OFFSHORE STRUCTURES
Analysis and Design

Dr. S. Nallayarasu
Associate Professor

Department of Ocean Engineering
Indian Institute of Technology Madras,
Chennai - 600036, India
<table>
<thead>
<tr>
<th>Contents</th>
<th>Contents</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.7.3 Foundation Concepts</td>
<td>21</td>
</tr>
<tr>
<td>2 DESIGN METHODOLOGY</td>
<td>23</td>
</tr>
<tr>
<td>2.1 General</td>
<td>23</td>
</tr>
<tr>
<td>2.2 Design Stages</td>
<td>24</td>
</tr>
<tr>
<td>2.2.1 FEED</td>
<td>24</td>
</tr>
<tr>
<td>2.2.2 Basic Design</td>
<td>24</td>
</tr>
<tr>
<td>2.2.3 Detailed Design</td>
<td>25</td>
</tr>
<tr>
<td>3 LOADS</td>
<td>29</td>
</tr>
<tr>
<td>3.1 General</td>
<td>29</td>
</tr>
<tr>
<td>3.2 Types of Loads</td>
<td>29</td>
</tr>
<tr>
<td>3.3 Gravity Loads</td>
<td>30</td>
</tr>
<tr>
<td>3.3.1 Dead Loads</td>
<td>30</td>
</tr>
<tr>
<td>3.3.2 Facility Dead Loads</td>
<td>30</td>
</tr>
<tr>
<td>3.3.3 Fluid Loads</td>
<td>30</td>
</tr>
<tr>
<td>3.3.4 Live Loads</td>
<td>31</td>
</tr>
<tr>
<td>3.3.5 Drilling Loads</td>
<td>31</td>
</tr>
<tr>
<td>3.4 Environmental Loads</td>
<td>31</td>
</tr>
<tr>
<td>3.4.1 Wind Loads</td>
<td>31</td>
</tr>
<tr>
<td>3.4.2 Wave and Current Loads</td>
<td>34</td>
</tr>
<tr>
<td>3.4.3 Current Profile</td>
<td>35</td>
</tr>
<tr>
<td>3.4.4 Marine Growth</td>
<td>36</td>
</tr>
<tr>
<td>3.4.5 Morison Equation</td>
<td>37</td>
</tr>
<tr>
<td>3.4.6 Wave-Current Interaction</td>
<td>37</td>
</tr>
<tr>
<td>Contents</td>
<td>Contents</td>
</tr>
<tr>
<td>------------------------------------------------------------------------</td>
<td>----------</td>
</tr>
<tr>
<td>3.4.7 Selection of Wave Theory</td>
<td>38</td>
</tr>
<tr>
<td>3.4.8 Wave Load on a Member</td>
<td>39</td>
</tr>
<tr>
<td>3.4.9 Maximum Global Loads</td>
<td>44</td>
</tr>
<tr>
<td>3.4.10 Buoyancy Loads</td>
<td>45</td>
</tr>
<tr>
<td>3.4.11 Ice Loads</td>
<td>47</td>
</tr>
<tr>
<td>3.4.12 Mud Loads</td>
<td>48</td>
</tr>
<tr>
<td>3.5 Load Estimation and Distribution</td>
<td>49</td>
</tr>
<tr>
<td>3.6 Load Combinations</td>
<td>51</td>
</tr>
<tr>
<td>4 MATERIALS AND CORROSION</td>
<td>53</td>
</tr>
<tr>
<td>4.1 General</td>
<td>53</td>
</tr>
<tr>
<td>4.2 Structural Steel</td>
<td>54</td>
</tr>
<tr>
<td>4.2.1 Structural Member Classification</td>
<td>54</td>
</tr>
<tr>
<td>4.2.2 Steel Grades</td>
<td>54</td>
</tr>
<tr>
<td>4.2.3 Manufacturing</td>
<td>56</td>
</tr>
<tr>
<td>4.2.4 Chemical Requirements</td>
<td>56</td>
</tr>
<tr>
<td>4.2.5 Carbon Equivalent (CE)</td>
<td>56</td>
</tr>
<tr>
<td>4.2.6 Mechanical Requirements</td>
<td>57</td>
</tr>
<tr>
<td>4.2.7 Notch Toughness Requirements</td>
<td>57</td>
</tr>
<tr>
<td>4.2.8 Supplementary Requirements</td>
<td>57</td>
</tr>
<tr>
<td>4.3 Corrosion</td>
<td>57</td>
</tr>
<tr>
<td>4.3.1 Seawater Corrosion</td>
<td>57</td>
</tr>
<tr>
<td>4.3.2 Factors influencing corrosion</td>
<td>58</td>
</tr>
<tr>
<td>4.3.3 types of corrosion</td>
<td>60</td>
</tr>
<tr>
<td>4.3.4 Rates of corrosion</td>
<td>61</td>
</tr>
</tbody>
</table>
## 5 SIMULATION

5.1 General ........................................................................................................... 67
5.2 Structure Geometry selection ........................................................................... 68
5.3 Geometry Simulation ......................................................................................... 68
  5.3.1 Axis System ............................................................................................... 69
  5.3.2 Joints or Nodes .......................................................................................... 69
  5.3.3 Members and properties ............................................................................. 70
  5.3.4 Offsets and Eccentricities ......................................................................... 71
  5.3.5 Wish-bone Connections ............................................................................. 71
  5.3.6 Dummy Structure Models ........................................................................... 72
5.4 Foundation Simulation ....................................................................................... 75
  5.4.1 Pile Modelling .......................................................................................... 75
  5.4.2 Soil Simulation ........................................................................................... 77
  5.4.3 Pile Group Effect ...................................................................................... 77
5.5 Load Simulation ................................................................................................. 80
  5.5.1 Dead Loads ............................................................................................... 80
  5.5.2 Equipment Loads ..................................................................................... 80
  5.5.3 Fluid Loads ............................................................................................... 80
  5.5.4 Drilling Loads ........................................................................................... 81
  5.5.5 Live Loads ................................................................................................. 81
  5.5.6 Wind, Wave and Current ......................................................................... 81
6 In-place Analyses

6.1 Principle .................................................. 85
6.2 Geometry Simulation .................................... 86
   6.2.1 Simplified Topside Model ......................... 87
   6.2.2 Load Simulation ...................................... 87
   6.2.3 Topsides Load COG Shift Envelope ................. 88
   6.2.4 Minimum and Maximum Water Depth ................ 88
   6.2.5 Maximum Wave Loads .............................. 89
   6.2.6 Allowable Stresses ............................... 89

7 Dynamic Analysis ........................................... 91

7.1 Principle .................................................. 91
7.2 Geometry Simulation .................................... 91
7.3 Load Simulation ........................................... 92
   7.3.1 Computer Generated Mass ......................... 92
   7.3.2 Direct Input Mass ................................... 92
7.4 Foundation Simulation .................................... 93
7.5 Solution .................................................. 93
   7.5.1 Dynamic Amplification factor ...................... 94

8 Fatigue Analysis ............................................. 95

8.1 General .................................................. 95
8.2 Deterministic Fatigue Analysis ......................... 95
8.3 Wave Scatter Data ....................................... 96
8.4 Spectral Fatigue Analysis .............................. 98
## 8.4.1 Principle

8.4.2 Wave Spectra

8.4.3 Wave Theory Selection

8.4.4 Wave Steepness

8.4.5 Transfer Functions

8.4.6 Selection of Frequencies

8.4.7 Stress Amplitudes

8.4.8 Fatigue Life Calculation

8.4.9 S-N Curves

8.4.10 Stress Concentration Factors

8.4.11 Foundation Linearisation

## 9 Ship Impact Analysis

9.1 Impact Vessels

9.2 Principle

9.3 Method of Computing Impact Energy Dissipation

9.4 Energy dissipation by member

9.5 Energy Dissipation by Overall jacket deflection

9.6 Structural Strength During Impact

9.7 Push Over Analysis
Chapter 1

INTRODUCTION

1.1 General

One of the greatest discovery of 20th century was oil and it has so many applications that it cannot be separated from mankind. The oil exploration has started as early as 1900 and the oil exploration initially was concentrated on on land. As the need for oil expands in an explosive rate, need for find new discoveries was eminent. During the middle of 20th century, oil discovery started in near shore and medium range of water depth.

The need for qualified offshore structural personnel are rapidly increasing as the oil industry moves into deeper water in the search for additional supplies of oil and gas, new technology is emerging at a rapid pace for the development of new concepts for offshore platforms.

This book gives brief introduction to offshore engineering with basic concepts of various types of offshore structures and provide insight into various design issues and requirements, fabrication and installation techniques.

Chapter 1 gives introduction in to types of offshore platforms based on water depth requirements, geometry and installation concepts.

Chapter 2 gives some introduction to design methodology, and various design stages in a offshore development project.

Chapter 3 gives basic loads applied on offshore structures and techniques of calculations of such loading.

Chapter 4 gives introduction to material requirement for offshore structures including corrosion.
Chapter 5 gives introduction to materials used for offshore structures, and corrosion and cathodic protection.

Chapter 6 describes inplace analysis methodology, load combinations and various principles involved in the design.

Chapter 7 describes methodology to carry out the dynamic analysis of an offshore platform and its application to fatigue and seismic analyses.

Chapter 8 gives method of fatigue analysis such as deterministic and spectral methods including, selection of S-N curves, SCF equations etc.

