ENGINEERING AND CONSTRUCTION STANDARD

FOR

OFFSHORE INSTALLATIONS

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1. SCOPE

This Standard contains Engineering Design Principles and good practices that have evolved during the development of offshore oil resources. It also discusses the foundations and structural components of offshore platforms together with materials and installation procedures in general terms, giving references to authentic international sources, wherever deemed necessary. Other structural components are covered under topside items and fittings.

2. REFERENCES

In this Standard the following standards and codes have been referred to and to the extent specified form a part of this Standard.

2.1 National Standards

API (AMERICAN PETROLEUM INSTITUTE)

RP 2A, 1989	"Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms"

AISC (AMERICAN INSTITUTE OF STEEL CONSTRUCTION, INC.)

Manual of Steel Construction, 9th Edition	"Specification for Structural Steel Buildings"

AWS (AMERICAN WELDING SOCIETY)

AWS D 1.1-88	"Structural Welding Code"

LRS (LLOYD'S REGISTER OF SHIPPING)

"Rules and Regulations for the Classification of Fixed Offshore Installation", July 1988

2.2 Iranian Petroleum Standards

IPS-E-CE-120	"Foundations"
IPS-E-CE-130	"Piles"
IPS-E-CE-500	"Loads"
IPS-G-CE-470	"Onshore Facilities"
IPS-E-TP-720	"Electrochemical Protection"

3. SYMBOLS AND ABBREVIATIONS

For detailed definition of symbols used in the text and formulas of this Standard refer to Pages 5-201 thru 5-206 of AISC "Manual of Steel Construction", 9th Edition, 1989.

4. UNITS

This Standard is based on International System of Units (SI), except where otherwise specified.

5. ENVIRONMENTAL CONSIDERATIONS

The following environmental factors, where applicable, are to be considered in the design of offshore facilities. For detailed information refer to Lloyd's Register, Part 3, Chapter 1, Section 2:

- Water depth
- Wind
- Waves
- Current
- Marine fouling
- Air and sea temperatures
- Snow and ice accretion
- Earthquake
- Wave/current interaction
- Directionality

6. SITE INVESTIGATION

6.1 The foundation design for offsite structures is to be based on the geotechnical conditions found at the actual site.

6.2 Sufficient appropriate laboratory and in-situ tests are to be performed in order to determine the strength and deformation characteristics of the strata underlying the seabed, and a full report of all tests is to be prepared.

6.3 Site investigation data and laboratory testing results should cover:

a) Seabed topography.
b) Nature and stability of seabed surface.
c) Geomorphology and engineering properties of the strata underlying the seabed.

The extent of investigations is to be sufficient in area, depth and detail to adequately cover the foundation design and engineering for the installation.

6.4 Methods of Investigation

6.4.1 The site investigation is to include combinations of the following methods where appropriate to the type and size of structure to be installed and the soil and seabed conditions anticipated:

a) Detailed bathymetric and side-scan sonar surveys.
b) Shallow seismic reflection surveys.
c) Shallow seabed sampling.
d) Borehole sampling.
e) Shallow and deep in-situ cone penetrometer testing.
f) In-situ testing, such as remote vane, radioactive borehole logging, pressuremeter and other proven methods.

6.4.2 The methods of investigation are to be adequate to give reliable information on the following:

a) Seabed topography in sufficient detail.
b) Presence of sand waves.
c) Surface deposits, rock outcrops and debris.
d) Variations in subgrade conditions within the sites of structures.
e) Stability of sloping seabeds.
f) Natural eruptions and erosions of the seabed due to emissions of gas, fresh water springs, etc.
g) Presence of shallow gas.

For general guidelines on site investigation refer to Lloyd's Register, Part 3, Chapter 2, Section 3.

7. DESIGN LOADS

The following loads and their appropriate combinations are to be considered in the design of the offshore installations:

a) Functional loads.
b) Environmental loads.
c) Loads in combination.
d) Construction and installation loads.
e) Accidental loads.

For detail description of design loads refer to IPS-E-CE-500 "Engineering Standard for Loads", Part II.

8. FOUNDATIONS

8.1 General

Foundations for offshore installations are either piled foundation or gravity (shallow) foundations.

8.2 Piled Foundations

Types of pile foundations used to support offshore structures are as follows:

8.2.1 Driven piles

Open ended piles are commonly used in foundations for offshore platforms. These piles are driven into the sea-floor with impact hammers which use steam, diesel fuel, or hydraulic power as the source of energy.

8.2.2 Drilled and grouted piles

This type of piles can be used in soils which will hold an open hole with or without drilling mud.

8.2.3 Belled piles

Bells may be constructed at the tip of piles to give increased bearing and uplift capacity through direct bearing on the soil. Drilling of the bell is carried out through the pile by underreaming with an expander tool. For detailed description of pile types refer to API Recommended Practice 2A (RP 2A), Section 6.

8.3 Pile Design

8.3.1 Foundation size

When sizing a pile foundation, the following items should be considered: diameter, penetration, wall thickness, type of tip, spacing, number of piles, geometry, location, mudline restraint, material strength, installation method, and other parameters as may be considered appropriate.

8.3.2 Foundation response

A number of different analysis procedures may be utilized to determine the requirements of a foundation. At a minimum, the procedure used should properly simulate the nonlinear response behavior of the soil and assure load-deflection compatibility between the structure and the pile-soil system.

8.3.3 Deflections and rotations

Deflections and rotations of individual piles and the total foundation system should be checked at all critical locations which may include pile tops, points of contraflecture, mudline, etc. Deflections and rotations should not exceed serviceability limits which would render the structure inadequate for its intended function.

8.3.4 Pile penetration

The design pile penetration should be sufficient to develop adequate capacity to resist the maximum computed axial bearing and pullout loads with an appropriate factor safety. The allowable pile capacities are determined by dividing the ultimate pile capacities by appropriate factors of safety which should not be less than the following values:

LOAD CONDITION	FACTORS OF SAFETY
1) Design environmental conditions with appropriate drilling loads	1.5
2) Operating environmental conditions during drilling operations	2.0
3) Design environmental conditions with appropriate producing loads	1.5
4) Operating environmental conditions during producing operations	2.0
5) Design environmental conditions with minimum loads (for pullout)	1.5

8.3.5 For detailed description of pile capacity for axial bearing and pullout loads, together with axial pile performance refer to API Recommended Practice 2A (RP 2A) Section 6.4, 6.5 and 6.6 respectively.

