unplugged, the coefficient of lateral earth pressure should be taken as:

K = 0.7 for compressive loading
K = 0.5 for tensile loading.

Table G.4.3-1 may be used for selection of δ if other data are not available. For long piles, f may not indefinitely increase linearly with the overburden pressure as implied in equation G.4-5. In such cases, it may be appropriate to limit f to the values given in Table G.4.3-1. Where it can be justified based on in situ cone penetrometer test results, the limiting unit shaft friction may be taken equal to 120 kPa for very dense sands.

For piles in cohesionless soils, the unit end bearing, q, may be computed by the equation:

$$q = p'_o N_q \qquad \text{(G.4-6)}$$

where:

p'_o = effective overburden pressure at the pile tip
N_q = dimensionless bearing capacity factor.

Recommended values of N_q and limiting unit end bearing values are presented in Table G.4.3-1. The limiting unit end bearing should also take into account the extent and quality of the soil investigation. Where it can be justified based on in situ cone penetrometer test results, the limiting unit end bearing may be taken equal to 15 MPa. In the absence of such tests the limiting values in Table G.4.3-1 should be used as guidance.

The shaft friction acts on both the inside and outside of the piles. However, the total resistance in excess of the external shaft friction plus pile wall end bearing is the total internal shaft friction or the end bearing of the plug, whichever is less. For piles considered to be plugged the bearing pressure may be assumed to act over the entire cross section of the pile. For unplugged piles the bearing pressure acts on the pile wall only. Whether a pile is considered to be plugged or unplugged should be based on static calculations. For example a pile could be driven in an unplugged condition but act plugged under static loading.

For soils that do not fall within the ranges of soil density and description given in Table G.4.3-1 or for materials with unusually weak grains or compressible structures, Table G.4.3-1 may not be appropriate for selection of design parameters. For example, very loose silts or soils containing large amounts of mica or volcanic grains may require special laboratory or field tests for selection of design parameters. Of particular importance are calcareous soils (soils containing a high proportion of calcium carbonate), which are found in many areas of the oceans. Available data suggest that driven piles in these soils may have substantially lower design strength than given in Table G.4.3-1. Factors of importance for assessment of limiting unit end bearing and skin friction values are, among others, the degree of cementation, grain crushability, relative density, compressive strength and carbonate content. Drilled and grouted piles in calcareous sand have a significantly higher capacity than driven piles. The characteristics of calcareous sand are highly variable and local experience should dictate the design parameters selected. These materials are discussed further in the Commentary.

Since the effect of drilling or jetting ahead of the pile tip on the bearing capacity is unquantifiable it is strongly recommended not to apply these techniques in cohesionless material. Except in unusual soil types such as described above, the values of f and q in Table G.4.3-1 may be used for drilled and grouted piles, with consideration given to the strength of the soil grout interface.

In layered soils unit shaft friction values, f, in the cohesionless layers should be as outlined in Table G.4.3-1. End bearing values for piles tipped in cohesionless layers with adjacent soft layers may also be taken from Table G.4.3-1, assuming that the pile achieves penetration at least two diameters into the cohesionless layer and the tip is at least three diameters above the bottom of this layer to preclude punch through. Where the distance of two diameters is not achieved reduction of the bearing capacity may be necessary. In order to preclude punch through the tip of the piles shall not be installed within a distance of three pile diameters from the top of the soft layer. Where adjacent layers are of comparable strength to the layer of interest, the proximity of the pile tip to the interface is not a concern.

G.9 PILE GROUP ACTION

G.9.2 Axial Behaviour

Add to this clause:

For piles acting in a group the axial bearing capacity of the pile group is considered to be the lesser of:

- the sum of the capacities of the isolated piles, or
- the capacity of the ‘equivalent pier', where the pier forms an envelope around all piles.

The axial bearing capacity of the pile group consists of skin friction along the outer perimeter of the pile group plus end bearing of the pier. Unit skin friction values, f, may be taken as:

- for a soil to soil failure surface; clay, f = c, where c = undrained shear strength
  sand, $f = Kp'_o \tan\phi$, where ϕ = angle of internal friction

- for a soil to steel failure surface; refer to Section G.4.3.

The end bearing capacity of the pier may be calculated using the equations G.4-4 and G.4-6. It should be noted that these equations are applicable for small size footings in uniform soils. In estimating the end bearing capacity of the pile group special considerations are required with respect to:

- size effect of the footing,
- allowable displacement,
- the presence of weak layers within a distance of 1.5 to 2 times the equivalent pier diameter from the tip of the foundation.

G.10 PILE WALL THICKNESS

G.10.3 Pile Design Checks

Replace first sentence with:

The pile wall thickness in the vicinity of the mudline, and possibly at other points, is normally controlled by the combined axial load, lateral load and bending moment which results from the factored loading conditions for the platform.

G.10.6 Minimum Wall Thickness

Add to the end of this clause:

Lower values of D/t may be required for the bottom section of the pile to prevent damage to the pile during installation. Experience indicates that for piles with high D/t ratio, minor local damage near the pile tip (e.g. out-of-roundness or denting) may propagate during installation (e.g. driving) to cause more extensive deformation and collapse of the pile. In addition to selecting an appropriate D/t ratio, the pile bottom section should be checked for all loads occurring during handling (e.g. pick-up, upending, stabbing etc.) to ensure that local damage does not occur.

G.10.7 Allowance for Underdrive and Overdrive

Add to this clause:

The possibility of underdrive and overdrive should also be considered when determining the required length of the piles and when determining the position and extent of shear keys in grouted pile sleeve connections.

G.10.8 Driving Shoe

Add to this clause:

For reducing the internal skin friction during driving in cohesive soils, an internal driving shoe at the pile tip at least one diameter in length with a minimum additional wall thickness of 0.5 inch (13 mm) may be considered. The effect of an internal driving shoe should be taken into account when evaluating the total ultimate bearing capacity of the pile. Internal driving shoes generally do not assist driving in predominantly granular soils.

G.12 SHALLOW FOUNDATIONS

Replace Sections G.12 to G.17 with G.12 to G.15 below.

G.12 MUDMAT DESIGN

The foundation design of mudmats should take account of the following:

- bearing capacity under the combined effect of horizontal and vertical loads
- penetration and settlements
- cyclic foundation behaviour under dynamic loading
- hydraulic instability such as scour and piping due to wave pressures.

Provisions for foundation design of mudmats are given in Sections G.13 to G.15.

The structural design of mudmat components and connections should normally satisfy the strength and stability requirements in Sections D, E and H2. However, some relaxations in strength requirements may be permitted (e.g. allowing local yielding in mudmat components) if it can be shown that bearing and sliding resistances of the mudmats are not impaired and the components are not required for the long term performance of the structure.