Chapter 9 give some introduction in to ship impact with offshore platforms and method to carry out push over analysis.

### 1.2 Types of Offshore Structures

The offshore structures built in the ocean to explore oil and gas are located in depths from very shallow water to the deep ocean. Depending on the water depth and environmental conditions, the structural arrangement and need for new ideas required. Based on geometry and behaviour, the offshore structures for oil and gas development has been divided into following categories.

1. Fixed Platforms
   - Steel template Structures
   - Concrete Gravity Structures

2. Compliant tower
   - Compliant Tower
   - Guyed Tower
   - Articulated Tower
   - Tension Leg Platform

3. Floating Structures
   - Floating Production System
   - Floating Production, Storage and Offloading System
1.3 Fixed Platforms

The fixed type of platform shall exhibit a low natural period and deflection against environmental loads.

1.3.1 Steel template Structures

The steel template type structure consists of a tall vertical section made of tubular steel members supported by piles driven into the sea bed with a deck placed on top, providing space for crew quarters, a drilling rig, and production facilities. The fixed platform is economically feasible for installation in water depths up to 500m.

These template type structures will be fixed to seabed by means of tubular piles either driven through legs of the jacket (main piles) or through skirt sleeves attached to the bottom of the jacket.

The principle behind the fixed platform design is to minimize the natural period of the structure below 4 seconds to avoid resonant behaviour with the waves (period in the order of 4 to 25 seconds). The structural and foundation configuration shall be selected to achieve this concept.

1.3.2 Concrete Gravity Platforms

Concrete gravity platforms are mostly used in the areas where feasibility of pile installation is remote. These platforms are very common in areas with strong seabed geological conditions either with rock outcrop or sandy formation.

Some part of north sea oil fields and Australian coast, these kind of platforms are located. The concrete gravity platform by its name derive its horizontal stability against environmental forces by means of its weight. These structures are basically concrete shells assembled in circular array with stem columns projecting to above water to support the deck and facilities.

Concrete gravity platforms have been constructed in water depths as much as 350m.
Figure 1.1: Different types of Offshore Structures
1.3. Fixed Platforms

Chapter 1. INTRODUCTION

Figure 1.2: Fixed Template type platform
Figure 1.3: Concrete Gravity Platform
1.4 Compliant Structures

In addition to the developing technologies for exploration and production of oil and natural gas, new concepts in deepwater systems and facilities have emerged to make ultra-deepwater projects a reality. With wells being drilled in water depths of 3000m, the traditional fixed offshore platform is being replaced by state-of-the-art deepwater production facilities. Compliant Towers, Tension Leg Platforms, Spars, Subsea Systems, Floating Production Systems, and Floating Production, Storage and Offloading Systems are now being used in water depths exceeding 500m. All of these systems are proven technology, and in use in offshore production worldwide.

1.4.1 Compliant Tower

Compliant Tower (CT) consists of a narrow, flexible tower and a piled foundation that can support a conventional deck for drilling and production operations. Unlike the fixed platform, the compliant tower withstands large lateral forces by sustaining significant lateral deflections, and is usually used in water depths between 300m and 600m.
1.4.2 Guyed Tower

Guyed tower is an extension of compliant tower with guy wires tied to the seabed by means of anchors or piles. This guy ropes minimises the lateral displacement of the platform topsides. This further changes the dynamic characteristics of the system.

1.4.3 Tension Leg Platforms

A Tension-leg platform is a vertically moored floating structure normally used for the offshore production of oil or gas, and is particularly suited for water depths around 1000m to 1200 metres (about 4000 ft). The platform is permanently moored by means of tethers or tendons grouped at each of the structure’s corners. A group of tethers is called a tension leg. A feature of the design of the tethers is that they have relatively high axial stiffness (low elasticity), such that virtually all vertical motion of the platform is eliminated. This allows the platform to have the production wellheads on deck (connected directly to the subsea wells by rigid risers), instead of on the seafloor. This makes for a cheaper well completion and gives better control over the production from the oil or gas reservoir.

Tension Leg Platform (TLP) consists of a floating structure held in place by vertical, ten-
sioned tendons connected to the sea floor by pile-secured templates. Tensioned tendons provide for the use of a TLP in a broad water depth range with limited vertical motion. The larger TLP’s have been successfully deployed in water depths approaching 1250m.

Mini-Tension Leg Platform (Mini-TLP) is a floating mini-tension leg platform of relatively low cost developed for production of smaller deepwater reserves which would be uneconomic to produce using more conventional deepwater production systems. It can also be used as a utility, satellite, or early production platform for larger deepwater discoveries. The world’s first Mini-TLP was installed in the Gulf of Mexico in 1998.

SPAR Platform (SPAR) consists of a large diameter single vertical cylinder supporting a deck. It has a typical fixed platform topside (surface deck with drilling and production equipment), three types of risers (production, drilling, and export), and a hull which is moored using a taut catenary system of six to twenty lines anchored into the seafloor. SPAR’s are presently used in water depths up to 1000m, although existing technology can extend its use to water depths as great as 2500m.
1.4.4 Articulated Tower

Articulated tower is an extension of tension leg platform. The tension cables are replaced by one single buoyant shell with sufficient buoyancy and required restoring moment against lateral loads.

The main part of the configuration is the universal joint which connects the shell with the foundation system. The foundation system usually consists of gravity based concrete block or some times with driven piles.

The articulated tower concept is well suited for intermediate water depths ranging from 150m to 500m.

Figure 1.7: Articulated Tower Platforms
1.5 Floating Structures

1.5.1 Floating Production System

Floating Production System (FPS) consists of a semi-submersible unit which is equipped with drilling and production equipment. It is anchored in place with wire rope and chain, or can be dynamically positioned using rotating thrusters. Production from subsea wells is transported to the surface deck through production risers designed to accommodate platform motion. The FPS can be used in a range of water depths from 600m to 2500m feet.

1.5.2 Floating Production, Storage and offloading System

Floating Production, Storage and Offloading System (FPSO) consists of a large tanker type vessel moored to the seafloor. An FPSO is designed to process and stow production from nearby subsea wells and to periodically offload the stored oil to a smaller shuttle tanker. The shuttle tanker then transports the oil to an onshore facility for further processing. An FPSO may be suited for marginally economic fields located in remote deepwater areas where a pipeline infrastructure does not exist. Currently, there are no FPSO’s approved for use in the Gulf of Mexico. However, there are over 70 of these systems being used elsewhere in the world.

1.6 Subsea System

Subsea System (SS) ranges from single subsea wells producing to a nearby platform, FPS, or TLP to multiple wells producing through a manifold and pipeline system to a distant production facility. These systems are presently used in water depths greater than 1500m.

1.7 Fixed Platform Concepts

For the last few decades, the fixed platform concept has been utilized extensively over 300m depth with various configurations.
1.7. Fixed Platform Concepts

Chapter 1. INTRODUCTION

1.7.1 Functional Classification

The offshore platforms for oil and gas exploration purpose can be classified based on functionality and purpose of installation.

- **Wellhead platform** - primarily meant for drilling and supporting wellhead equipment. It supports very few equipment such as wellhead control panel and piping. Occasionally it also supports helicopter landing structure for emergency evacuation.

- **Process Platform** - primary meant for production facilities (oil or gas) and it may support in addition to equipment for production, such as power generation, utilities and living quarters.

- **Riser Platform** - This is another kind of structure specially built to support all the incoming and outgoing risers on a planned complex. This will also be connected to the main platform by bridge.

- **Living Quarters Platform** - Some times due to safety requirements, the living quarters will be supported on a separate structure away from the wellhead and process platforms. This types of platform will be located atleas 50m away from the neighboring process platforms and will be connected by a bridge.
1.7. Fixed Platform Concepts

1.7.2 Geometrical Classification

The structural configuration of fixed template type structures vary extensively from location to location depending on the requirement and environmental conditions such as water depth, wave and current loads etc. Based on geometry, jackets can be classified into following categories.

- **Tripod** - basically to support minimum facility such as few wellhead and riser or to support a bridge between two major platforms or to support a flare boom

- **4 Legged** - typically for wellhead platforms

- **6 or 8 Legged** - mainly for process complex
1.7.3 Foundation Concepts

The offshore platforms shall be fixed to the seabed by means of piles either driven through the main legs of the jacket or through skirt sleeves attached to the jacket legs or the combinations of both main and skirt piles. This kind of arrangement is shown in the following pictures.
Chapter 2

DESIGN METHODOLOGY

2.1 General

The design of offshore structure is not an single step design process. The structural configuration, arrangement, member sizes and its specification requirements can be arrived after few design cycles. In order to achieve a optimum design suitable for the installation method proposed and satisfy the final operating requirements, a design procedure suitable for the project shall be developed.

In an offshore project, the design of structural elements cannot be initiated unless the basic understanding of the needs are identified. The basic needs are

- What is the type of platform? Oil or Gas, Process or Wellhead or Quarters etc
- What is floor area of topsides required?
- Expected maximum weight of facilities?
- What is basic water depth and environmental parameters such as wave and current?
- Where is it located?. Earthquake prone ?.
- What is type of installation? Lift installed or Launch installed?
- Any CAPEX constraints?

The answer to the above questions will give some indication of type of jacket and topsides required.
2.2 Design Stages

The various design stages in an offshore project is listed below.