8.3.6 Soil reaction for laterally loaded piles

The pile foundation should be designed to sustain lateral loads, whether static or cyclic. Additionally overload cases in which the design lateral loads on the platform foundation are increased by an appropriate safety factor should be considered. The lateral resistance of the soil near the surface is significant to pile design, and the effects on this resistance of scour and soil disturbance during pile installation should be considered.

Generally, under lateral loading, clay soils behave as a plastic material which makes it necessary to relate pile-soil deformation to soil resistance. To facilitate this procedure, lateral soil resistance deflection (p-y) curves should be constructed using stress-strain data from laboratory soil samples.

For more detailed study of the construction of p-y curves refer to Section 6.7 of API Recommended Practice 2A (RP 2A), 1989.

8.3.7 Pile group action

Consideration should be given to the effects of closely spaced adjacent piles on the load and deflection characteristics of pile groups, since the ultimate axial capacity of a group can be less than the sum of the individual capacities.

The surface area of soil to pile shear should be maximized, and the area of soil shear should be minimized, in order to give the most conservative solution. The group capacity should not exceed the sum of the individual pile capacities. Due consideration should be given to the possibility of weak soil layers existing below the base of the pile or group.

8.3.8 Pile wall thickness

The wall thickness of the pile may vary along its length and would be controlled at a particular point by anyone of several loading conditions or requirements which are discussed in Section 6.9 of API Recommended Practice 2A (RP 2A), 1988.

8.3.9 Length of pile sections

In selecting pile section lengths consideration should be given to:

1) The capability of the lift equipment to raise, lower and stab the sections,

2) the capability of the lift equipment to place the pile driving hammer on the sections to be driven,

3) the possibility of a large amount of downward pile movement immediately following the penetration of a jacket leg closure,

4) stresses developed in the pile section while lifting,

5) the wall thickness and material properties at field welds,

6) avoiding interference with the planned concurrent driving of neighboring piles, and

7) the type of soil in which the pile tip is positioned during driving interruptions for field welding to attach additional sections. In addition, static and dynamic stresses due to the hammer weight and operation should be considered.

8.4 Shallow Foundations

Shallow foundations are those foundations for which the depth of embedment is less than the minimum lateral dimension of the foundation element. The design of shallow foundations should include, where appropriate to the intended application, consideration of the following:

1) Stability, including failure due to overturning, bearing, sliding or combinations thereof.

2) Static foundation deformations, including possible damage to components of the structure and its foundation or attached facilities.

3) Dynamic foundation characteristics, including the influence of the foundation on structural response and the performance of the foundation itself under dynamic loading.

4) Hydraulic instability such as scour or piping due to wave pressures, including the potential for damage to the structure and for foundation instability.

5) Installation and removal, including penetration and pull out of shear skirts or the foundation base itself and the effects of pressure build up or draw down of trapped water underneath the base.

8.4.1 Stability of shallow foundations

For evaluation of the stability and static deformation of shallow foundations refer to Sections 6.12 and 6.13 of of API Recommended Practice 2A (RP 2A), 1988.

8.4.2 Dynamic behavior of shallow foundations

Dynamic loads are imposed on a structure-foundation system by current, waves, ice, wind, and earthquakes. Both the influence of the foundation on the structural response and the integrity of the foundation itself should be considered.

8.4.3 Hydraulic instability of shallow foundations

8.4.3.1 Scour

Positive measures should be taken to prevent erosion and undercutting of the soil beneath or near the structure base due to scour. Examples of such measures are:

1) Scour skirts penetrating through erodible layers into scour resistant materials or to such depths as to eliminate the scour hazard, or

2) riprap emplaced around the edges of the foundation.

8.4.3.2 Piping

The foundation should be so designed to prevent the creation of excessive hydraulic gradients (piping conditions) in the soil due to environmental loadings or operations carried out during or subsequent to structure installation.

8.4.4 Installation and removal of shallow foundations

Installation should be planned to ensure the foundation can be properly seated at the intended site without excessive disturbance to the supporting soil. Where removal is anticipated an analysis should be made of the forces generated during removal to ensure that removal can be accomplished with the means available.

9. STEEL STRUCTURES

9.1 Structural Requirements

9.1.1 General

The platform should be designed so that all members are proportioned for basic allowable stresses specified by AISC "Specification for Structural Steel Building", 9th Edition, 1989.

Note:

The AISC Load and Resistance Factor Design, First Edition code is not recommended for design of offshore platforms.

9.1.2 Application

The requirements of this Clause are to be considered for the design of the jacket structure, module support frame and topside structure. Where topside structure is subject to particular requirements, (e.g., padeyes, helideck, etc.) these are given in sub-clause 9.2. The requirements for materials are given in Clause 10.

9.1.3 Structural analysis and design

a) Finite element analysis

- In general, the primary members of the structure are to be analysed by a three dimensional finite element method. Alternative methods of analysis will be considered if they are shown to be adequate for the structure under consideration.

- The finite element model and the element types used in the computer program for the analysis, are to be sufficiently representative of the primary components of the structure to enable accurate determination of member forces and displacements.

b) Elastic method

- The methods of analysis and the associated assumptions are to be consistent with the overall design principles.

- In general, the structure may be designed on the basis of linear elastic theory using static methods of analysis provided that any non-linear interaction between soil and structure is accounted for.

c) Plastic method

Where plastic methods of design are employed, the properties of the steel and the welding materials specified are to be such that they minimize the possibility of brittle fracture, allow for the formation of plastic hinges with sufficient rotational capacity, and provide sufficient fatigue resistance.

d) Loading combinations

Loading combinations covering all anticipated aspects of usage during the life span of the installation are to be considered so that the critical design cases are established, see IPS-E-CE-500, "Loads", Part II.

9.1.4 Factors of safety

a) Elastic method of design

Where the elastic method of design is used for structural steel, the member maximum allowable stress for loading conditions as detailed in IPS-E-CE-500 and for structural members as described in Clause 9.2 is to be based on the minimum factors of safety detailed in Table 1.