G.13 MUDMAT BEARING CAPACITY

The equations to be used for evaluating the bearing capacity of mudmats under combined horizontal and vertical loading conditions (inclined loading) are given below and in Comm. G.13. These equations are applicable to homogeneous soil conditions, i.e. uniform undrained and uniform drained material. Where use of these equations is not justified, e.g. layered soils, a more refined analysis or special solutions should be considered.

G.13.1 Acceptance Criteria

The equations presented in Sections G.13.2 to G.13.4 should be used to develop a soil resistance envelope as shown in Figure G.13-1. The mudmat plan area should be sized to meet the criteria in the Sections below.

In soft soils (e.g. normally consolidated clays) the mudmat will penetrate into the seabed to the depth at which the soil bearing capacity is in equilibrium with the applied bearing pressure; i.e. factor of safety equals 1.0. In such cases, the mudmat design may be based on displacement criteria allowing additional penetration under environment loads. The allowable uneven settlement of the structure depends upon the type of structure and installation tolerances, etc. Adequate precautions should be taken to minimise differential settlements between mudmats.

G.13.1.1 Installation Storm Conditions

All combinations of horizontal and vertical foundation reactions resulting from combined gravitational and environmental load should satisfy the following condition:

$$E_F \leq \Phi \, R_{V,H} \quad \text{(G.13-1)}$$

where:

E_F = environmental load vector as defined in Figure G.13-1

$R_{V,H}$ = soil resistance vector as defined in Figure G.13-1

Φ = resistance factor for bearing capacity, 0.75 for undrained and drained soil conditions.

The above criteria represent the capacity of individual mudmats. For structures comprising several mudmats they may not be representative of the ultimate capacity of the total foundation system. In those cases where the above criteria are not met, load transfer between individual mudmats may be considered, providing the structure can accommodate the resulting load distribution.

G.13.1.2 Still Water Conditions

Vertical foundation reactions resulting from gravitational load should satisfy the following condition:

$$V \leq \Phi \, Q \quad \text{(G.13-2)}$$

where:

V = vertical foundation reaction resulting from factored gravitational load

Q = ultimate vertical bearing capacity of mudmat determined using Equation G.13-3 or G.13-4

Φ = resistance factor for bearing capacity, 0.75 for undrained and drained soil conditions.

G.13.2 Undrained Bearing Capacity

The ultimate vertical bearing capacity, Q, of a mudmat under undrained conditions is:

$$Q = (c\, N_c K_c + q)A' \quad \text{(G.13-3)}$$

where:

- c = undrained shear strength of soil
- N_c = dimensionless constant equal to 5.14 for $\phi = 0$
- ϕ = undrained friction angle = 0
- K_c = correction factor, which accounts for load inclination, mudmat shape and depth of embedment
- q = $\gamma' X$
- X = depth of embedment
- γ' = effective unit weight of soil
- A' = effective area of the foundation.

Methods for determining the correction factor, K_c , and the effective area, A', are given in COMM. G.13.

Equation G.13-3 is applicable for uniform shear strength. In case of a linear increase of undrained shear strength with depth, the method proposed by DAVIS AND BOOKER may be used.

G.13.3 Drained Bearing Capacity

The ultimate vertical bearing capacity, Q, of a mudmat under drained conditions (c = 0) is:

$$Q = (c' N_c K_c + q N_q K_q + 0.5\gamma' B N_\gamma K_\gamma)A' \quad \text{(G.13-4)}$$

where:

- c' = effective cohesion
- ϕ' = effective friction angle
- N_c, N_q, N_γ = dimensionless factors depending on ϕ'
- B = minimum width of foundation
- K_c, K_q, K_γ = correction factors, which account for load inclination, mudmat shape and depth of embedment.

A complete description of the K factors as well as curves showing the numerical values of N_c, N_q and N_γ as functions of ϕ' are given in COMM. G.13.

In cohesionless soils it can be assumed that c' = 0. Generally the cohesion term is only applicable in cohesive soil during long term sustained loading, after consolidation. Silty soils may behave drained during installation of the structure, but may behave partly drained or undrained during cyclic loading, due to its low permeability.

G.13.4 Sliding Stability

The limiting conditions of the bearing capacity equations in Sections G.13.2 and G.13.3, with respect to inclined loading, represent sliding failure and result in the following equations.

1. Undrained Analysis:

$$H = c\, A_o \quad \text{(G.13-5)}$$

Where:

H = horizontal resistance

A_o = actual foundation area

2. Drained Analysis:

$H = c'A_o + F_{VH}\tan\delta$ .... (G.13-6)

Where:

F_{VH} = vertical mudmat load (unfactored)

δ = friction angle between soil and mudmat = ϕ' - 5 degrees

G.14 MUDMAT SETTLEMENT

Mudmat settlements should be calculated taking into account the following:

- elastic behaviour of the soil (immediate settlements)
- consolidation of the soil (long term deformation)
- cyclic loading resulting in excess pore water pressure.

Guidance on the calculation of immediate and consolidation settlements is given in standard soil mechanics references such as G33 and G34.

During storms, cyclic loading may generate excess pore water pressure resulting in liquefaction of the soil. Detailed liquefaction analyses should be performed to analyse cyclic settlements. Special attention should be given to silts and silty sands.

G.15 SCOUR

The possibility of scour, during the period when the mudmat is required to carry load, should be considered in the design.

SECTION H STRUCTURAL COMPONENTS AND SYSTEMS

H.1 SUPERSTRUCTURE DESIGN

Delete clauses H.1.2, H.1.3, and H.1.4

H.3 CRANE SUPPORTING STRUCTURE

Delete this section.

H.4 GROUTED PILE-TO-STRUCTURE CONNECTIONS

H.4.1 General

Add to this clause:

Fabrication should be simplified by specifying a minimum number of shear keys and by sizing weld bead shear keys so that they can be placed in one pass, see Figure H.4.1-2.

The relative displacements between the pile and sleeve should be investigated in case temporary clamps or pile grippers are needed to reduce movements during grout curing.

H.4.2 Computation of Applied Axial Force

Add to this clause:

The ultimate capacity of the connection should exceed the ultimate pile capacity.

H.4.3.4 Other Design Methods

Replace this clause with:

The equations given in clauses H.4.3.1, H.4.3.2 and H.4.3.3 have been shown to be unconservative for some pile to sleeve connection designs, particularly those using larger diameter radially flexible piles. The alternative equations provided in DEn have been shown to be more appropriate in these situations. The lower pile to sleeve connection capacity of either DEn or clauses H.4.3.1, H.4.3.2 and H.4.3.3 should be used. This approach should also be used for designing the grout to steel bond in drilled and grouted piles. Where the alternative equations are used, appropriate load and resistance factors should be applied to provide the same overall safety factors as specified by DEn.

H.5 CONDUCTORS

Replace this section by:

Guidance on the design and analysis of marine conductors is provided in EP 87-0160.