- Front End Engineering Design (FEED) or Concept Selection
- Basic Design
- Detailed Design

2.2.1 FEED

The first step in initiating an offshore project is a FEED or concept selection. This stage of project will involve following steps in all disciplines such as Process, Mechanical, Electrical and Instrumentation in addition to Structural Engineering.

- Collection Process Data and identifying process needs and equipment
- Preliminary equipment sizing and area requirements
- Weight estimation based on past projects
- Identification of Structural configurations
- Preliminary estimation of structural weight
- Identification of installation methods
- Estimation of CAPEX within ±40%.

The above activities will define the project to a basic understanding and will provide enough insight into carrying out further engineering activities.

2.2.2 Basic Design

At this stage of the project, the data collected during the FEED stage will be further verified to make sure the authenticity and reliability of such data for further use. A detailed weight estimates of all items involved in the project will be carried out. The process and mechanical requirements will be further defined and identified. A Design Basis (DB) will be developed for the proposed facility containing following information.
2.2. Design Stages

- **Process** information containing type of well fluid (oil or gas) and its characteristics, safety requirements and kind of process technology to be adopted.

- **Mechanical** requirement such as type of facility and basic equipment required for the process, and material handling and safety

- **Electrical** requirement such power generation equipment, lighting and switch gears etc

- **Instrumentation** requirement such as basic control system, feedback requirement etc.

- **Piping** information such as pressures, pipe sizes required etc.

- **Meta-Ocean** information such as water depth, wave, current, wind and tidal information at the site.

- **Structural** requirement such as materials proposed or available for use in the country, design method to be adopted, codes and specifications to be used etc.

- **Installation** information such as type of barge, lifting crane, loadout-method, piling hammer etc.

At the basic design stage, the deck area required, number deck levels, etc will be defined. This will lead to identification of number of legs required to support the deck. Normally the spacing between deck legs for a typical platform can vary from 10m to 20m beyond which it may become uneconomical to design a braced deck truss structure.

Basic weight estimates for various disciplines such as structural, mechanical, electrical, instrumentation and piping will be carried out. Based on the weight of total deck, it may then be decided to fabricate the deck in one piece or in various modules. This kind decision can only be taken together with the viable installation options such ”Availability of Heavy Lift vessels in the region” or use of float-over technique. In case such methods are not possible, then the total topsides shall be divided in to various functional modules such as compression module, process module module, utility module, quarters module, etc. These modules are self contained units with structure, piping, equipment etc fabricated and transported to the site. These modules are then installed on top of the ”module Support Frame”, which transfers the loads to the jacket. Some times this module support frame may not needed, if the modules are organised properly over the legs. This kind of basic ideas shall be made at the basic design stage.
2.2. Design Stages

Figure 2.1: Deck Installation Concepts - Integrated Deck (Single Lift)

Figure 2.2: Deck Installation Concepts - Integrated Deck (Float Over)
2.2. Design Stages

Chapter 2. DESIGN METHODOLOGY

Figure 2.3: Deck Installation Concepts - Modular Deck (MSF)

Figure 2.4: Deck Installation Concepts - Modular Deck (Modules)
2.2.3 Detailed Design

Detailed design of offshore platform will be initiated once the basic design confirms the economic viability and technical feasibility.

In the Detailed design or engineering of an offshore platform following items of the jacket and deck will be developed in detail.

<table>
<thead>
<tr>
<th>Deck</th>
<th>Jacket</th>
</tr>
</thead>
<tbody>
<tr>
<td>Framing Plans</td>
<td>Framing Plans</td>
</tr>
<tr>
<td>Framing Elevations</td>
<td>Framing Elevations</td>
</tr>
<tr>
<td>Joint Details</td>
<td>Joint Details</td>
</tr>
<tr>
<td>Plate Girder Details</td>
<td>Pile make-up</td>
</tr>
<tr>
<td>Connection Details</td>
<td>Launch Truss Details</td>
</tr>
<tr>
<td>Welding Details</td>
<td>Welding Details</td>
</tr>
<tr>
<td>Deck plating &amp; grating</td>
<td>Launch Cradle details</td>
</tr>
<tr>
<td>Stairways and Walkways</td>
<td>Walkways</td>
</tr>
<tr>
<td>Lifting Padeyes</td>
<td>Lifting Padeyes</td>
</tr>
<tr>
<td>Transportation tie-down</td>
<td>Transportation Tie-down</td>
</tr>
<tr>
<td>Monorails Details</td>
<td>Caissons and supports</td>
</tr>
<tr>
<td>Equipment Support Details</td>
<td>Conductor guides</td>
</tr>
<tr>
<td>Flare Boom</td>
<td>Barge Bumpers</td>
</tr>
<tr>
<td>Bridge</td>
<td>Boat Landing</td>
</tr>
<tr>
<td>Crane Pedestal</td>
<td>Closure Plate details</td>
</tr>
<tr>
<td></td>
<td>Riser Clamps</td>
</tr>
</tbody>
</table>

Table 2.1: Detailed Design Items

All necessary analysis required to complete the design above items shall be carried out.
### 2.2. Design Stages

#### Chapter 2. DESIGN METHODOLOGY

<table>
<thead>
<tr>
<th>Deck</th>
<th>Jacket</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inplace</td>
<td>Inplace</td>
</tr>
<tr>
<td>Loadout</td>
<td>Loadout</td>
</tr>
<tr>
<td>Sea Transportation</td>
<td>Sea Transportation</td>
</tr>
<tr>
<td>Lifting or Floatover</td>
<td>Lifting or Launch</td>
</tr>
<tr>
<td>Miscellaneous Calculations</td>
<td>Upending</td>
</tr>
<tr>
<td></td>
<td>Dynamic</td>
</tr>
<tr>
<td></td>
<td>Fatigue (Deterministic or Spectral)</td>
</tr>
<tr>
<td></td>
<td>On bottom Stability</td>
</tr>
<tr>
<td></td>
<td>Pile Driveability</td>
</tr>
<tr>
<td></td>
<td>Cathodic Protection</td>
</tr>
<tr>
<td></td>
<td>Ship Impact</td>
</tr>
</tbody>
</table>

Table 2.2: Detailed Design Analysis
2.2. Design Stages

Chapter 2. DESIGN METHODOLOGY
Chapter 3

LOADS

3.1 General

3.2 Types of Loads

Loads on offshore structures are gravity loads and environmental loads. Gravity loads are arising from dead weight of structure and facilities either permanent or temporary. Seismic loads are arising from gravity loads and is a derived type.

Environmental loads play a major role governing the design of offshore structures. Before starting the design of any structure, prediction of environmental loads accurately is important. Various environmental loads acting on the offshore platform is listed below.

- Gravity Loads
  - Structural Dead Loads
  - Facility Dead Loads
  - Fluid Loads
  - Live Loads
  - Drilling Loads

- Environmental Loads
  - Wind Loads
  - Wave Loads
  - Current Loads
3.3.1 Dead Loads

Dead loads includes the all the fixed items in the platform deck, jacket, bridge and flare structures. It includes all primary steel structural members, secondary structural items such as boat landing, padeyes, stiffeners, handrails, deck plating, small access platforms etc.

The primary structural steel members will be calculated based on the structural information in the model automatically when a computer program is used to analyse the structure. But the weight of secondary structural steel items shall be calculated applied to the structural model at appropriate locations.

3.3.2 Facility Dead Loads

The structure built either for drilling or wellhead type platform or for process type platform supports various equipment and facilities. These are fixed type items and not structural components. they do not have any stiffness to offer in the global integrity of the structure and shall not be modelled. The weight of such items shall be calculated and applied at the appropriate locations according the plan of the structure. These items include

- Mechanical equipment
- Electrical equipment
- Piping connecting each equipment
- Electrical Cable trays
- Instrumentation items
3.3.3 Fluid Loads

The fluid loads are weight of fluid on the platform during operation. This may include all the fluid in the equipment and piping. The weight of these items shall be calculated accurately and applied to the correct locations.

3.3.4 Live Loads

Live loads are defined as movable loads and will be temporary in nature. Live loads will only be applied on areas designated for the purpose of storage either temporary or long term. Further, the areas designed for laydown during boat transfer of materials from boat shall also be considered as live loads.

Other live load include open areas such as walkways, access platforms, galley areas in the living quarters, helicopter loads in the helipad, etc. These loads shall be applied in accordance with the requirement from the operator of the platform. This load vary in nature from owner to owner but a general guideline on the magnitude of the loads is given below.

<table>
<thead>
<tr>
<th>Sl. No</th>
<th>Location</th>
<th>Load (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Storage / laydown</td>
<td>10</td>
</tr>
<tr>
<td>2</td>
<td>Walkway</td>
<td>5</td>
</tr>
<tr>
<td>3</td>
<td>Access Platform</td>
<td>5</td>
</tr>
<tr>
<td>4</td>
<td>Galley</td>
<td>10</td>
</tr>
</tbody>
</table>

Table 3.1: Live Loads

3.3.5 Drilling Loads

Drilling loads are due to drill rigs placed on top of the platform for drilling purposes. These are large equipment assembled together and placed on top. Normally, drilling rigs are as heavy as 500 Tonnes to 1000 Tonnes. These will deliver reaction forces on the deck and the stiffness of the drilling rigs are not considered in the structural analysis. Hence the weight of the structure shall be applied as load on the structure. Further, during drilling, additional loads will be developed due to drill string and pulling operations. these loads also shall be considered in the analysis.
3.4 Environmental Loads

3.4.1 Wind Loads

The wind speed at 10m above LAT (Lowest Astronomical Tide) is normally provided \( (V_o) \). This wind speed shall be extrapolated to the height above for the calculation of wind speed. The extrapolation shall be calculated as below.

\[
V = V_o \left( \frac{y}{10} \right)^{\frac{1}{8}}
\]  

(3.1)

where \( y \) is the elevation of point in consideration in m above LAT and \( V \) is the velocity at that point. Wind loads shall be calculated as per API RP2A guidelines.