TABLE 1 - BASIC FACTORS OF SAFETY

LOADING CONDITIONS	SAFETY FACTORS
1 - Operating Conditions	Basic Factors of Safety
For shear (based on the "Yield Stress")	2.50
For shear buckling (based on the Shear Buckling Stress)	1.67
For axial tension (based on the "Yield Stress")	1.67
For axial compression where yielding controls (based on the Steel Yield Stress adjusted for column and local buckling effects)	1.67
For tension and compression in bending (based on 1.1 times the "Yield Stress")	1.67
For the combined "Comparative " Stress, (based on the "Yield Stress")	1.43
For hoop compression under hydrostatic pressure (based on the lesser of the Least Hoop Buckling Stress or the "Yield Stress")	2.00
For hoop tension under Internal Liquid/Gas Hydrostatic Pressure (based on the "Yield Stress")	1.67
2 - Extreme Storm Conditions	Reduced Factors of Safety
For shear (based on the "Yield Stress")	1.88
For shear buckling (based on the Shear Buckling Stress)	1.25
For axial tension (based on the "Yield Stress")	1.25
For axial compression where yielding controls (based on the Steel Yield Stress adjusted for column and local buckling effects)	1.25
For tension and compression in bending (based on 1.1 times the "Yield Stress")	1.25
For combined "Comparative" stress (based on the "Yield Stress")	1.11
For hoop compression under hydrostatic pressure (based on the lesser of the Least Hoop Buckling Stress or the "Yield Stress")	1.50
3 - Earthquake Loading Conditions	Minimum Factors of Safety
For shear (based on the "Yield Stress")	1.72
For shear buckling (based on the Shear Buckling Stress)	1.00
For axial tension and combined "Comparative" stress (based on the "Yield Stress")	1.00
For axial compression where yielding controls (based on the Steel Yield Stress Adjusted for column and local buckling effects)	1.00
For tension and compression in bending (based on 1.1 times the "Yield Stress")	1.00
For hoop compression under hydrostatic pressure (based on the lesser of the Least Hoop Buckling Stress or the "Yield Stress")	1.20

For the boat impact case as given in IPS-E-CE-500, the structure may be designed against ultimate collapse with a factor of safety of 1.10. The combined comparative stress is to be determined where necessary from the following:

$$\sigma_{CC} = \sqrt{\sigma X^2 + \sigma Y^2 - \sigma X \sigma Y + 3T^2}$$

Where:

σX and σY = The direct stresses, (i.e., combined axial and bending, compression or tension) in the orthogonal X and Y directions respectively).

T = The combined shear stress due to torsion and/or bending in the X-Y plane.

b) Plastic method of design

Where the plastic method of design, (i.e., based on the ultimate strength) is used, the factors of safety or the load factors are to be in accordance with AISC Specification for Structural Steel Buildings, 9th Edition, 1989, Chapter N.

9.1.5 Structural member design

9.1.5.1 Basic allowable stresses

Where the elastic method of design is used, the member basic allowable stresses to be used for normal operating conditions are to be determined as detailed in this Sub-clause. For normal operating conditions see IPS Standard E-CE-500, "Loads".

The basic allowable stresses for non-cylindrical steel members, (i.e., I, H, channel and box shapes) are to be in accordance with AISC Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design (9th Edition, 1989).

The basic allowable stresses for cylindrical steel members are to be in accordance with API's RP 2A "Planning, Designing and Constructing Fixed Offshore Platforms", Section 3.

The basic allowable stresses are to be based on the minimum factors of safety as described in Table 1.

The basic allowable stresses are to be limited to the appropriate allowable buckling stresses where local or overall elastic instability is likely to occur before the member stresses reach their allowable limits.

Where member stresses are due to occurrences of extreme events except earthquakes (e.g., extreme storm) acting in combination with dead loads and/or imposed loads, the basic allowable stresses, as determined in this Clause may in general, be increased by up to one-third providing the following requirements are met:

a) The minimum factors of safety associated with the increased allowable stresses are not less than those in Part 2 of Table 1.

b) The member sizes calculated on the basis of the increased allowable stresses are not less than those required for normal operating conditions without any increase in the allowable stresses.

Where member stresses are due to earthquake loadings acting in combination with dead and imposed loads, the basic allowable stresses may in general be increased by up to two-thirds providing the associated factors of safety are not less than those in Part 3 of Table 1.

9.1.5.2 Stresses in connections

The design of the connections of structural members is to develop the strength required for effective load transmission between joined members. Joint details are to be such as to minimize stress concentration and constraint against ductile behavior.

Undue concentration of welding is to be avoided, and good access for the welding of structural components is to be provided. The strength of tubular joints is to be in accordance with the API's RP 2A, Section 4.

9.1.5.3 Structural response to earthquake loadings

The effects of earthquake loadings are to be considered for the design of installations located in seismically active areas (see IPS-E-CE-500 "Loads").

Two levels of earthquake loadings are to be considered. First, the strength level earthquake requires the installation to be adequately sized for strength and stiffness, to maintain all nominal stresses within their yield or buckling limits. Secondly, the ductility level earthquake requirements are intended to ensure that the installation has sufficient energy absorption capacity to prevent its collapse during rare intense earthquake motions, although structural damage may occur.

The strength and ductility design requirements are to be achieved in accordance with API's Code of Practice RP 2A, Section 2, Clauses 2.3.6 c and 2.3.6 d.

9.1.5.4 Fatigue design

9.1.5.4.1 General

All steel structures are to be capable of withstanding the fatigue loading to which they are subjected. In general, the designer will be expected to have performed a fatigue analysis on the platform's jacket. Other parts of the structure subjected to fatigue loading are also to be designed to meet the applied cyclic loading. The design fatigue life is to be at least the intended field life or 20 years, whichever is the greater, see also sub-clause 9.1.5.4.8.

9.1.5.4.2 Fatigue loading

The placing of horizontal levels or members in the vicinity of the still water level is to be avoided wherever possible since the fatigue loading is very much higher in this region.

Members in the vicinity of the still water level or penetrating the water line are to be designed for cyclic buoyancy forces and wave slamming loads in addition to cyclic wave loading due to drag and intertia. The effect of current on cyclic wave loading is to be included in the design.

Fatigue due to wind loading can be important for topside structures such as flare towers, drilling derricks, etc., and is to be taken into account in the design. Dynamic amplification could be significant for lattice tower structures and should be considered.