H.6 GUYLINE SYSTEM DESIGN

Replace this section with:

H.6 ANCILLARY SYSTEMS AND MISCELLANEOUS ATTACHMENTS

Attachments of any ancillary systems required for the installation of the platform and other miscellaneous attachments should be designed to withstand loads incurred during all phases of fabrication, load-out, transportation, lifting and installation as well as in-place loads. The design should account for static loads and for fatigue. Attachments should be located such that the risk of damage due to accidental load is minimised. Attachments should be supported from secondary structural members in preference to primary members. Doubler plates should be used at the connections of vulnerable attachments to protect the main structural member in the event of damage. These should be designed so that the attachment to doubler connection will fail before the doubler to primary member connection.

Large cyclic stresses and fatigue damage can occur due to vibrations during pile and conductor driving in the vicinity of the hammer, the pile and the pile sleeves. These should be considered, and if necessary accounted for in the design of attachments and their supports. Where possible miscellaneous attachments should be located to avoid these areas. See also EP 93-2005 for further guidance on this subject.

Add new section:

H.7 CORROSION PROTECTION SYSTEM DESIGN

Corrosion protection of the platform should be provided by a cathodic protection system for the submerged parts of the platform and protective coatings on components in the splash zone and above.

Corrosion allowances on thickness should also be provided for components in the splash zone.

The cathodic protection system should be designed in accordance with DDD 37.19.30.30-Gen. Consideration should be given to the requirements for providing positive electrical connections between the structure and appurtenances such as conductors, caissons, sumps and chutes. Pipeline risers may require to be connected to or isolated from the structure cathodic protection system, depending upon whether they are connected to the pipeline protection system. If the structure cathodic protection system is required to protect risers and/or other appurtenances, then the surface area of these components should be taken into account when calculating the required anode capacity.

DEP 30.48.00.31-Gen. provides guidance on the specification of coating systems for the splash zone and above. Components requiring coating should be detailed to permit proper blasting and coating.

Corrosion allowances on thickness for components in the splash zone should be determined taking account of the intended service life and the rate of corrosion at the platform location. The extent of the splash zone should be determined accounting for the effects of tidal ranges, wave action and uncertainty in the final platform elevation.

The additional thickness of steel should not be taken into account when calculating stresses, however its deadweight should be allowed for.

The possibility of internal corrosion in caissons or other components which are flooded or contain corrosive fluids should be considered, and appropriate coatings or additional corrosion allowances on thickness should be provided.

All members and components should be detailed to avoid corrosion traps, using seal welds where required to avoid crevices.

SECTION I MATERIAL

Replace Sections I.1, I.2 and I.3 with I.1 to I.3 below:

I.1 MATERIAL SPECIFICATIONS

Material specifications are required covering all materials used in the structure. When preparing material specifications account should be taken of the nature of the design and loading, the local environmental conditions and locally available material supply and fabrication facilities. This section provides guidance on the issues which should be addressed in the selection or preparation of specifications to ensure that the materials used are compatible with the design approach described in this DEP.

I.2 STEEL PLATES, HOT ROLLED SECTIONS AND SEAMLESS TUBULARS

I.2.1 General

DEP 37.19.10.30-Gen. provides the specification for structural steel plates, hot rolled sections and seamless tubulars for primary applications. Steels for secondary applications should be selected from a suitable locally applicable specification. Suitable steel grades should be selected during design, for all components of the structure, taking account of the service and fabrication conditions. Guidance on the selection of steel grades is given in the following sections.

I.2.2 Strength

DEP 37.19.10.30-Gen. includes steels with minimum yield stress ranging from 275 MPa to 450 MPa. For components which are governed by static strength, the higher yield stress steels should generally be used in order to minimise weight, unless considerations such as availability, cost or welding dictate otherwise.

Where higher strength steels are considered, particularly those with minimum yield stress above 400 MPa, special attention should be given to:

- weldability and requirements for special welding procedures,
- fatigue problems which may result from the use of higher working stresses,
- toughness requirements in relation to other elements of fracture control such as fabrication specifications, inspection procedures, service stress, and service temperature,
- potential effect of seawater and corrosion protection system performance on fatigue characteristics,
- reduced strain hardening associated with higher ratio of yield strength to ultimate strength.

I.2.3 Primary and Secondary Steels

For the purpose of defining toughness requirements and requirements for testing to verify weldability, steel should be categorised either as primary or secondary. The steel category should be determined based on the function and importance of the component for which it is used.

Secondary steel should be used for components which are not essential to the overall integrity and operational safety of the structure. Examples of such components include ladders, walkways, walkway supports, mudmats, anode supports, conductor guide cones, top of pile sleeve cones and guides, installation piping, support steelwork for installation piping, J-tubes, caissons, and seafastenings.

Primary steel should be used for components which are essential to the overall integrity or operational safety of the structure. Primary components include all items not specifically designated as secondary.

I.2.4 Toughness Requirements

Toughness properties should be verified for all steels by performing Charpy impact testing as specified in DEP 37.19.10.30-Gen. Test temperatures should be determined using the table below:

Charpy Test Temperature (degrees Celsius)		
Thickness	Primary	Secondary
< 40 mm	T - 20	T
≥ 40 mm	T - 30	T - 10

T = minimum design temperature (degrees Celsius)

The minimum design temperature should take account of the air and water temperatures to which the structure will be exposed during construction and whilst in service.

I.2.5 Weldability Testing

The weldability should be verified for all primary steel by performing weldability testing as specified in DEP 37.19.10.30-Gen.

I.2.6 Through Thickness Properties

DEP 37.19.10.30-Gen. includes steel grades with verified through thickness properties.

The use of steel with verified through thickness properties should be considered for all welded connections between components, to avoid the risk of lamellar tearing. Verified through thickness properties should be specified where any of the following conditions exist:

- through thickness loading
- complex (e.g. tri-axial) stress distribution
- stress distributions which cannot be accurately predicted
- high restraint during welding.

I.2.7 Modified Grades

DEP 37.19.10.30-Gen. includes a number of M or modified grades, which have non-standard modifications to the base specification properties. The modifications apply to the chemistry, tensile strength and impact test requirements and are aimed at providing improved weldability and toughness. These modifications have been evolved to meet requirements for applications in the North Sea and are also recommended for other areas which have a similar environment.

I.3 FABRICATED STEEL TUBULARS AND SECTIONS

Steel plates used for fabrication of tubulars and sections should comply with the requirements of Section I.2. The fabrication specification should define all requirements relating to the fabrication of tubulars and sections from steel plate.

Unless sufficient test data are obtained to verify their suitability, spiral welded tubulars should not be used for structural applications.