Sustained wind speeds (10min mean) shall be used to compute global platform wind loads and gust wind (3 second) shall be used to compute the wind loads to design individual members.

The wind pressure can be calculated as

\[
f_w = \frac{\rho g}{2} V^2
\]  

(3.2)

where \( F \) is the wind pressure per unit area, \( \rho \) (0.01255 kN/m\(^3\)) is the density of air, \( g \) is the gravitational acceleration (9.81 m/sec\(^2\)) and \( V \) is the wind speed in m/sec. The above equation can be simplified by substituting the values and can be expressed as

\[
f_w = 0.6V^2 \quad kN/m^2
\]  

(3.3)

The total wind load on the platform can be calculated using the wind blockage area and the pressure calculated as above. The shape coefficient \( (C_s) \) shall be selected as per AP RP2A guidelines. But for the calculation of global wind load (for jacket and deck global analysis) shape coefficient can be 1.0.

The total force on the platform can be calculated as

\[
F_x = f_w A_x C_s
\]  

(3.4)

\[
F_y = f_w A_y C_s
\]  

(3.5)
The exposed areas \((A_x\) and \(A_y\)) shall be calculated as \(\text{length} \times \text{height}\) or \(\text{width} \times \text{height}\) depending on the axis system followed.

Wind load on oblique directions can be calculated using following relationship.

\[
F_\theta = F_x\cos(\theta) + F_y\sin(\theta)
\]  

(3.6)

In practical design, it is often only \(F_x\) and \(F_y\) will be calculated and applied in the structural analysis as basic loads and the wind load effect due to non-orthogonal directions are simulated using factors in terms of \(F_x\) and \(F_y\) in the load combinations. The factors can be calculated as

The projected areas can be calculated as \(A_1 = A_x\cos(\theta)\) and \(A_2 = A_y\sin(\theta)\)

\[
F_\theta = f_w (A_1 + A_2))
\]  

(3.7)

\[
F_\theta = f_w (A_x\cos(\theta) + A_y\sin(\theta))
\]  

(3.8)

\[
F_{\theta x} = f_w (A_x\cos(\theta) + A_y\sin(\theta)) \cos(\theta)
\]  

(3.9)

\[
F_{\theta y} = f_w (A_x\cos(\theta) + A_y\sin(\theta)) \sin(\theta)
\]  

(3.10)
where $F_{\theta x}$ and $F_{\theta y}$ are the components of $F_\theta$ in x and y directions respectively. Ratio between $F_{\theta x}$ and $F_x$ can be expressed as

$$\frac{F_{\theta x}}{F_x} = \frac{f_w \left(A_x \cos(\theta) + A_y \sin(\theta)\right) \cos(\theta)}{f_w A_x}$$

(3.11)

$$\frac{F_{\theta x}}{F_x} = \cos^2(\theta) + \left(\frac{A_y}{A_x}\right) \sin(\theta) \cos(\theta)$$

(3.12)

Similarly, ratio between $F_{\theta y}$ and $F_y$ can be expressed as

$$\frac{F_{\theta y}}{F_y} = \frac{f_w \left(A_x \cos(\theta) + A_y \sin(\theta)\right) \sin(\theta)}{f_w A_y}$$

(3.13)

$$\frac{F_{\theta y}}{F_y} = \sin^2(\theta) + \left(\frac{A_x}{A_y}\right) \sin(\theta) \cos(\theta)$$

(3.14)

### 3.4.2 Wave and Current Loads

**Methodology**

In applying design waves load onto the offshore structures, there are two ways of applying it.

- **Design Wave method**
- **Spectral Method**

In design wave method, a discrete set of design waves (maximum) and associated periods will be selected to generate loads on the structure. These loads will be used to compute the response of the structure.

In the spectral method, a energy spectrum of the sea-state for the location will be taken and a transfer function for the response will be generated. These transfer function will be used to compute the stresses in the structural members.
3.4. Environmental Loads

Design Wave method

The forces exerted by waves are most dominant in governing the jacket structures design especially the foundation piles. The wave loads exerted on the jacket is applied laterally on all members and it generates overturning moment on the structure.

Period of wind generated waves in the open sea can be in the order of 2 to 20 seconds. Theses waves are called gravity waves and contain most part of wave energy.

Maximum wave shall be used for the design of offshore structures. The relationship between the significant wave height \( H_s \) and the maximum wave height \( H_{\text{max}} \) is

\[
H_{\text{max}} = 1.86 H_s
\]  

The above equation correspond to a computation based on 1000 waves in a record.

The design wave height for various regions is tabulated below.

<table>
<thead>
<tr>
<th>Region</th>
<th>1 year</th>
<th>100 year</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bay of Bengal</td>
<td>8</td>
<td>18</td>
</tr>
<tr>
<td>Gulf of Mexico</td>
<td>12</td>
<td>24</td>
</tr>
<tr>
<td>South China Sea</td>
<td>11</td>
<td>24</td>
</tr>
<tr>
<td>Arabian Sea</td>
<td>8</td>
<td>18</td>
</tr>
<tr>
<td>Gulf of Thailand</td>
<td>6</td>
<td>12</td>
</tr>
<tr>
<td>Persian Gulf</td>
<td>5</td>
<td>12</td>
</tr>
<tr>
<td>North Sea</td>
<td>14</td>
<td>22</td>
</tr>
</tbody>
</table>

Table 3.2: Maximum design waves in various regions

API RP2A requires both 1 year and 100 year recurrence wave shall be used for the design of jacket and piles. Appropriate combination of loads with these waves shall be used in the design. A one-third increase in permissible stress is allowed for 100 year storm conditions.

Spectral Method

Instead of simulating the design wave environment by discrete maximum wave, a design sea-state described by energy spectrum of for the given site will be used in the load simulation. A directional spectrum can also be used to simulate the changes design wave sea-state.
Various sea-state energy spectra are available for use, some of which are listed below.

- PM Spectra
- Jonswap Spectra
- ISSC Spectra

### 3.4.3 Current Profile

Oceans currents induce drag loading on offshore structures. These currents together with the action of waves generate dynamic loads.

Ocean currents are classified into few types based on their nature viz, tidal current, wind driven current and current generated due to ocean circulation. Wind driven currents are small in nature and it varies linearly with depth where as tidal currents vary nonlinearly with depth. Similarly, the currents generated due to ocean circulation will vary nonlinear with depth and can be as much as 5 m /sec.

The current variation with depth is shown in Figures and can be expressed as below.

\[
V_T = V_{oT} \left( \frac{y}{h} \right)^{\frac{1}{7}} 
\]  

(3.16)

where \( V_T \) is the tidal current at any height from sea bed, \( V_{oT} \) is the tidal current at the surface, \( y \) is the distance measure in m from seabed nd \( h \) is the water depth.

\[
V_W = V_{oW} \frac{y}{h} 
\]  

(3.17)
3.4. Environmental Loads

where $V_W$ is the wind driven current at any height from sea bed, $V_{oW}$ is the wind driven current at the surface, $y$ is the distance measure in m from seabed and $h$ is the water depth.

### 3.4.4 Marine Growth

Marine growth is an important part in increasing the loads on offshore structures. The growth of marine algae increases the diameter and roughness of members which in turn cause the wave or current loading to increase. Detailed discussion on the member roughness and its relationship with hydrodynamic coefficients can be found in API RP2A.

The thickness of marine growth generally decrease with depth from the mean sea level and it is maximum in the splash zone. The thickness of marine growth in the splash zone can be as much as 20cm and will reduce below to 5cm. In deeper zones, the thickness may be negligible.

Splash Zone is a region where the water levels fluctuate between low to high. The actual elevation of the bottom and top of these vary from location to location due to different tidal conditions. In general terms, the splash zone will vary from -3m to +5m.

In structural analysis, the increased diameter of the member ($D = d + t_m$) shall be included so that the wave and current loads can be calculated correctly. $D$ and $d$ are the diameter of increased member and original member respectively and $t_m$ is the thickness of marine growth.

The roughness of the marine growth is an important parameter in determining the drag and inertia coefficients. reference shall be made relevant API RP2A clauses for more details.

### 3.4.5 Morison Equation

Wave and current loading can be calculated by Morison equation.

Morison equation can be written as:

$$F_T = \frac{1}{2} C_D \rho_w D V |V| + \frac{\pi D^2}{4} C_M \rho_w a$$

(3.18)

where $F_T$ is the total force, $\rho_w$ is the density of water, $C_D$ and $C_M$ are the drag and inertia coefficients respectively, $D$ is the diameter of the member including marine growth, $V$ is the velocity and $a$ is the acceleration.
The first term in the equation is drag component \( F_D \) and the second term is the inertia component \( F_I \). This can be expressed as

\[
F_T = F_D + F_I
\] (3.19)

Most of the time, current exist in the same direction of the wave propagation and hence the current shall be taken into consideration in the load calculation. However, algebraic sum of wave and current loads is different from calculation of load by adding the horizontal water particle velocity with the current velocity and computing the loads. This is because of nonlinear term in the drag equation.