For platform jacket structures, the contribution of wind to the fatigue loading is small and can generally be neglected.

Where cyclic loading from other sources is present, (e.g., rotating machinery, temperature cycling, vortex shedding, etc.) this is to be considered in the fatigue design. For long duration tow-out fatigue could be significant and should be accounted for in the design.

9.1.5.4.3 Wave spectrum

Wherever possible the actual wave spectrum for the proposed location of the offshore installation should be used. An appropriate percentage of the total cumulative spectrum is to be used for each wave and current direction under consideration.

9.1.5.4.4 Dynamic amplification

For structures where the period of the first mode of vibration is less than 3 seconds, dynamic amplification of wave excited deformations need not be considered.

Where the period of the first mode of vibration of the structure is over 3 seconds, account is to be taken of the effects of dynamic amplification.

9.1.5.4.5 Fatigue analysis

Where the S/N curves given in this Standard are used (see 9.1.5.4.7), it may be assumed that only fluctuating stresses cause damage and the effect of mean stress may be neglected.

Experience indicates that nearly all fatigue failures on fixed platform structures occur at welded joints. However, it should not be assumed that only brace to chord connections (saddle welds) are prone to fatigue failure. The behavior of any weld subject to fatigue loading is to be considered.

9.1.5.4.6 Stress concentration factors (SCFs)

Peak local stress concentrations may be determined from specific experimental tests of analytical methods (e.g., joint finite element analysis).

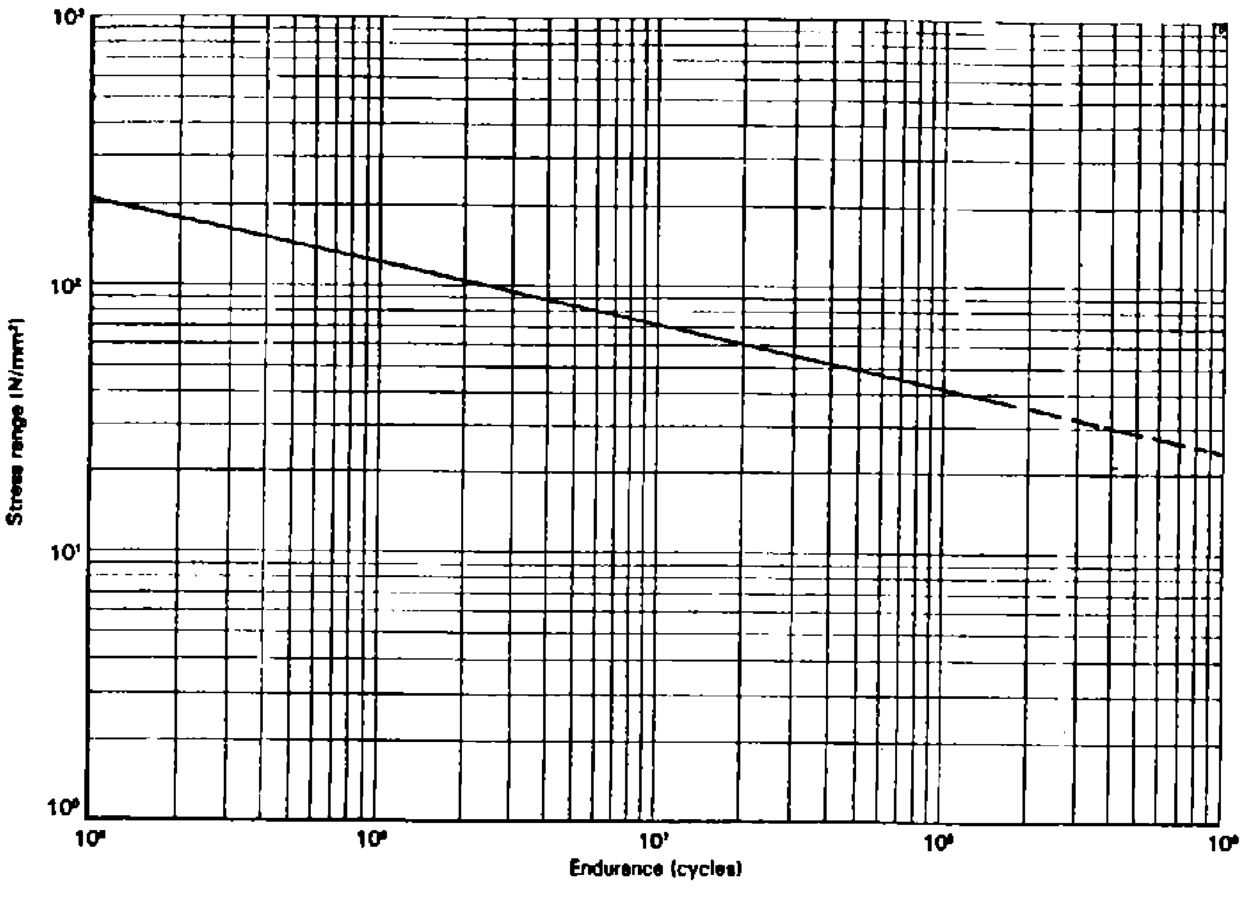
In the absence of SCFs derived from specific experimental tests or detailed joint analysis the brace and chord side fatigue lives may be estimated using SCFs obtained from parametric formulas. A minimum SCF of 1.5 is to be adopted when using parametric formula.

Due allowance, (i.e., suitable SCF) is to be made in butt joints and appurtenance connections for any eccentricity, mismatch or other geometric effect.

9.1.5.4.7 Allowable fatigue stresses (S/N curves)*

For brace to chord connections the S/N curve shown in Fig. 1 is to be used. For standard details (e.g., butt joints, appurtenance connections, reinforced holes, etc.) appropriate S/N curves are to be used.

*** Stress-number (or S/N) curve is obtained in fatigue tests by subjecting a series of specimens of a given material to different ranges of stress and plotting the range of stress against the number of cycles required to produce failure.**



STRESS/ENDURANCE CURVE
Fig. 1

9.1.5.4.8 Cumulative damage and fatigue life

Cumulative damage may be calculated using the Palmgren-Miner cumulative damage law. The design fatigue life is to be at least the intended field life, except for brace to chord connections using the S/N curve in Fig. 1, where the design fatigue life is to be at least twice the intended field life. In no case is the design fatigue life to be less than 20 years.