I.4 CEMENT GROUT AND CONCRETE

I.4.1 Cement Grout

Replace the first sentence with:

If required by the design, the space between the piles and the surrounding structure, or between the piles and soil in a drilled and grouted foundation, should be carefully filled with grout. If the design of the grouted connection is based on the equations provided in DEn, then the grout material should also be in accordance with the requirements in DEn. If the design of the grouted connection is based on Sections H.4.3.1, H.4.3.2 and H.4.3.3, then the grout material should be in accordance with the provisions given herein.

Add new section:

I.5 STEEL CASTINGS

Cast steel may be used in components and connections providing it can be demonstrated that it is suitable for all conditions during fabrication and in service. A specification should be prepared defining all manufacturing requirements. For components or connections subject to long term fatigue loading, adequate fatigue performance must be demonstrated, taking account of the environment in which the casting will be used and the defect acceptance criteria applied to the casting.

Add new section:

I.6 TIMBER

Where timber is used for skid beams, launch runners or mudmats a specification should be prepared taking account of locally available timber types and locally applied standards relating to timber materials. The specification should detail the strength requirements, the grading and moisture content applicable and should define requirements for testing and storage of the timber prior to use. Dimensional tolerances and constraints on timber arrangement should be specified and requirements for coating, sealing, lubrication and fastening of the timber should be defined. The stiffness of the timber should be specified if this is important for the design.

Glue laminated beams should be considered for launch runners as they can provide a cost effective solution. The adhesives used should be suitable for marine applications and the intended orientation of the lamination planes relative to the loading should be defined.

SECTION J DRAWINGS AND SPECIFICATIONS

J.1 GENERAL

Replace this section with:

This section provides guidance on the content and detail required in drawings and specifications. Where reference is made to items which are specific to the topside structure and facilities, these should be disregarded in the context of this DEP.

SECTION K WELDING

Replace this section with:

K.1 GENERAL

Requirements for welding should be detailed in the fabrication specification and should take account of the materials used, the local fabrication practices and the specific attributes of the structure. The fabrication specification should ensure that welding is compatible with all aspects of the design. See also COMM. L.

K.2 STRESS RELIEF

Post weld heat treatment (PWHT) of welded joints in plates, tubulars and structural assemblies is normally required when the joint reference thickness exceeds the following values:

- 40 mm for tubular intersection welds,
- 50 mm for all other welds.

However, joints with a reference thickness greater than above but less than 100 mm may be exempt from PWHT if satisfactory toughness in the as welded condition can be demonstrated by means of crack tip opening displacement (CTOD) testing on the weld and heat affected zone. The joint reference thickness is the thickness of the thinner of the two parts joined, as defined in Figure K.2-1. The toughness should be demonstrated for the maximum weld thickness used. The required toughness should be determined by the design Contractor.

SECTION L FABRICATION

Replace this section with:

L.1 GENERAL

A fabrication specification should be prepared detailing all relevant requirements including materials, welding, inspection, testing, dimensional tolerances and fabrication and assembly practices, as well as requirements for engineering, procedures, documentation and approvals. This should take account of the specific attributes of the structure, the anticipated fabrication and assembly methods, and the contractual arrangements between the fabrication Contractor and Owner. The fabrication specification should ensure that fabrication is compatible with all aspects of the design. See also COMM. L.

SECTION M INSTALLATION

M.1 GENERAL

M.1.4 Temporary Bracing and Rigging

Replace last two sentences with:

If any of the installation aids, temporary struts or bracings are to be welded to the structure, then all welding shall be in accordance with Section K. Removal shall be in accordance with Section L.

M.2 TRANSPORTATION

M.2.2.5 Seafastenings

Add to this clause:

The design of the seafastenings should be considered at a sufficiently early stage during design of the structure, to establish loading on the structure, to identify conditions which affect sizing of the structure and to ensure that efficient details for connections and supports can be developed.

M.2.2.8 Buoyancy and Flooding Systems

Add to this clause:

The design should provide adequate stability and bottom clearance during all stages of the installation process and should consider contingency conditions in which individual buoyancy compartments are accidentally flooded. Minimum acceptable values for the longitudinal and transverse metacentric height in intact and damaged conditions should be established for all phases of the upending process, taking account of uncertainties in weight, buoyancy and centre of gravity position. The minimum acceptable bottom clearance during upending should take account of the tide level used in the analysis, sea bottom conditions, draught mark reading inaccuracy, uncertainties in water depth, jacket weight, buoyancy and centre of gravity and the effects of motions and current induced trim and heel.

Temporary buoyancy tanks should be considered if inherent buoyancy in the structure is inadequate to provide sufficient reserve buoyancy, stability or bottom clearance. Capping of pile sleeves or legs should be considered if this is more cost effective than providing auxiliary buoyancy tanks. Use of derrick barge assisted upending should be considered as an alternative to the addition of buoyancy to provide adequate bottom clearance during upending. Where temporary buoyancy tanks are used the design should provide for simple removal offshore.

Leak and pressure testing of all buoyancy compartments and flooding systems should be performed during fabrication.

The provisions of sections M.2.3.2, M.2.3.3 and M.2.3.4 also apply to structures which are launched from a barge into a floating position then upended and placed on the bottom by controlled flooding of buoyancy compartments.

M.3 REMOVAL OF JACKET FROM TRANSPORT BARGE

M.3.3.3 Flotation

Add to this clause:

Where hook assisted upending is used, the effects of dynamics and relative motions between the derrick barge and the floating structure should be accounted for as these may cause loads which are substantially higher than would be obtained in static conditions.

M.4 ERECTION

M.4.6 Guyline System Installation

Delete this clause including M.4.6.1, M.4.6.2, M.4.6.3 and M.4.6.4.

M.5 PILE INSTALLATION

M.5.5 Obtaining Required Pile Penetration

Replace second paragraph with:

The fact that the pile has met premature refusal, either during continuous driving or during re-drive, does not assure that it is capable of supporting the design load. Back analysis of the actual driving records may give an indication on the soil resistance during driving. Pile driving data may provide some additional information for assessing the ultimate static pile bearing capacity. However it is emphasised that the ultimate static bearing capacity can not be based solely on pile driving behaviour. For example, during driving, the pile may behave unplugged, whilst during the operational period of the platform the foundation pile may behave plugged.

In order to minimise delays during installation a pile acceptance procedure should be established. The procedure should outline the required actions to be taken for possible pile driving scenarios, i.e.:

- premature pile driving refusal,
- significantly lower blow count at design depth than anticipated, based on pile driveability analysis,
- significant decrease of blow count near its design penetration.

Any required remedial measure should be approved by the design engineer.

M.5.6 Driven Pile Refusal

Add to this clause:

In order to confirm whether the hammer performs in accordance with the specification, it is recommended to instrument and monitor the pile/ hammer during pile installation.