Current velocity shall be added vectorially with the water particle velocity before computation of drag force, i.e. \( V = V_w + V_c \) where \( V \) is the total velocity, \( V_w \) is the velocity due to waves and \( V_c \) is the velocity of current. This is required since there is a square term in the drag force equation.

### 3.4.6 Wave-Current Interaction

The wave current interaction is an important phenomenon since the waves propagate on the current. Both current modifies the wave and wave modifies the current exist. But the former takes most priority in the calculations of wave loads. This interaction modifies the wave parameters and modifies the wave field. Depending on the direction of current in respect of wave direction, it either stretches the wave longer or shortens it.

If the current travels in the same direction as the wave, then the wave period becomes longer and it is called apparent wave period \( T_{app} \). Recommendation of API RP2A shall be used to estimate the apparent wave period. Following simultaneous equations shall be solved to obtain the apparent wave period.

\[
\frac{L}{T} = \frac{L}{T_{app}} + V_I
\] (3.20)

\[
T_{app} = \frac{2\pi L}{g \tanh kh}
\] (3.21)

\[
V_I = \frac{4\pi / L}{\sinh kh} \int_{-h}^{0} U_c(z) \cosh 2k(z + h)dz
\] (3.22)

Refer to API clause 2.3.1.b (1) for more details on how to compute this apparent wave period.
3.4.7 Selection of Wave Theory

The computation of wave kinematics such as velocity and acceleration involves the equations from wave theory. There are various kinds of solutions available depending on the accuracy required, and parameters involved in the computation. The various wave theories are listed below.

- Linear / Airy Wave Theory
- Stokes Wave Theory (up to 5th order approximations)
- Stream Function Wave Theory (up to 22nd order approximations)
- Cnoidal Wave Theory

Depending on the location such as deep water or shallow water and associated wave parameters, a suitable wave theory shall be selected for use. API RP 2A recommends to use a chart for such selection based on $d/gT^2$ and $H/gT^2$ as the X and Y axis. Refer to Figure.

The wave theories discussed above are for non-breaking waves. For $H/h > 0.78$, these theories are not applicable as the waves tend to break. In such situation, empirical equations shall be used to calculate the breaking wave forces on the structures.

3.4.8 Wave Load on a Member

The various steps involved in calculating loads on a member can be shown graphically.

Morison equation is a general form and can not be applied to all members in the offshore structure. It was developed specifically for a surface piercing cylinder like pile of a structure. But in reality, the members of the offshore structure may be horizontal or inclined in space and can not used without modification.

Water Wave Kinematics

Airy wave theory is considered in the calculation of wave kinematics. Consider a progressive wave with water surface elevation depicted by cosine curve,

$$\zeta = \frac{H}{2} \cos(kx - \omega t)$$

(3.23)
and the corresponding velocity potential is given by

\[ \phi = -\frac{H}{2} \omega \frac{\cosh k(h + z)}{\sinh kh} \sin(kx - \omega t) \] (3.24)

The horizontal and vertical velocity and acceleration of water particle can be calculated using the following equations.

\[ V_h = -\frac{\partial \phi}{\partial x} = \frac{H}{2} \omega \frac{\cosh k(h + z)}{\sinh kh} \cos(kx - \omega t) \] (3.25)
3.4. Environmental Loads

Figure 3.3: Procedure for Calculation Wave Plus Current Loads (Extract from API RP 2A)

\[ V_v = -\frac{\partial \phi}{\partial z} = \frac{H}{2} \omega \frac{\sinh k(h + z)}{\sinh kh} \sin(kx - \omega t) \]  

\[ a_h = \frac{\partial V_h}{\partial t} = \frac{H}{2} \omega^2 \frac{\cosh k(h + z)}{\sinh kh} \sin(kx - \omega t) \]  

\[ a_v = \frac{\partial V_v}{\partial t} = -\frac{H}{2} \omega^2 \frac{\sinh k(h + z)}{\sinh kh} \cos(kx - \omega t) \]  

where \( k \) is the wave number defined by \( 2\pi/L \), \( \omega \) is the wave circular frequency defined by \( 2\pi/T \), \( L \) is the wave length, and \( x \) is the distance of the point in consideration from origin.

**Maximum Load on a vertical member**

Consider a case of a surface piercing cylinder such as pile of a structure or a leg of a jacket, the combined drag and inertia force (total force) varies with time and will be maximum only at one occasion. In order find the maximum force, phase angle at which the maximum force occurs shall be found first.

Let us express the total force on the pile by substituting the velocity and acceleration components and integrating between the limits (from surface to seabed, i.e., 0 to -h),
3.4. Environmental Loads

Figure 3.4: Wave Loads on Jacket Structure

\[ F_T = \frac{1}{2} C_D \rho D \frac{\pi^2 H^2}{T^2} \frac{\cos \theta \cos \theta}{\sinh^2 kh} \left[ \frac{\sinh(2kh)}{4k} + \frac{h}{2} \right] \]

\[ -C_M \rho \frac{\pi D^2}{4} \frac{2\pi^2 H}{T^2} \frac{\sin \theta}{k} \]

The total force will be maximum when,

\[ \frac{\partial F_T}{\partial \theta} = 0 \]
3.4. Environmental Loads

Substituting the values of velocity and acceleration components into the drag and inertia force equation and differentiating with respect to $\theta$ and rearranging the terms, we get

$$
\theta_{\text{max}} = \cos^{-1} \left[ -\frac{\pi D}{H} \frac{C_M}{C_D} \frac{2 \sinh^2 k h}{(\sinh 2 k h + 2 k h)} \right]
$$

(3.31)

**Maximum Load on a horizontal member**

Consider a case of a horizontal cylinder such as brace of a jacket, the combined drag and inertia force (total force) varies with time and will be maximum only at one occasion. In order to find the maximum force, phase angle at which the maximum force occurs shall be found first.

Let us express the total force on the pile by substituting the velocity and acceleration,

$$
F_T = \frac{1}{2} C_D \rho D \frac{H^2 \omega^2}{4} \cos \theta | \cos \theta | \left[ \frac{\cosh k(z + h)}{\sinh k h} \right] \\
- C_M \rho \frac{\pi D^2}{4} \frac{H \omega^2}{2} \sin \theta \left[ \frac{\cosh k(z + h)}{\sinh k h} \right]
$$

(3.32)

The total force will be maximum when,

$$
\frac{\partial F_T}{\partial \theta} = 0
$$

(3.33)

Substituting the values of velocity and acceleration components into the drag and inertia force equation and differentiating with respect to $\theta$ and rearranging the terms, we get

$$
\theta_{\text{max}} = \sin^{-1} \left[ -\frac{\pi D}{2 H} \frac{C_M}{C_D} \frac{\sinh k h}{\cosh k (h + z)} \right]
$$

(3.34)

**Maximum Load on a inclined member**

The resultant force on an arbitrarily oriented circular cylinder in water waves can be calculated using vector analysis combined with Morison equation.
The resultant force on a cylinder in general has component normal to the cylinder axis $F_n$ and a component along the axis of the cylinder (a tangential component) $F_t$. Thus, the total force per unit length of the cylinder can be written as

$$\vec{F} = \vec{F}_n + \vec{F}_t \quad (3.35)$$

Each of these components can be expressed as functions of the fluid particle motions by using Morison’s equation. The force in normal direction can be expressed as

$$F_n = F_{Dn} + F_{In} \quad (3.36)$$

where $F_{Dn}$ and $F_{In}$ are the drag and inertia forces respectively. These forces can be expressed as

$$F_{Dn} = \frac{1}{2} C_D n D \rho \vec{V}_n |\vec{V}_n| \quad (3.37)$$

$$F_{In} = \frac{1}{4} \pi C_M n D^2 \rho \vec{a}_n \quad (3.38)$$

where

- $C_D = \text{Drag coefficient for flow normal to the cylinder}$
- $C_M = \text{Inertia coefficient for flow normal to the cylinder}$
- $D = \text{Diameter of cylinder}$
- $\rho = \text{Density of seawater}$
- $\vec{V}_n = \text{Velocity of fluid particle normal to the cylinder axis}$
- $\vec{a}_n = \text{Acceleration of fluid particle normal to the cylinder axis}$

In the tangential direction, only a skin friction drag term exists since inertial component along the member axis does not exist unless an axial inertia coefficient is specified. Hence the equation for tangential force can be written as

$$\vec{F}_t = \vec{F}_D \quad (3.39)$$

$$\vec{F}_D = \frac{1}{2} C_D n D \rho \vec{V}_t |\vec{V}_t| \quad (3.40)$$

where
3.4. Environmental Loads

$C_D^n = \text{Drag coefficient for flow tangential to the cylinder}$

$V_t = \text{Velocity of fluid particle tangential to the cylinder axis}$

These forces can be summed and expressed in terms of cylinder local axis as below.

\[
\vec{F}_x = \frac{1}{2} C_D^n D \rho \vec{V}_t |\vec{V}_t| \\
\vec{F}_y = \frac{1}{2} C_D^n D \rho \vec{V}_n |\vec{V}_n| + \frac{1}{4} \pi C_M D^2 \rho \vec{a}_y \\
\vec{F}_z = \frac{1}{2} C_D^n D \rho \vec{V}_n |\vec{V}_n| + \frac{1}{4} \pi C_M D^2 \rho \vec{a}_z
\]

The maximum forces can only be found numerically by calculating the forces for one wave cycle.