9.1.5.4.9 Method of analysis

The two basic methods of fatigue analysis that are available are Deterministic Fatigue Analysis and Spectral Fatigue Analysis. Both are acceptable to Iranian Petroleum Industries.

Deterministic method of fatigue analysis uses non-linear wave theory and takes full account of the wave profile. Deterministic analysis is generally considered most suitable for fixed platforms where dynamic excitation is not significant. Spectral method of fatigue analysis may be used if dynamic excitation of a structure is expected to be significant.

9.2 Topside Items and Fittings

9.2.1 Deck structure

In general, the design of topside structure is to be based on the requirements laid down in Clause 9. This Clause presents the requirements for topside items and fittings which warrant specialized examination by virtue of the purpose for which they are designed.

Topside items and fittings which do not fall into the categories covered by the sub-clauses of this Clause and are outside the scope of Clause 9 will be specially considered.

9.2.2 Design deck loadings

The design deck loadings are to be not less than those defined in IPS-E-CE-500, Part II. The scantlings of decks are to be sufficient to withstand the actual local loadings plus any additional loadings superimposed due to frame action within the stress limitations given in sub-clause 9.1.4, Table 1.

Deck plating is not normally required to contribute directly to the overall integrity of the structure and may be designed primarily to withstand the local imposed and dead loadings.

9.2.3 Decks loaded by wheeled vehicles

Where it is proposed to use wheeled vehicles such as fork lift trucks and mobile cranes on deck structures, the deck plating and the supporting structure are to be designed on the basis of the maximum loading to which they may be subjected in service and the minimum scantlings are to comply with this Standard.

The vehicle types and axle loads, for which the vehicle carrying decks have been approved are to be stated in the Operating Manual and be contained in a notice displayed on each deck.

Details of the deck loading resulting from the operation of wheeled vehicles are to be supplied by the designer. These details are to include the wheel load, axle and wheel spacing, tyre print dimensions and type of tyre for the vehicles.

For design purposes, where wheeled vehicles are to be used for handling stores, etc., on storage decks, the deck is to be taken as loaded with the appropriate design loading, except in way of the vehicle.

For the design of the deck plate thickness, t, and the section modules, Z, of deck stiffeners refer to Lloyd's Register, Part 4, 1988, Chapter 2, sub-section 1.3.5 thru 1.3.11.

9.2.4 Helicopter landing areas

This Clause gives the requirements for decks intended for helicopter operations. Attention is drawn to the requirements of National and other Authorities concerning the construction of helicopter decks and the landing area arrangements necessary for helicopter operations.

9.2.4.1 Plans and data

Plans and data are to be submitted giving the arrangements, scantlings and details of the helicopter deck. The type, size and weight of helicopters to be used are also to be indicated. Relevant details of the largest helicopters, for which the deck is designed, are to be stated in the Operating Manual.

9.2.4.2 Arrangements

The landing area is to comply with applicable regulations with respect to size, landing and take-off sectors, freedom from height obstructions, deck markings, safety nets and lighting, etc.

The landing area is to have an overall coating of non-slip material. A drainage system is to be provided in association with a perimeter guttering system or slightly raised kerb to prevent spilled fuel falling on to other parts of the installation. A sufficient number of tie-down points is to be provided to secure the helicopter.

9.2.4.3 Landing area plating

For designing the deck plate thickness, t, within the landing area, deck stiffening and supporting structure, reference is made to Lloyd's Register, Part 4, 1988, Chapter 2, sub-sections 2.4 , 2.5 and 2.6.

9.2.5 Bridges and access gangways

This sub-clause presents the requirements for the structural design of bridges between fixed installations and access gangways between fixed installations and mobile units.

The width of bridges between fixed installations, gangways between fixed installations and mobile units and access to them is to be not less than 1 meter.

Bridges between fixed installations and access gangways between fixed installations and mobile units are to be fitted with guard-rails complying with the requirements of sub-clause 9.2.7. The walkway should have a non-slip surface.

Bridges are to be designed in the same way as other topside structure with particular attention being given to environmental loads.

9.2.6 Access gangways between fixed installations and mobiles units

In addition to the effects of dead load, access gangways between fixed installations and mobile units are to satisfy the following load conditions:

a) Gangway deployed (outboard end supported on mobile installation):

- **i)** Maximum gangway operating wind load plus imposed loading of 3 kN/mm^2 applied to the walkway area.
- **ii)** Local loading on walkway of 4.5 kN/mm^2.

b) Gangway in stowed or stand-by position:

- **i)** 50 year return storm wind.
- **ii)** Maximum ice loading with appropriate wind as defined in IPS-E-CE-500.

c) Loss of gangway support at outer end:

i) The loading (a) (i) plus the loading resulting from the maximum gangway operating sea state from any direction.

ii) The gangway and its supports and rigging are to be designed for any dynamic loading resulting from this loss of support. The gangway is to remain in an attitude that will not diminish the safety of any persons on it.

The load cases in 9.2.6(a) are to be considered as "Normal load conditions" and those in 9.2. 6(b) and (c) are to be considered as "Extreme loading conditions" in the context of permissible stresses (see Table 1).

A minimum safety factor of five, against breaking, is to be used for any rigging wire rope subjected to the maximum loading conditions in 9.2. 6.

Access gangway are to be subjected to the following tests and subsequent thorough examination before being taken into use for the first time, or after any subsequent alternation or repair which may affect its strength.

a) Both ends of gangway supported:

The walkway area is to be tested to 1.25 times imposed loading (i.e. 3.75 kN/m²). This test may take place at the manufacturer's premises.

b) If loss of gangway end support is a design case, see 9.2. 6(c) then the walkway area is to be tested, after installation, with the end unsupported to 1.25 times imposed loading (i.e. 3.75 kN/m²).

Access gangway are to be fitted with a visual and audible alarm system, routed to a manned control point and set to alarm if gangway movements approach the design limit for normal operations.

9.2.7 Guard-rails and ladders

The requirements may be modified to take account of relevant IPC's Regulations, where these exist.

9.2.7.1 Guard-rails

The edges of every floor, gangway and stairway, and all openings down which a person could fall more than 2 m or into the sea, are to be fitted with guard-rails constructed of steel and of an approved type.