Replace second to fourth paragraphs by:

The definition of refusal for a particular installation should be included in the installation contract. The following pile driving refusal criteria may be applied, assuming the hammer is operated at the pressure and net energy rate recommended by the manufacturer. Refusal is defined as the point where pile driving resistance exceeds one of the following criteria:

- continuous driving
  - a minimum of 150 blows/0.25 m over 6 consecutive intervals of 0.25 m, or
  - a minimum of 200 blows/0.25 m over 2 consecutive intervals of 0.25 m,
- last interval of 0.25 m at the end of driving or first interval after restart 400 blows/0.25 m.

M.5.8 Drilled and Grouted Piles

Replace first paragraph with:

Drilling the hole for drilled and grouted piles should be accomplished without the use of drilling mud to facilitate maintaining an open hole. Drilling mud may be detrimental to the surface of some soils. If drilling with mud is essential, due consideration should be given to the effect of the mudcake on the pile shaft friction. Drilling operations should be done carefully to maintain proper hole alignment and to minimise the possibility of hole collapse. The insert pile with an upset drilling bit on its tip may be used as the drill string so that it can be left in place after completion of the hole.

M.5.11 Grouting Piles to Structure

Add to this clause:

Where inflatable packers are used, packer inflation lines should be provided with a back-up inlet to the packer. The grouting system should provide for setting a grout plug in the event that the packer system becomes damaged. Back-up grouting systems should be provided in case the primary system becomes blocked or has been used to set a grout plug. Provision should be made for grout monitoring equipment at the top of the pile sleeve or grout return location. Details of the grouting system should be developed in conjunction with the Installation Contractor.

M.6 SUPERSTRUCTURE INSTALLATION

Replace this section including clauses M.6.1, M.6.2, M.6.3, M.6.4 and M.6.5 with:

M.6.1 Installation Aids

Guides, bumpers or stabbing cones should be provided to assist in the location and placement of the topsides onto the substructure. These should be designed to withstand local impact loads during installation and should protect the substructure and deck against damage due to accidental overloads. The design of seatings or stabbing points should provide for simple installation offshore, considering practical fabrication tolerances in the substructure and deck and should facilitate levelling of the deck within the required tolerances on the substructure. Specific criteria for the design of installation aids should be determined in conjunction with the Installation Contractor, and should be consistent with limiting environmental conditions specified in the installation procedures.

SECTION N INSPECTION

Replace this section with:

N.1 GENERAL

Requirements for inspection should be detailed in the fabrication specification and should take account of the materials used, the specific attributes of the structure and the fabrication process envisaged. The fabrication specification should ensure that inspection is compatible with all aspects of the design. See also COMM. L.

SECTION O SURVEY

Replace this section with:

O.1 GENERAL

A detailed plan for in-service inspection and survey of the structure should be developed as a part of the overall operating procedures. The inspection philosophy should be considered during the design phase so that effective and efficient inspection can be provided for in the design. See also COMM. O.

SECTION P PLATFORM REUSE

Delete this section.

SECTION Q MINIMUM STRUCTURES

Q.2.2 Fatigue Analysis

Replace this section with:

A fatigue analysis including dynamic effects should be performed in accordance with Section F.

Q.4 MATERIAL

Delete this section.

COMMENTARY ON LRFD METHODOLOGY

Add to the introductory clause:

The load and resistance factors presented in this recommended practice have been calibrated to provide appropriate overall safety levels, when used in conjunction with the formulations for loads and resistances specified in this recommended practice. Care is required when using alternative formulations for loads and resistances from other sources, to ensure that appropriate partial factors are applied to achieve the required safety levels.

The partial factors given in API RP 2A-LRFD were developed so that when used in conjunction with traditional design practice in the USA, they will provide structures with appropriate reliability for this area. For other geographic areas, the same partial factors used in conjunction with traditional design practice for that area will also provide structures with similar or better reliability than provided with traditional Working Stress Design (WSD) practice. See also Comm C.3.1.2.

COMMENTARY A PLANNING

Add new section:

COMM. A.2 PLATFORM TYPES

Within API RP 2A-LRFD, nomenclature is used which differs from, or is not entirely consistent with, the recommended nomenclature given in A.2.1.1 and A.2.1.2. This DEP has not attempted to amend all such occurrences. This should be recognised by users of this DEP. For example, where provisions of API RP 2A-LRFD are stated as being specific to a structure type, they may in fact be more generally applicable rather than being intentionally specific to a particular structure type.

COMM. A.3 OPERATIONAL CONSIDERATIONS

Comm. A.3.8 Wells and Risers

Add to this clause:

The most effective means of minimising the exposure of wells and risers to accidental damage is to locate these components within the boundaries of the structure. Where this is not feasible (e.g. for operational or installation reasons) exposure of wells and risers on external faces can be reduced by using riser guards or fenders or by placing restrictions on operations to minimise the hazard sources (e.g. restrictions on vessel approach sectors and crane operational areas).

COMMENTARY C LOADS

Add new clause:

Comm. C.2.10 Weight Estimates

An example of a comprehensive weight estimating procedure is given in ERD EN/001.

COMM. C.3 WIND, WAVE AND CURRENT LOADS

Comm. C.3.1.2 Extreme Wind, Wave and Current Load

Add to the end of this clause:

For some sea areas, substantial databases are becoming available with which it is possible to establish statistics of joint occurrence of wind, wave and current magnitudes and directions. When such a database is available, it is recommended that this should be used to develop environmental conditions based on joint occurrence, which provide the 100 year return period environmental load. The load factors used in conjunction with this environmental load should be determined using structural reliability analysis principles to ensure that an appropriate structural reliability is achieved. This approach is recommended since it provides more consistent reliability for different geographic areas than has been provided by existing practice of using separate (marginal) statistics of winds currents and waves.

Comm. C.3.1.4 Operating Wind, Wave And Current Load

Add to this clause:

The return period used for operating environmental conditions are different in different regions. This is legitimate in view of the intent of the design check for operating conditions and in view of the differing nature of environmental conditions. In the South China Seas, 1 year return period conditions have traditionally been used. Since these are mild by comparison with the extreme conditions they do not have an unduly severe impact on the design. This is consistent with the approach described in API RP 2A-LRFD for the Gulf of Mexico. In the Northern North Sea, however, 1 year conditions are relatively severe by comparison with 100 year conditions and their use would have an unduly large impact on design. In this area a milder operating condition is selected, based on the maximum wave in the most frequently occurring seastate in the wave scatter diagram, used in conjunction with the one month return period wind and current.

Where dead or live loads on the structure during temporary conditions are significantly heavier than normal maximum conditions (e.g. during hydro-test), it may be appropriate to define limiting environmental conditions in which the particular mode of operation may proceed. These limiting conditions would then be applicable for determining the operating wind, wave and current load for that mode of operation.

Comm. C.3.2.1 Apparent Wave Period

Add to this clause:

The definitions of apparent wave period (for an observer moving with the current) and actual wave period (for a stationary observer) should be noted. In the literature it is not uncommon to find these definitions reversed. The text of Section C.3.2.1 and Figure C.3.2.2 are only for the definitions as given by API.