3.4.9 Maximum Global Loads

Maximum global loads on a platform can be calculated using two principles.

- Maximum Base Shear Method
- Maximum Overturning Moment Method

It is important that the wave loads on the structure be checked for both conditions. The maximum overturning moment method will give more pile loads than the other. Similarly, the maximum base shear method may govern the design of some jacket leg members near seabed due to high shear.

Maximum Base Shear

Maximum base shear or maximum total force on a structure has to be determined for the global analysis of structures. As the wave propagates across structure wave force on each member is different and all the locations will not be attaining the maximum forces. To find the maximum total force a structure, following steps need to be considered.

- Position the wave crest at the origin of the structure as shown in Figure.
• Divide one wave cycle into number of segments either in terms of $\theta$ or in terms of length.

• Compute the wave forces on all members at that instant of time using water wave velocities and accelerations computed.

• Sum up the forces in horizontal direction for all the members.

• Repeat the calculation in step 4 for all the points for one wave cycle.

• The maximum of all the total forces computed in step 5 is the maximum base shear or total force.

Maximum Overturning moment

Maximum overturning moment on a structure can be determined following the procedure for the maximum base shear case. In this case, the loads on the members shall be multiplied by the lever arm from mud-line. This shall be summed up and the procedure shall be repeated for all the steps in the wave.

3.4.10 Buoyancy Loads

The offshore structural members mostly made buoyant by air tight sealing of the welds to avoid water entry. This is purposely planned so that the overall structure will have adequate buoyancy during installation. Typical example is the jacket structure. This kind of structure requires at least a reserve buoyancy of 10% to 15%. The reserve buoyancy is defined as buoyancy in excess of its weight. To obtain this buoyancy, structural tubular members are carefully selected such that their buoyancy / weight ratio is greater than 1.0. This means that the member will float in water. On other hand, if the member is part of a structure supported at its two ends and forced to be submerged by weight of other members, this member will experience a upward force equal to the displaced volume of water. This is called buoyancy force. The buoyancy force can be calculated by two methods.

• Marine Method

• Rational Method

The marine method assumes that the member in consideration considered to have rigid body motion. This means that the weight of the member is calculated using submerged density
3.4. Environmental Loads

Chapter 3. LOADS

Figure 3.5: Buoyancy Calculation methods

of steel and applied to the member vertically down as an uniformly distributed load. This
buoyant weight $W_B$ of the member per unit length can be calculated as

$$W_B = \frac{1}{4} \pi (D^2 - (D - t)^2)(\rho_s - 1.025)$$ (3.44)

where $\rho_s$ is the density of steel

Unlike gravity which is a true body force acting on every particle of a body, buoyancy is the
resultant of fluid pressure acting on the surface of the body. These pressures can only act
normal to the surface.

The rational method takes in to account this pressure distribution on the structure, results
in a system of loads consisting of distributed loads along the members and concentrated
loads at the joints. The loads on the members are perpendicular to the member axis and in
the vertical plane containing the member. The magnitude of this distributed member load
can be expressed as

$$B_B = \frac{1}{4} \pi D^2 \rho_w \cos \alpha$$ (3.45)

where $\alpha$ is the angle between the member and its projection on a horizontal plane

The joint loads consists of forces acting in the directions of all of the members meeting at the
joint. These joint forces act in a direction that would compress the corresponding members
if they acted directly on them, and have magnitude of:

\[ P_B = \rho_w A h \]  

(3.46)

where

\[ A = \text{"displaced" area i.e. the material area for flooded members,} \]
\[ h = \text{water depth at the end of the member being considered} \]

3.4.11 Ice Loads

For structures located in polar regions and cold countries, ice loading shall be considered in the design. In this regions, the ice sheets of varying thicknesses can move from one location to other due to tide and under water current. These ices sheets when come closer and hit the offshore structures, large impact force is experienced by the structure.

Figure 3.6: Ice Loading on a structure
This kind of force cannot be calculated by means of analytical tools. However, based on experimental studies, an empirical equation is available and can be used to estimate the force $F_{\text{ice}}$.

$$F_{\text{ice}} = C f A \quad (3.47)$$

where

- $f_{\text{ice}} = \text{Crushing strength of ice vary between 1.5 MPa to 3.5 MPa}$
- $C_{\text{ice}} = \text{Ice force coefficient vary between 0.3 to 0.7}$
- $A = \text{Area struck by ice (Diameter of member x ice sheet thickness)}$

### 3.4.12 Mud Loads

Platforms located in the vicinity of the river mouth (shallow water platforms) may experience the mud flow loads. The river flow brings sediment transport and nearby mud towards the platform and may slide through the location.

Sometimes over a long period of time sediment settlement at the location of the platform may have sloping surface and mud slides can also generate mud loads.

These loads can be calculated using

$$F_{\text{mud}} = C_{\text{mud}} \tau D \quad (3.48)$$

where

- $C_{\text{mud}} = \text{Force Coefficient vary from 7 to 9}$
- $\tau = \text{Shear strength of soil 5 KPa to 10 kPa}$
- $D = \text{Diameter of pile or or member}$

### 3.5 Load Estimation and Distribution

The gravity loads on the topside shall be estimated with care, especially at the initial stage of the project. During initial stage, no reliable information may be available. Based on past experience weight of various items shall be assumed. Hence the possibility of weight overrun during the project can occur.

In order to reduce the risk of such situation and further consequences of re-design, a planned method of estimation and control shall be introduced called "Weight Control procedure".
In the weight control procedure, the weight elements of an topside shall be divided into the following categories.

- Primary Structure
- Secondary Structure
- Mechanical Equipment
- Piping & Bulks
- Electrical Equipment
- Instrumentation

In a typical wellhead platform, the distribution or break down of total weight or payload is shown in Table.
The weight estimation of various components will start as early as FEED stage. Structural analysis of concepts selected will require reasonable data for loading the structure with and subsequent analysis work. However, the equipment and other information may not be available at so early in the project. Hence weight estimation with suitable "contingency" shall be used in the design. Further, during the progress of the project, design requirement may change due to fundamental change in process techniques, equipment selection etc. This contingency is used to allow for such unforeseen growth in the topside load. This contingency is normally kept at higher during the initial stage and can be reduced once the information on each item is available.

Further, the variation in weight due to mill tolerances, fabrication tolerance, welding etc shall also be accounted for in the design. This allowances can be normally estimated and included. The total allowance for this shall be at least 5%.

There is another allowance normally kept aside during the design is the allowance for future expansion. This is normally decided by owner or developer of the field and identified future expansion. This is called **Future expansion allowance**.
3.6 Load Combinations

The load combinations used for adequacy checking of any offshore structure can be divided into following four categories.

- **Normal Operating Case** - Maximum gravity loads arising from normal operation of the platform with 1 year return period wave, current and wind. This case is used to check the structure against loads during the normal operation of the platform.

- **Hydro-test case** - This is a case where hydro-testing and commissioning of equipment and piping is carried out offshore. In this case, the equipment and piping will be fully filled with sea water and maximum gravity loads will be developed. It should be noted that not all equipment will be hydro-tested simultaneously. It will be done one by one. This case is used to check the local strength of the deck structure.

- **Extreme Storm Case** - Maximum gravity loads arising from extreme case with 100 year return period storm wave, current and wind. This case is used to check the structure due to loads during 100 year return period storm together with platform gravity loads.

  - Some platforms are designed to be unmanned and will be operation during the storm with remote control. Typically, a wellhead platform with only very mini-
mum personnel onboard will be fully functional except that the personnel will be evacuated during the storm.

- Second type of platforms are process platforms with attached living quarters where large number of people staying on board, platform will be shutdown, and the people will be evacuated during a severe storm.

- **Pull out case** - Minimum gravity loads arising from extreme case with 100 year return period storm wave, current and wind. This case is used to check the maximum tension loads on the piles and structural members.

- **Seismic case** - Maximum gravity loads with seismic loads from either strength level earthquake or ductility level earthquake as per actual design requirement. This case is used to check the seismic condition if the platform is located in seismically active region.

Depending on type of platform operation such as process, wellhead / drilling or living quarters, the required load combinations during a normal operation or extreme storm may change. This is illustrated in the Tables.