Where it is not practicable to fit guard-rails, special consideration will be given to the following alternative means of protection:

a) Safety nets or safety sheets.
b) Lifelines and safety belts.
c) Life-jackets.

Guard-rails and their supports are to be designed to withstand a horizontal loading of 0.74 kN/m applied at the top rail. The stresses derived using this loading may be considered as "Extreme Storm Conditions" in the context of allowable stresses. See Table 1. The minimum height of guard-rails is to be 1 meter and stanchions are to be not more than 2 m apart.

Guard-rails are normally to be constructed with three courses and where practicable are to be fitted with a toeboard not less than 150 mm high.

The gap between courses is not to exceed 40 mm and the gap between the lowest course and the top of the toeboard is not to exceed 150 mm.

Wire mesh of sufficient strength and suitably protected against corrosion may be substituted for guard-rails below the top rail, except in the splash zone.

Guard-rails are not to be used as support points for any other item of equipment or structure unless specifically reinforced.

For stairways which are protected on either side by a wall, the minimum requirement is for a single handrail to be fitted to each wall.

Guard-rails located in the splash zone, may have the rails below the top rail substituted by adequately tensioned wires of sufficient strength.

Guard-rails for purposes other than personnel protection, e.g., crash barriers, etc., will be specially considered.

9.2.7.2 Ladders

Fixed ladders are to be designed to the general requirements of an approved Code of Practice which gives acceptable strength and deflection criteria, e.g., BS MA 39: Part 1: 1973. Whichever Code of Practice is used, ladder rungs are to pass through and be welded to both faces of their support stringers.

All ladders over 6 m in height (including those incorporated in equipment) and not intended for use solely in cases of emergency, are to be protected either by safety cages or, if this is not possible, by efficient safety harness devices. Where provided, safety cages are to extend from 2.5 m above the bottom of the ladder.

Rest platforms are to be provided, where practicable at not more than 9 m intervals. Safety gates or an acceptable equivalent are to be fitted at the tops of ladders.

9.2.8 Machinery supports

For general information refer to Lloyds Register, Part 7.

9.2.9 Davits

Davits and launching appliances are to comply with the requirements of the relevant National or International Standards.

They are to be efficiently supported and the structure supporting them is to be capable of withstanding the imposed loads including allowance for the recommended factor of safety.

10. MATERIALS

10.1 Structural Steel

10.1.1 General

Steel should conform to a definite specification and to the minimum strength level, group and class specified by the designer. Certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM A6 or A20, as applicable to the specification listed in Table 2, constitutes evidence of conformity with the specification. Unidentified steel should not be used.

10.1.2 Steel groups

Steel may be grouped according to strength level and welding characteristics as follows:

10.1.2.a Group I designates mild steels with specified minimum yield strengths of 280 MPa or less. Carbon equivalent is generally 0.40% or less, and these steels may be welded by any of the welding processes as described in AWS D1.1.

10.1.2.b Group II designates intermediate strength steels with specified minimum yield strengths of over 280 MPa through 360 MPa. Carbon equivalent ranges of up to 0.45% and higher, and these steels require the use of low hydrogen welding processes.

10.1.2.c Group III designates high strength steels with specified minimum yield strengths in excess of 360 MPa. Such steels may be used provided that each application is investigated with regard to:

1) Weldability and special welding procedures which may be required.
2) Fatigue problems which may result from the use of higher working stresses, and
3) Notch toughness in relation to other elements of fracture control, such as fabrication, inspection procedures, service stress, and temperature environment.

10.1.3 Steel classes

Consideration should be given for the selection of steels with notch toughness characteristics suitable for the conditions of service. For this purpose, steels may be classified as follows:

10.1.3.a Class C steels are those which have a history of successful application in welded structures at service temperatures above freezing, but for which impact tests are not specified. Such steels are applicable to primary structural members involving limited thickness, moderated forming, low restraint, modest stress concentration, quasi-static loading (rise time 1 second or longer) and structural redundancy such that an isolated fracture would not be catastrophic. Examples of such applications are piling, jacket braces and legs, and deck beams and legs.

10.1.3.b Class B steels are suitable for use where thickness, cold work, restraint, stress concentration, impact loading, and/or lack of redundancy indicate the need for improved notch toughness. Where impact tests are specified, Class B steels should exhibit Charpy V-notch energy of J for Group I, and 34 J for Group II, at the lowest anticipated service temperature. Steels enumerated herein as Class B can generally meet these Charpy requirements at temperatures ranging from, 10° to 0°C. When impact tests are specified for Class B steel, testing in accordance with ASTM A 673, frequency H, is suggested.

10.1.3.c Class A steels are suitable for use at subfreezing temperatures and for critical applications involving adverse combinations of the factors cited above. Critical applications may warrant Charpy testing at 20-30°C below the lowest anticipated service temperature. This extra margin of notch toughness prevents the propagation of brittle fractures from large flaws, and provides for crack arrest in thicknesses of several inches. Steels enumerated herein as Class A can generally meet the Charpy requirements stated above at temperatures ranging from -20° to -40°C. Impact testing frequency for Class A steels should be in accordance with the specification under which the steel is ordered; in the absence of other requirements, heat lot testing may be used.

10.1.4 Structural shape and plate specifications

Unless otherwise specified by the designer, structural shapes and plates should conform to one of the specifications listed in Table 2. Steels above the thickness limits stated may be used, provided applicable provisions of Section 10.1.2.c are considered by the designer.

10.2 Structural Steel Pipe

10.2.1 Specifications

Unless otherwise specified, seamless or welded pipe[1] should conform to one of the specifications listed in Table 3. Pipe should be prime quality unless the use of limited service, structural grade, or reject pipe is specifically approved by the designer.

1) With longitudinal welds and circumferential butt welds.

10.2.2 Fabrication

Structural pipe should be fabricated in accordance with API Spec. 2B, ASTM A139(1), ASTM A252(1), ASTM A381, or ASTM A671 using grades of structural plate listed in Table 2 except that hydrostatic testing may be omitted.

Note:

1) With longitudinal welds and circumferential butt welds.