Comm. C.3.2.12 Local Member Design

Replace the first five paragraphs, discussing VIV, with:

The Morison equation accounts for mean in-line drag and inertia forces, but not for the oscillating in-line and cross-flow forces due to periodic vortex shedding from the downstream side of the member. These periodic forces can cause significant member vibrations when the shedding frequency coincides with the natural frequencies of the member. Vortex shedding forces can be neglected in the calculation of global structural loads; however, they should be considered in the calculation of local member forces and responses. Vortex induced vibrations can occur on long spans due to wind in the construction yard and during transportation, as well as due to waves and currents on the in-place structure. Guidance on methods to assess vortex shedding is given in EP 93-0455.

Add after penultimate paragraph:

Further guidance on methods to design for wave slam is given in NMI R 158.

Add new clause:

Comm. C.3.2.13 Recommended Static Wave Analysis

The API Recipe

The wave force recipe in Sections C.3.2.1 to C.3.2.10 has been substantially revised from previous editions of API RP 2A. The revisions resulted from a review by the API Wave Force Task Group during the period 1988 to 1992. The review evaluated the historically based provisions and found these to be hard to defend alongside contemporary knowledge on wave loading.

A key feature of the new API recipe is the adoption of drag and inertia force coefficients which are consistent with the values measured in various laboratory and full scale measurement programmes conducted in recent years. The traditional values for the drag coefficient of 0.6 to 0.7 for circular cylinders is replaced by higher values, which are acknowledged as being appropriate for the real flow conditions in the offshore environment. The drag coefficient for rough (marine fouled) members takes a value of 1.05 or higher depending on Reynolds number and Keulegan Carpenter number. Compatible values for the inertia force coefficient have also been adopted with a reduction from the traditional value of 2.0 to a value of 1.6 or lower.

Wave kinematics are still calculated using a traditional regular periodic wave theory, with a wave height and period selected to represent a particular maximum condition. Kinematics are however reduced to account for wave directionality, and free stream current speeds are reduced to account for blockage. For closely spaced conductor arrays, loads are also reduced to account for shielding. For Gulf of Mexico applications there are also some reductions in design wave heights, and current speeds are now specified in the RP.

For the spaceframe structures evaluated by the task group the new provisions result in higher computed wave forces compared to what had been the practice in the US Gulf of Mexico. Examination of available information on platform loads during recent hurricanes did not contradict the larger forces. This, the better scientific basis, and the relatively low cost increase for new designs led to the adoption by API.

The NEWWAVE Approach

Wave loading research within the Group in recent years has also confirmed that higher drag coefficients are required to provide consistency with laboratory and full scale measurements in realistic flow conditions. The use of a regular periodic wave theory for representing realistic random wave conditions has however also been found to be inappropriate, and generally over-predicts the kinematics in extreme storms. Group research has therefore also focused on more appropriate models for wave kinematics to replace the traditional regular wave theories.

Real sea conditions comprise random rather than regular waves. They are composed of a complex system of individual 'wavelets' of different amplitudes and periods, travelling at different speeds and in different directions. As an alternative to using a regular periodic wave, time domain simulations of random waves in an extreme storm can be performed. The full range of amplitudes, periods and directions are included in the simulations, and the extreme conditions extracted represent the actual superposition of individual 'wavelets' to produce a maximum at a particular time. Wave kinematics and loads produced by such random directional wave load models have been extensively validated against offshore measurements and laboratory experiments and have provided excellent agreement, when used in conjunction with realistic force coefficients. However, application of random time domain methods for routine design is not practical due to the extensive computer and manpower effort required.

As an alternative, NEWWAVE theory has been developed, which uses a statistically based superposition of random linear waves to define a deterministic wave profile and associated kinematics representing the most probable maximum wave conditions in a real random seastate. The wave kinematics calculated using NEWWAVE are corrected to account for wave directionality and are combined with current speeds reduced to account for blockage. NEWAVE is valid in the range of waterdepths where Stokes V order wave theory is also valid.

The loads produced using NEWWAVE theory have been validated against time domain simulations of random waves and against offshore measurements. This validation has focused on drag dominant structures for which maximum wave forces occur with the wave crest positioned at the structure and has demonstrated good agreement. NEWWAVE theory, used in conjunction with realistic force coefficients, is therefore recommended as the preferred method for calculating extreme wave loads for such structures.

Comm. C.3.3 Extreme Wave Dynamic Analysis

Add to the end of this clause:

Further discussion on the additional 1.25 load factor is given in EP 93-2525. Where the guidance given by EP 87-0170 and EP 93-2525 is followed, several of the major uncertainties associated with the calculation process are rationally addressed. Under these circumstances, for structures exhibiting moderate dynamic response, a reduction in load factor may be justified for specific applications based on detailed evaluation of the reliability based calibration.

Delete Comm. clauses C.3.3.1, C.3.3.2, C.3.3.2.3, C.3.3.2.6 and C.3.3.3

COMM. C.4 EARTHQUAKE LOADS

Delete Comm. clauses C.4.1.3 and C.4.4.2

COMM. C.5 FABRICATION AND INSTALLATION FORCES

Comm. C.5.1 General

Add to this clause:

The calibration of the LRFD partial factors presented in this Recommended Practice has been based primarily on evaluation of in-place performance. The factors have not been extensively calibrated for installation conditions; however, they are considered to be reasonable based on simple correlations with implied safety factors in WSD methods. Whilst use of the LRFD approach for installation conditions is fully endorsed, in view of its limited calibration, designers should exercise due caution in its application. Results obtained should be reviewed and their validity carefully evaluated if they are found to be substantially different from those which would be produced using WSD methods.

Comm. C.5.5 Lifting Forces

Comm. C.5.5.1 General

Add new clause:

Experience based lifting criteria in which the effects of dynamics, lift weight, sling load distribution, etc. are more explicitly accounted for have been developed by a number of Group Operating Companies. ERD EM/039 provides an example of the comprehensive criteria applied in the North Sea. These criteria are to some extent specific to the practices and conditions in the area; however, they provide a useful basis for developing criteria for other areas.

COMM. C.6 ACCIDENTAL LOADS

Replace this section with:

Comm. C.6.1.2 Collision Events

For platforms which are not located close to major shipping lanes, the velocity and size of vessel used for impact analyses will be representative of the vessels used in the operation and servicing of the platform (e.g. supply boats). By way of example, for the northern North Sea a vessel mass of 5000 tonnes is commonly used, whereas in the southern North Sea a mass of 2500 tonnes is normally used.

The two energy levels specified for vessel impact analysis represent a serviceability condition and an ultimate condition. The low energy level represents a serviceability condition based on economic considerations, and is intended to ensure that the structure will not require to be shut down after minor collisions. The high energy level represents an ultimate condition in which progressive collapse must not occur and the safety of personnel must be assured, although the structure may be substantially damaged.