The above table is based on the Working stress method (WSD) and does not include any contingencies. Suitable contingency shall be added to the load combination.
### Load Combinations

Table 3.5: Load Combination for an offshore platform design based WSD

<table>
<thead>
<tr>
<th>No</th>
<th>Load Category</th>
<th>Normal Operating case</th>
<th>Hydro-test case</th>
<th>Extreme Storm case</th>
<th>Tension Pullout case</th>
<th>Seismic case</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Gravity Loads</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Structural Dead Loads</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>0.90</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>Mechanical Equipment</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>0.90</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>Piping &amp; Bulks</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>0.90</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>Electrical Equipment</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>0.90</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>Instrumentation</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>0.90</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>Operating Fluids</td>
<td>1.00</td>
<td>-</td>
<td>1.00</td>
<td>-</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>Hydro-test Fluids</td>
<td>1.00</td>
<td>1.00</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Live Loads</td>
<td>1.00</td>
<td>0.50</td>
<td>-</td>
<td>-</td>
<td>0.50</td>
</tr>
<tr>
<td>2</td>
<td>Drilling Loads</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Drilling Equipment</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>0.90</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>Supplies</td>
<td>1.00</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0.50</td>
</tr>
<tr>
<td></td>
<td>Hook Loads</td>
<td>1.00</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Rotary Loads</td>
<td>-</td>
<td>-</td>
<td>1.00</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Environmental Loads</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Operating Wind</td>
<td>1.00</td>
<td>1.00</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Extreme Wind</td>
<td>-</td>
<td>-</td>
<td>1.00</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Operating Wave+current</td>
<td>1.00</td>
<td>1.00</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Extreme Wave+Current</td>
<td>-</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>Seismic Loads</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Chapter 3. LOADS
Chapter 4

MATERIALS AND CORROSION

4.1 General

Selection of suitable material for the offshore structure is a very important aspect. Materials for offshore applications include the following.

- **Structural Materials**
  - Structural Steel - Low carbon steel used for ll kinds of structural members in the platforms
  - Structural concrete - Marine grade concrete used for construction of columns and bases of gravity type platforms
  - Cement grout - used for grouting annulus between the pile and jacket leg or skirt

- **Piping Materials**
  - Carbon Steel used for normal process piping
  - Stainless steel
  - Steel Alloys such duplex steel, super duplex steel etc for transporting corrosive fluids such crude oil etc
  - Fibre Reinforced Plastics - used for sea water service such as fire water system

- **Miscellaneous Materials**
  - Fibre Reinforced Plastics - used for splash zone protection of piles and members, marine growth preventer etc
  - Fibre Glass - used for grating on offshore platforms
  - Timber - used for miscellaneous applications such as launch skid, temporary works, mudmat, etc.
4.2 Structural Steel

Offshore structures are generally constructed of structural steel. But these structures are mostly either fixed template type structures and deep water complaint structures. Some times the concrete gravity type structure are also constructed. In this chapter, the type of steel materials and its behaviour under water is discussed.

4.2.1 Structural Member Classification

Structural members in the offshore structures can be classified into following categories based on the criticality.

- **Primary Structural Member** - Primary Structural members shall include members and components essential to the overall integrity and safety of the primary structure.

- **Secondary Structural Member** - Secondary Structural Steel shall include members and components essential to the local integrity of the structure where failure of these members will not affect the overall integrity and safety of the primary structure.

- **Special Members** - Special members are parts of primary members located in, or at the vicinity of, critical load transmission areas and of stress concentration locations.

- **Ancillary Members** - Ancillary’ members are minor members and attachments, which do not fall in the above categories.

Typical examples in each category is given in Table 4.1.

<table>
<thead>
<tr>
<th>Category</th>
<th>Deck</th>
<th>Jacket</th>
</tr>
</thead>
<tbody>
<tr>
<td>Primary</td>
<td>Legs, main truss members,</td>
<td>Legs, piles, braces</td>
</tr>
<tr>
<td></td>
<td>plate girders</td>
<td></td>
</tr>
<tr>
<td>Secondary</td>
<td>stringers, floor plates</td>
<td>Minor braces, caissons,</td>
</tr>
<tr>
<td></td>
<td></td>
<td>conductor supports</td>
</tr>
<tr>
<td>Special</td>
<td>Nodes, lifting padeyes</td>
<td>Nodes, padeyes</td>
</tr>
<tr>
<td>Ancillary</td>
<td>handrails, platforms, supports</td>
<td>walkways, bumpers and guides</td>
</tr>
</tbody>
</table>

Table 4.1: Typical member types in Deck and Jacket

4.2.2 Steel Grades

The four (4) grades of steel, which may be used to fabricate plate, rolled shapes, built-up girders and beams, and tubulars, are
• Mild Steel (MS)
• Special Mild Steel (SMS)
• High Strength Steel (HS)
• Special High Strength Steel (SHS)

Mild Steel (MS)

a) Mild Steel (MS) plate, shapes, and plate for tubulars, shall conform to ASTM A 36, “Specification for Structural Steel”.

b) Structural grade seamless line pipe may be used for tubular less than 508 mm O.D., for handrails, grating supports, walkways, or as shown on the Fabrication Drawings. Line pipe shall meet the requirements of API Specification 5L Grade B or ASTM A 53 Grade B.

c) All mild steel shall have a minimum of 250 MPa yield strength.

Special Mild Steel (SMS)

Special Mild Steel (SMS) plate shall conform to ASTM A 36, except that this steel shall be manufactured to a fully killed, fine grain practice, grain size six (6) and finer as determined by ASTM E 112.

High Strength Steel (HS)

a) High Strength Steel (HS) plate shall conform to the requirements of API 2H Grade 50 or ASTM A 633 Grade C.

b) High Strength Steel (HS) for rolled shapes shall conform to the requirements of ASTM A 633 Grade D.

c) These grades of steel shall be fully killed, fine grain practice, grain size six (6) or finer as determined by ASTM E 112.

d) The carbon equivalent index shall be for Grade 50 steels as defined by API Specification 2H.

e) High Strength Steel for tubular less than 508 mm O.D. may conform to seamless API Specification 5L.
Special High Strength Steel (SHS)

- Two (2) special high strength steels may be used in the structure(s):

  - "SHS-50" shall conform to the requirements of API 2H Grade 50 or ASTM A 633 Grade C, Normalized High Strength Low Alloy Structural Steel with API 2H supplements S-1, S-4 and S5 applicable to either designation.

    b) These materials shall be fully killed and normalized, grain size six (6) or finer as determined by ASTM E 112.

- The steels shall meet the requirements of ASTM A 578, "Straight Beam Ultrasonic Examination of Plain and Clad Steel Plates for Special Applications" and ASTM A770. For ultrasonic examination acceptance level II shall be used and the entire plate shall be scanned. Supplementary requirements S1 and S4 shall apply.

- The maximum carbon equivalent shall be as defined by API Specification 2H for Grade 50 steels.

4.2.3 Manufacturing

Materials shall be produced by a manufacturing process which includes ladle vacuum degassing, calcium argon stirring, or other suitable techniques which shall result in removal or shape control of the complex oxy-sulfides. The supplier shall provide a guarantee and substantial proof that the required properties and weldability can be achieved with the proposed process. Melting by the open hearth process shall not be allowed.

4.2.4 Chemical Requirements

The maximum allowable sulfur content shall be 0.006 percent. Rare earth metal (REM) additions shall not be permitted. This approval shall be contingent upon a review of the details concerning the type and amount of REM additions, how and at what point in the melting - ladle degassing process are the REM added, the sulfur level and the extent of degassing prior to the REM addition, and the method used for analysis and control.

4.2.5 Carbon Equivalent (CE)

\[
CE = C + \frac{Mn}{6} + \frac{Cr + Mo + V}{5} + \frac{Ni + Cu}{15} \tag{4.1}
\]
4.2.6 Mechanical Requirements

For grade SHS-50, the minimum yield strength shall be 345 MPa and the minimum tensile strength shall be 483 MPa for plates through 100mm in thickness.

4.2.7 Notch Toughness Requirements

The Charpy V-notch impact test shall be used for all high strength ASTM materials for notch toughness requirements. The test shall be conducted and acceptance criteria shall be in accordance with API Specification 2H.

4.2.8 Supplementary Requirements

Supplementary requirements for structural steel such as S1 through S11 shall be followed as per API 2H.

4.3 Corrosion

Corrosion of metals is a electro-chemical process of loss of material from one location. This may get deposited in the same metal in another area. Corrosion in a wet environment attacks the metal by removing the atoms on the metal surface. The metal atoms at the surface lose electrons and become actively charged ions that leave the metal and enter the wet electrolyte. The metal ions join with/to oppositely charged ions from another chemical and form a new, stable compound.

4.3.1 Seawater Corrosion

Corrosion in a wet environment attacks the metal by removing the atoms on the metal surface. The metal atoms at the surface lose electrons and become actively charged ions that leave the metal and enter the wet electrolyte. The metal ions join with/to oppositely charged ions from another chemical and form a new, stable compound.

Corrosion requires energy. During corrosion the reacting components go from a higher to a lower energy state and release the energy needed for the reaction. In the dry corrosion process of Figure 1 the metal and the oxygen combine to produce the oxide on the surface because the reaction leads to a compound (the oxide) at a lower energy level.
The oxide layer shields the metal from the oxygen and forms a barrier. The oxide will not react with the oxygen in the air or the metal. The barrier makes it difficult for oxygen in the air to contact the metal and it eventually grows so thick that the movement of electrons and ions across it stop. Provided the oxide layer does not crack, or is not removed, the metal is protected from further corrosion. But if the bare metal is exposed to the oxygen, it will again react to form the oxide. In this case the presence of oxygen benefits the metals protection. Removal of the oxygen removes the metals ability to create its own protective corrosion barrier.

Corrosion of steel in marine environmental is basically a electro-chemical reaction. This is caused by flow of electrons from one location to another location results in recombined metal or rust deposited in the receiving location called **Cathode**. The location from where the metal is being taken away is called **Anode**.

The essential requirement of such metal transfer to occur is closed circuit between the two locations. This is provided by the surrounding seawater which is a best electrolyte. The presence of chloride, accelerates the process of corrosion.