10.2.3 Selections for conditions of service

Consideration should be given for the selection of steels with toughness characteristics suitable for the conditions of service. For tubes cold-formed to D/t less than 30, and not subsequently heat-treated, due allowance should be made for possible degradation of notch toughness, e.g., by specifying a higher class of steel or by specifying notch toughness test run at reduced temperature.

TABLE 2 - STRUCTURAL STEEL PLATES AND SHAPES

GROUP	CLASS	SPECIFICATION & GRADE	YIELD STRENGTH MPa	TENSILE STRENGTH MPa
I	C	ASTM A36 (to 50 mm Thick)	250	400-550
		ASTM A131 Grade A (to 13 mm Thick)	235	400-490
		ASTM A285 Grade C (to 20 mm Thick)	205	380-515
I	B	ASTM A131 Grades B, D	235	400-490
		ASTM A516 Grade 65	240	450-585
		ASTM A573 Grade 65	240	450-530
		ASTM A709 Grade 36T2	250	400-550
I	A	ASTM A131 Grade CS, E	235	400-490
II	C	ASTM A572 Grade 42 (to 50 mm Thick)	290	415 min.
		ASTM A572 Grade 50 (to 13 mm Thick[1])	345	450 min.
II	B	ASTM A709 Grade 50T2, 50T3	345	450 min.
		ASTM A131 Grade AH32	315	470-585
		ASTM A131 Grade AH36	350	490-620
II	A	API Spec. 2H Grade A2	290	430-550
		Grade 50 (to 60 mm Thick)	345	483-620
		(Over 60 mm Thick)	325	483-620
		API Spec. 2W Grade 42 (to 25 mm Thick)	290-462	427 min.
		(Over 25 mm Thick)	290-427	427 min.
		Grade 50 (to 25 mm Thick)	345-517	448 min.
		(Over 25 mm Thick)	345-483	448 min.
		Grade 50T (to 25 mm Thick)	345-522	483 min.
		(Over 25 mm Thick)	345-517	483 min.
		Grade 60 (to 25 mm Thick)	414-621	517 min.
		(Over 25 mm Thick)	414-586	517 min.
		API Spec. 2Y Grade 42 (to 25 mm Thick)	290-462	427 min.
		(Over 25 mm Thick)	290-427	427 min.
		Grade 50 (to 25 mm Thick)	345-517	448 min.
		(Over 25 mm Thick)	345-483	448 min.
		Grade 50T (to 25 mm Thick)	345-572	483 min.
		(Over 25 mm Thick)	345-517	483 min.
		ASTM A131 Grades DH32, EH32	315	470-585
		Grades DH36, EH36	350	490-620
		ASTM A537 Class I (to 60 mm Thick)	345	485-620
		ASTM A633 Grade A	290	435-570
		Grades C, D	345	485-620
		ASTM A678 Grade A	345	485-620
III	A	ASTM A537 Class II (to 60 mm Thick)	415	550-690
		ASTM A678 Grade B	415	550-690
		API Spec. 2W Grade 60 (to 25 mm Thick)	414-621	517 min.
		(Over 25 mm Thick)	414-586	517 min.
		API Spec. 2Y Grade 60 (to 25 mm Thick)	414-621	517 min.
		(Over 25 mm Thick)	414-586	517 min.
		ASTM 710 Grade A Class 3		
		(quenched and precipitation heat treated)		
		thru 50 mm	515	585
		50 mm to 100 mm	450	515
		over 100 mm	415	485

Note:

1) To 50 mm thick for Type 1, killed, Fine Grain Practice.

10.3 Steel for Tubular Joints

Tubular joints are subject to local stress concentrations which may lead to local yielding and plastic srains at the design load. During the service life, cyclic loading may initiate fatigue cracks, making additional demands on the ductility of the steel, particularly under dynamic loads. These demands are particularly severe in heavy-wall joint-cans designed for punching shear.

10.3.1 Underwater joints

For underwater portions of redundant template-type platforms, steel for joint cans (such as jacket leg joint cans, chords in major X and K joints, and through-members in joints designed as overlapping) should meet one of the following notch toughness criteria at the temperature given in Table 4.

1 NRL Drop-weight Test no-break performance.

2 Charpy V-notch energy: 20 Joules for Group I steels and 34 Joules for Group II steels, and for Group III steels transverse test.

TABLE 3 - STRUCTURAL STEEL PIPE

GROUP	CLASS	SPECIFICATION & GRADE	YIELD STRENGTH MPa	TENSILE STRENGTH MPa
I	C	API 5L Grade B[2]	240	415 min.
		ASTM A53 Grade B	240	415 min.
		ASTM A135 Grade B	240	415 min.
		ASTM A139 Grade B	240	415 min.
		ASTM A500 Grade A (round)	230	310 min.
		(shaped)	270	310 min.
		ASTM A501	250	400 min.
I	B	ASTM A106 Grade B (normalized)	240	415 min.
		ASTM A524 Grade I (thru 10 mm w.t.)	240	415 min.
		Grade II (over 10 mm w.t.)	205	380-550
I	A	ASTM A333 Grade 6	240	415 min.
		ASTM A334 Grade 6	240	415 min.
II	C	API 5L Grade X42 2% max. cold	290	415 min.
		expansion	360	455 min.
		API 5L Grade X52 2% max. cold	290	400 min.
		expansion	320	400 min.
		ASTM A500 Grade B, (round)	345	485 min.
		(shaped)		
II	B	ASTM A618	360	455 min.
II	A	API 5L Grade X52 with SR5, SR6 or SR8		
		See sub-clause 11.2.2		

Note:

2) Seamless or with longitudinal seam welds.

TABLE 4 - INPUT TESTING CONDITIONS

D/t	TEST TEMPERATURE	TEST CONDITION
over 30	20°C Below LAST[3]	Flat Plate
20-30	30°C Below LAST	Flat Plate
under 20	10°C Below LAST	As Fabricated

Note:

3) LAST = Lowest Anticipated Service Temperature.

For water temperature of 4°C or higher, these requirements may normally be met by using the Class A steels listed in Table 2.

10.3.2 Above water joints

For above water joints exposed to lower temperatures and possible impact from boats, or for critical connections at any location in which it is desired to prevent all brittle fractures, the tougher Class A steels should be considered, e.g., API Spec. 2H, Grade 42 or Grade 50. For 345 MPa yield and higher strength steels, special attention should be given to welding procedures.