For low energy impacts a vessel velocity of 0.5 m/s is commonly used, representing a minor accidental 'bump' during normal manoeuvring of the vessel upon approach or departure or while standing alongside the platform. For high energy conditions a vessel velocity of 2.0 m/s is commonly used, representing a vessel drifting out of control in a limiting seastate with significant wave height around 4.0 metres. EP 89-0230 provides guidance on determining vessel velocities for different limiting seastates

Comm. C.6.1.3. Collision Process

The majority of ship impact analyses performed to date have used quasi-static methods. However, when the duration of the collision process (i.e. the time during which the vessel is in contact with the structure) is of the same order as or less than the natural periods governing the structure's response, then dynamic effects may be significant. In such cases an assessment of dynamic behaviour during the collision should be considered. The collision duration is dependent on the size and configuration of the structure and the vessel and on the nature of the collision.

Comm. C.6.1.4 Structural Performance

After a high energy impact, where it can be assured that the platform can be shut-down and evacuated in a reasonable time after the collision (e.g. within one hour), then the environmental conditions used for checking post-impact strength would be those representing the limiting seastate for supply boat operations. If a longer time is required then a more severe seastate should be considered.

The post high energy impact requirements provided here are intended to assure the safety of personnel on the platform. From an economic point of view, consideration could also be given to designing the structure so that in an unmanned, shut down condition it can survive the environmental conditions which may be expected to occur during the period required to plan and implement a repair.

COMMENTARY F FATIGUE

Delete this section.

COMMENTARY G FOUNDATION DESIGN

COMM. G.4 AXIAL PILE CAPACITY IN CLAY

Rename this section as:

COMM. G.4 PILE CAPACITY FOR AXIAL BEARING LOADS

Existing G.4 commentary to be titled as:

Comm. G.4.2 Skin Friction and End Bearing in Cohesive Soils

Add new clause:

Comm. G.4.3 Shaft Friction and End Bearing in Cohesionless Soils

The current edition of API RP 2A-LRFD recommends the use of an earth pressure coefficient , K, of 0.8 when calculating the shaft friction of open ended driven piles in silica sands, both for tensile and compressive capacity. The value of 0.8 is based on the studies described in ASCE 1983. Prior to 1984, API RP 2A recommended K values of 0.5 and 0.7, respectively for tensile and compressive loading. The substantial increase of 60% in tensile capacity has a significant impact on the design of piles in sand. Hence additional studies have been performed including new test data, and documented in OTC 6422. The conclusions of the latter studies are that the current API method is not reliable for loose or very dense sand sites. For long piles in particular the method overpredicts pile capacity. Hence it is recommended to apply an earth pressure coefficient equal to 0.5 and 0.7, respectively for tensile and compressive bearing capacity calculations of open ended pipe piles.

COMM. G.13 STABILITY OF SHALLOW FOUNDATIONS: supplemental alternatives

Rename this section as:

COMM. G.13 MUDMAT BEARING CAPACITY

Under '1. Bearing Capacity', in first sentence, replace '...G.13-1 through G.13-6...', with '...G.13-3 and G.13-4...'.

Replace second and third paragraph with: 'Equations G.13-3 and G.13-4 use the factors N_c, N_q and N_γ. Figure Comm. G.13-1 provides a plot and tabulation of these factors for varying friction angles, ϕ'.'

Under '3. Correction Factors', replace all text before 'Inclination Factors:' with:

3. Correction Factors. The correction factors K_c , K_q and K_γ are usually written:

$$K_c = i_c \cdot s_c \cdot d_c$$
$$K_q = i_q \cdot s_q \cdot d_q$$
$$K_\gamma = i_\gamma \cdot s_\gamma \cdot d_\gamma \quad \text{(Comm.G.13-11)}$$

where i, s and d are individual correction factors related to load inclination, foundation shape and embedment depth respectively. The subscripts c, q and γ identify the factor (N_c , N_q and N_γ) with which the correction term is associated.

Delete section under 'Base and Ground Surface Inclination Factors:', up to and including FIG. COMM. G.13-4.

Under '4. Applications and Limitations.', delete last sentence in first paragraph and delete second, third and fourth paragraphs.

Under '5. Special Considerations.', in first sentence, replace '..G.13-8..', with '..G.13-6..'.

Delete COMM. sections G.14, G.15 and G.17.

COMMENTARY H STRUCTURAL COMPONENTS AND SYSTEMS

COMM. H.1 SUPERSTRUCTURE DESIGN

Delete this section.

COMM. H.4 GROUTED PILE-TO-STRUCTURE CONNECTIONS

Comm. H.4.3.4 Other Design Methods

Delete this clause.

COMM. H.5 CONDUCTORS

Delete this section.

COMM. H.6 GUYLINE SYSTEM DESIGN

Delete this section.

COMMENTARY I MATERIAL

COMM. I.2 STRUCTURAL STEEL PIPE

Renumber and rename this section as:

COMM. I.3 FABRICATED STEEL TUBULARS AND SECTIONS

COMMENTARY K WELDING

Delete this section

COMMENTARY L FABRICATION

Add new section:

API RP 2A-LRFD Sections K, L and N provide guidance relating to welding, fabrication and inspection which is intended to ensure that these are compatible with the design of the structure. This includes issues which the designer should be aware of in order to produce a design which can be efficiently fabricated and measures which should be implemented during fabrication to ensure that the design intent is fulfilled. These sections cannot be considered as constituting a complete fabrication specification. The provisions of these sections are also in some respects specific to the practices on design, materials and fabrication in the USA.

Consequently, Sections K, L and N of this DEP include the provision that a suitable fabrication specification should be prepared or established during design addressing all issues necessary to ensure that the fabrication, welding and inspection, are compatible with the design. The provisions of API RP 2A-LRFD provide useful guidance in this respect; however, their suitability should be carefully evaluated for the specific application.

COMMENTARY O SURVEY

Add new section:

API RP 2A-LRFD Section O provides guidance relating to in-service surveys. Some of its provisions are specific to the structures, practices, equipment and environment in the USA. This should be considered by users of this DEP.

COMMENTARY Q MINIMUM STRUCTURES

COMM. Q.2 DESIGN LOADS AND ANALYSIS

Add before last paragraph:

Any minimum structure which cannot be classified as a low consequence of failure structure is subject to the provisions of Section B.6.

PART III TYPOGRAPHICAL CORRECTIONS TO API RP 2A-LRFD

Equation C.3-8	Second line should read: $0.15\ (z/z_s)^{-0.275}$ for $z>z_s$
Equation D.3.2-1	f_e should read f_c (two places)
Equation D.3.2-3	should read: $f_c < \phi_c F_{xc}$
Equation E.3-1	After the definition of F_y add: NOTE: The tensile strength limitation on F_y is intended to apply throughout Section E
Comm. C.3.1	In fourth paragraph third sentence L_2 should read L_1

PART IV REFERENCES

In this DEP reference is made to the publications listed in the following. These references supplement those in API RP 2A-LRFD.

NOTE: 1. Unless specifically designated by date, the latest edition of each publication shall be used together with any amendments/supplements/revisions thereto.

SHELL STANDARDS

Index to DEP publications and standard specifications	DEP 00.00.05.05-Gen.
The use of SI units	DEP 00.00.20.10-Gen.
Painting and coating for new construction projects	DEP 30.48.00.31-Gen.
Structural steel for offshore applications	DEP 37.19.10.30-Gen.
Design of cathodic protection systems for offshore structures	DDD 37.19.30.30-Gen.
Practice for the analysis and design of marine conductors	EP 87-0160
Practice for the dynamic analysis of fixed offshore platforms for extreme storm conditions	EP 87-0170
Practice for the fatigue analysis of steel substructures for fixed offshore platforms	EP 93-2005
Practice for the assessment of vortex-induced vibrations of structural members	EP 93-0455
Practice for calculation of an extreme inertial load for steel substructures for fixed offshore platforms	EP 93-2525

AMERICAN STANDARDS

Recommended Practice for planning, designing and constructing fixed offshore platforms - Load and resistance factor design	API RP 2A-LRFD First edition, July 1, 1993

Issued by:
American Petroleum Institute
Publications and Distribution Section
1220 L Street
NW Washington DC 20005
USA.

BRITISH STANDARDS

Offshore installations: Guidance on design, construction and certification, Fourth Edition, 1990	DEn

Issued by:
HMSO Publications Centre
PO Box 276
London SW8 5DT
United Kingdom.

INTERNATIONAL STANDARDS

Quality management and quality assurance standards - Guidelines for selection and use	ISO 9000

Issued by:
Central Secretariat of ISO
1 rue de Varembé
1211 Geneva 20
Switzerland.

NORWEGIAN STANDARDS

Rules for the design, construction and inspection of offshore structures, 1977, Appendix B, Loads, (Reprint with corrections 1979)	DNV

Issued by:
Det Norske Veritas
Veritasveien 1
PO Box 300
Norway.

BIBLIOGRAPHY

NOTE: The documents listed in this Bibliography are for information only and do not form an integral part of this DEP.

NEWWAVE and current blockage: Their implementation and application within SESAM, February 1994	EP 94-0161
Design for offshore steel structures exposed to accidental loads, VERITEC, August 1988, (published as an SIPM EP report)	EP 89-0230
Interim guidance notes for the design and protection of topside structures against explosion and fire, The Steel Construction Institute, January 1992	SCI-P-112
Code checks exceeding unity: The refined checking of various structural components	EP 92-0595
The effect of Increasing Strength with Depth on the Bearing Capacity, Geotechnique, 1973, Vol 23, No. 4	DAVIS AND BOOKER
Weight engineering principles and procedures, Shell Expro Engineering Reference Document	ERD EN/001
A study of theoretical aspects of slamming, Ridley, National Maritime Institute Report 158, November 1982	NMI R 158
Guidelines for lifting point design, fabrication and inspection, Shell Expro Engineering Reference Document	ERD EM/039
Axial Capacity Of Steel Pipe Piles In Sand, Dennis and Olson, Proceedings of Conference on Geotechnical Practice In Offshore Engineering, American Society of Civil Engineers, 1983	ASCE 1983
An Appraisal of API RP 2A-LRFD Recommendations for Determining Skin Friction of Piles in Sand,	OTC 6422
Toolan, Lings and Mizra, Proceedings of the Offshore Technology Conference, Houston, Paper 6422	

FIGURES

FIGURE G.13-1 SOIL RESISTANCE ENVELOPE

Q ultimate vertical bearing capacity of mudmat determined using equation G.13-1 or G.13-2.

H horizontal component of bearing capacity in conjunction with **Q**.

V vertical foundation reaction from factored gravitional load.

E_F environmental load vector, $E_F = \sqrt{(V_E^2 + H_E^2)}$.

V_E vertical foundation reaction from factored environmental load.

H_E horizontal foundation reaction from factored environmental load.

$R_{V,H}$ soil resistance vector, $R_{V,H} = \sqrt{[(Q-V)^2 + H^2]}$

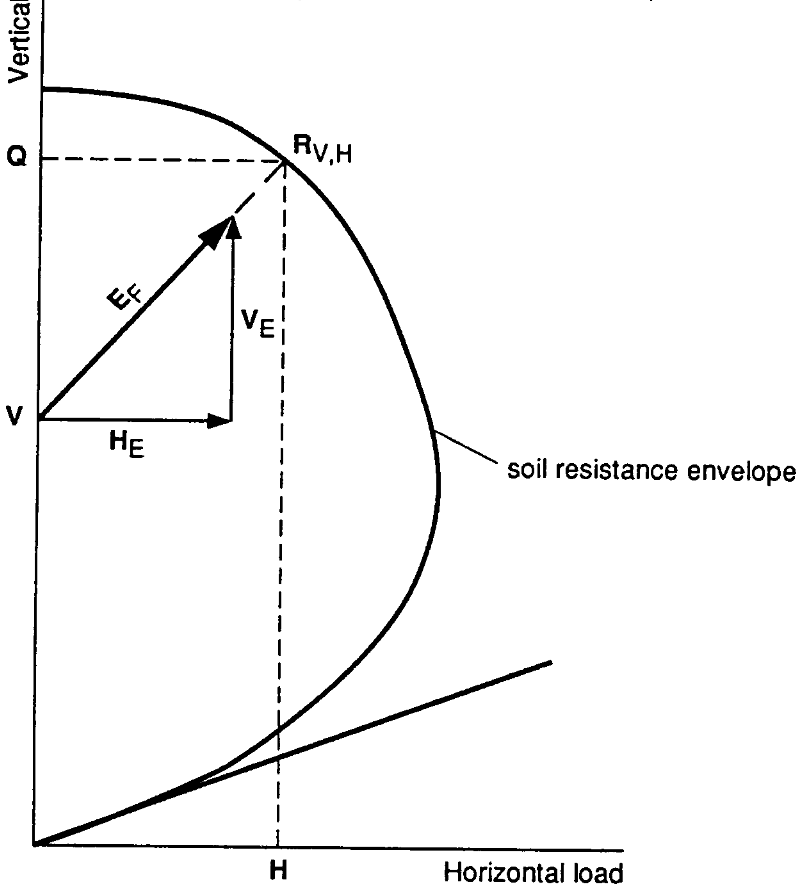

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FIGURE K.2-1 ILLUSTRATION OF JOINT REFERENCE THICKNESSES