10.3.3 Critical joints

For critical connections involving high restraint (including adverse geometry, high yield strength and/or thick sections), through-thickness shrinkage strains, and subsequent through-thickness tensile loads in service, consideration should be given to the use of steel having improved through-thickness (Z-direction) properties, e.g., API Spec. 2H, Supplement S4 and S5.

10.3.4 Brace ends

Although the brace ends at tubular connections are also subject to stress concentration, the conditions of service are not quite as severe as for joint-cans. For critical braces, for which brittle fracture would be catastrophic, consideration should be given to the use of stub-ends in the braces having the same class as the jointcan, or one class lower. This provision need not apply to the body of braces (between joints).

10.4 Cement Grout and Concrete

10.4.1 Cements, aggregates and additives

10.4.1.1 Cements for grouting may be ordinary portland, rapid-hardening portland, sulphate resisting portland, oil well or high alumina and should comply with IPS-M-CE-165, see 10.4.1.7 and 10.4.1.8.

10.4.1.2 Aggregates, if used, are assumed to be fine graded. Marine aggregates may be used but see 10.4.1.5 with reference to the chloride content.

10.4.1.3 Additives may be used but calcium chloride and chloride based additives are expressly forbidden.

10.4.1.4 Water is to be clean and free from any harmful matter. Sea water may be used, except with high alumina cement when only fresh water is acceptable, but see 10.4.1.5. When using sea water it should be drawn directly from the sea at a point remote from any waste outfalls or exhausts no more than one hour before use.

10.4.1.5 Although the specific use of calcium chloride is not allowed, due to the use of sea water or marine aggregate some may be present. This should be limited such that the calcium chloride content of the mixed grout, expressed as the equivalent percentage by weight of cement of anhydrous calcium chloride is not to exceed 1.0 percent. When using high alumina cement this value is not to exceed 0.1 percent.

10.4.1.6 Manufacturers' certificates of compliance with approved standards or codes are to be presented to AR* with all material deliveries. AR reserves the right to refuse the use of any materials that do not conform to specification.

10.4.1.7 Special cements and fillers, such as blast furnace slag or pozzolana will be considered by AR, should they be required in a specification.

10.4.1.8 Any deviation from conventional grout mixes may well require extensive tests being carried out. Fillers usually cause low strength gain and this would require strict attention to environmental conditions which could cause the joint to move whilst the grout is setting, and so affect the final strength.

10.4.1.9 If cement fillers are allowed in the mix they are to be pre-blended on shore prior to delivery to the offshore installation vessel.

10.4.1.10 Mix specification

For high alumina cement the water cement ratio should be not greater than 0.4.

10.4.2 Cement grout

If required by the design, the space between the piles and the surrounding structure should be carefully filled with grout. Prior to installation, the compressive strength of the grout mix design should be confirmed on a representative number of laboratory specimens cured under conditions which simulate the field conditions. Laboratory test procedures should be in accordance with ASTM 109. The unconfined compressive strength of 28 day old grout specimens computed as described in ACI 214-77 equating f'_c to f_{cu}, should not be less than either 17.25 MPa or the specified design strength.

A representative number of specimens taken from random batches during grouting operations should be tested to confirm that the design grout strength has been achieved. Test procedures should be in accordance with ASTM 109. The specimens taken from the field should be subjected, until test, to a curing regime representative of the situ curing conditions, i.e., underwater and with appropriate seawater salinity and temperature.

10.4.3 Concrete

The concrete mix used in belled piles should be selected on the basis of shear strength, bond strength and workability for underwater placement including cohesiveness and flowability. The concrete mix may be made with aggregate and sand, or with sand only. The water-cement ratio should be less then 0.45. If aggregate is used, the aggregate should be small and rounded, the sand content should be 45% or greater, the cement content should be not less than 445 kg/m^3, and the workability as measured by the slump test should be 180 to 230 mm. To obtain the properties required for proper placement, a suitable water-reducing and plasticizing admixture may be necessary.

10.4.4 Grouting procedures

10.4.4.1 Prior to mixing and pumping any grout, the primary and secondary grout lines leading to the annulus are to be flushed with water to ensure there are no blockages. If bottom packers are present in the annulus these should be inflated first. If top packers are present, initial flushing of the primary and secondary lines should take place prior to inflation to clear any foreign matter. The top packer should then be inflated and further flushing take place to ensure the exhaust pipe or return line is clear of obstructions.

*** AR = Authorized Representative of the Owner.**

10.4.4.2 The grout formulation employed will normally be mixed as a continuous process in a re-circulating jet mixer to the specified density. A calibrated pressurized slurry balance (as defined in API RP 10B) is to be used to establish not only the density of the grout itself, but also the calibration of any densiometers on the mixing plant. Grout that does not fall within the specified density tolerances is not to be pumped to the annulus.

10.4.4.3 Once the density is satisfactorily achieved pumping may commence and continue until the annulus is completed to the satisfaction of the AR.

10.4.5 Grouting restrictions

10.4.5.1 The following restrictions are intended as a guide, they may be increased or decreased by the AR depending on local conditions.

a) Pile driving

Ideally all piles should be driven prior to commencement of grouting. If this proves impossible, pile driving after grouting must be delayed until an agreed grout strength has been achieved, taking account of the grouted pile loading. This time delay will usually be a minimum of 48 hours.

b) Pile drilling

Where drilled piles are to be used a minimum delay of 12 hours after completion of grouting a pile in a cluster is to be maintained before drilling may re-commence on that cluster. The time may be increased for some grout mixes and also if the structure is particularly prone to vibration.

c) Environmental restrictions

As a minimum the specification is to indicate the maximum significant wave heights that can be tolerated on the structure within a 24 hour period of grouting the first pile on each corner leg.

10.4.5.2 Bottom packers are to stay inflated until the supported grout has reached a strength capable of being self-supporting. In the absence of other information the packers are to stay inflated for 12 hours.

10.4.5.3 When grout plugs are used to support the annulus grout, in the absence of further information, they should be left to cure for 18 hours.

10.5 Corrosion Protection

Unless specified otherwise by the designer, the systems for corrosion protection should be designed in accordance with NACE PR-01-76. Cathodic Protection should be designed in accordance with IPS-E-TP-720, "Electrochemical Protection".