This process is schematically showing in Figure 4.1.

In the above process the electrons from the corroding anode metal move to the connected cathode where they recombine with the atoms of oxygen and water in the electrolyte to make a new hydroxyl ion (OH-). This new negatively charged ion then reacts to make a stable compound with the positively charged metal ions ($F^2+$) that originally lost the electrons. In this case, the electrons have a continuous pathway to escape the parent metal and the parent metal, which cannot develop a protective barrier, disassociates or falls apart. Once corrosion starts it continues until the ingredients are all used up.

### 4.3.2 Factors influencing corrosion

The factors influencing or accelerating the sea water corrosion process are listed below.

- **PH value of sea water**- The electrolyte in wet corrosion can be neutral, acidic or alkaline. For corrosion in near neutral solutions (pH 6 - 8) under oxygenated conditions the predominant cathodic reaction is the oxygen absorption reaction (O2 + 2H2O + 4e- = 4OH-) shown in Figure 1. If instead the bimetallic cell has no oxygen present in the electrolyte the hydrogen evolution reaction (H+ + e- = H followed by H + H = H2 gas) becomes the cathodic process and the anode continues to corrode. This reaction is a much slower reaction (the H+ ion has a very low concentration in solution) than the oxygen absorbing reaction. In acidic solutions (pH 0 - 6) the hydrogen ion concentration is higher and the hydrogen evolution reaction is the predominant one. Corrosion rates become extreme as the pH drops (acid gets stronger).
4.3. Corrosion

Cathode

Anode

Electrolyte - Sea Water

Presence of Oxygen

\[ \text{H}_2\text{O} \]

\[ \text{O}_2 \]

\[ 2\text{H}_2\text{O} + \text{O}_2 + 4\text{e}^- \rightarrow 4\text{OH}^- \]

\[ \text{Fe}^{2+} + \text{OH}^- \rightarrow \text{Fe(OH)}_2 \]

\[ \rightarrow \text{Rust} \]

Corroding Metal - Fe

\[ \text{Fe} \rightarrow \text{Fe}^{2+} + 2\text{e}^- \]

Figure 4.1: Schematic showing the sea water corrosion process

- **Salt Content** - The presence of chloride in sea water increases the chemical reaction that takes place during the corrosion process.

- **Temperature of water** - The effect of temperature on corrosion rate is very important. The rate of corrosion increases as the electrochemical reaction gets faster in high temperature.

- **Velocity of flow** - The rate of corrosion is higher in water circulating or flowing through than in the stagnant water. During corrosion, ions build up immediately around the anode and cathode saturating their respective regions. The corrosion rate begins to fall due to the concentration of stagnant ions blocking the creation of more ions in the electrolyte. If the ions are removed or more voltage is provided the corrosion rate again picks up. If you want fast corrosion then agitate the electrolyte and add oxygen.
• **Presence of Oxygen** - Presence of oxygen is main cause of corrosion process to start. The corrosion reaction requires oxygen and where oxygen is present the metal is cathodic and where oxygen is depleted the metal is anodic and corrodes. The parts of the metal in contact with the highest oxygen concentration become cathodic and are protected, and the areas where oxygen concentration is low will corrode. Steel posts dug into the ground will rust just below the surface because of this effect.

### 4.3.3 types of corrosion

- **General / Uniform Corrosion** - this is basically a global corrosion occurring uniform over the exposed surfaces of the metal. Typical example of uniform corrosion is the corrosion of offshore structures in sea water.

- **Localised Corrosion** - Some of the localised corrosion effects are listed with brief description of how it happens.
  - Pitting Corrosion - This type of corrosion happen due to presence of some kind of opening in the metal surface such as fatigue cracks. In pitting corrosion the metal at the top of the pit has access to the oxygen in the air and becomes the cathode. At the bottom of the pit oxygen is depleted and the metal becomes the anode. The deeper the pit is the less the oxygen available at the bottom and the corrosion rate increases.
  
  - Crevice Corrosion - A crevice is created whenever two objects are brought together. Unless they are perfectly flat a crevice is present and oxygen cannot easily enter the gap but is plentiful outside. Corrosion starts in the crevice because of differential aeration.
  
  - Stress Corrosion - Metal under tensile stresses can corrode at higher rates than normally expected. The stressed areas have changed electrical potentials to the neighbouring metal and are also more likely to develop microscopic surface cracks. Both situations promote increased corrosion rates.
  
  - Galvanic Corrosion -
    
    Galvanic corrosion needs to be watched. Dissimilar metals of different potentials joined together by an electrolyte, like process water or rainwater, will cause the more anodic metal to corrode. Running copper water pipe to a galvanised tank will cause the tank to corrode very quickly. Joining copper to steel is nearly as bad. In the galvanic series listed in Table 1 only join metals that are near each other.

    Some protection from galvanic corrosion can be achieved if the electrolyte is not present. Without the availability of water molecules the corrosion reaction stops because the electrons cannot find a host to complete the chemical reaction. Where dissimilar metals must be used, for example aluminum fins on the copper coils of a refrigeration chiller condenser, protect them from contact with water. If water
must be used in contact with dissimilar metals insure it is deionised and oxygen free.

![Diagram of corrosion](image)

Figure 4.2: Schematic showing the pitting and crevice corrosion

A metal can corrode without being in contact with another metal. In this case different areas of the metal take on different electrical potentials. This can occur because of variations in the metal metallurgical properties or because of variations in the surface oxide layer, such as a break, thinning, inclusion like mill scale, contaminant like dirt, etc.

In pitting corrosion the metal at the top of the pit has access to the oxygen in the air and becomes the cathode. At the bottom of the pit oxygen is depleted and the metal becomes the anode. The deeper the pit is the less the oxygen available at the bottom and the corrosion rate increases. Figure 2 shows the mechanism of pitting corrosion.

### 4.3.4 Rates of corrosion

Variation of corrosion along the depth of water depends on many parameters discussed in the previous section. However, a general trend is that the corrosion rate decrease as the depth increase. This is due to lack of presence of oxygen and decrease in temperature. However, the salt content increases with the depth. Hence the rates of corrosion is a complex phenomenon changes from location to location. The corrosion rate of 0.1mm to 0.5mm per year in the splash zone and 0.05mm to 2mm below the splash zone has been given in the literature.

In case of jacket type structures, generally the corrosion allowance in the splash zone vary from 6mm to 12mm and for other regions below, normally no corrosion allowance is provided since sacrificial anodes are provided to protect the steel. The longer the design life of the structure, the corrosion allowance in the splash shall be higher. But some times, a non-corrosive splash zone protection in the form wrapping using FRP materials are also provided in place of corrosion allowance.
4.4 Corrosion Protection

There many ways of protecting the structures against corrosion. The possible methods are listed below.

- Select base materials such that they have corrosion resistant property inherently.
- Providing protective coating or other means to stop the environment from attacking the steel surface.
- Cathodic Protection by means of sacrificial anodes or impressed current system.
- Providing corrosion inhibitors to stop the corrosion process
- Providing sacrificial corrosion allowance in areas where protection by other method does not work.

4.5 Cathodic Protection

Cathodic protection has applications in most of the offshore structures and facilities.
4.5. Cathodic Protection

4.5.1 Sacrificial Anode System

A schematic showing the sacrificial anode system is shown in Figure 4.4. The metal surfaces to be protected shall be connected to a sacrificial anode. In this method, the protective current that flow from the sacrificial anode prevents the flow of corrosive current flowing from anode to cathode. This way the metal corrosion is stopped. Instead, the sacrificial anode corrodes during the process.
For a freely corroding mild steel in sea water has a potential difference of -0.50 to -0.60 Volts when compared with Silver/Silver Chloride reference electrode. This negative potential increases further when cathodic protection is applied. This negative volts increases more than -0.80 Volts and this indicates that the cathodic protection system is in working condition.

This method is very suitable for offshore fixed type platforms. This method does not require any maintenance and no external resources for operation. A typical fixed offshore platform as shown in Figure 4.6 is provided with many number of anodes distributed from mudline to LAT. It is to be noted that the cathodic protection by means of sacrificial anodes does not work in the splash zone due to intermittent exposure. Hence the anodes need not be provided in the splash zone.

The amount of sacrificial anodes required to protect the structure depends on the following parameters and shall be carefully studied.

- Seawater Resistivity, Salinity, temperature and flow velocity
- Total Surface area to be protected
- Type of Anode Material and its composition, size and shape

Among the various types of anodes used in the industry, the slender stand-off type anode commonly selected due to its simplicity.
Figure 4.6: Offshore platform protected with sacrificial anodes
Chapter 5

SIMULATION

5.1 General

With the advancement in computer and software technology and availability of computers, the structural analysis of structures has been made easy and fast. There are a number of commercial computer programs available specifically coded to carry out three dimensional structural analysis for offshore structures. Few programs are listed below.

- **SACS** - Structural Analysis Computer System - from Engineering Dynamics Inc. USA
- **Strucad** - Also from Engineering Dynamics INC. USA
- **SESAM** - from Det Norske Veritas, Norway

The modern day offshore development project schedules does not permit designers to carry out hand calculations due to faster requirement of design and drawings for fabrication. Usually, the first discipline to produce documents and drawings is structural so that the materials can be ordered to mill for production. Hence the structural designers are under very high pressure from fabricators to produce the structural material take off for order placement. The use of structural analysis programs with fast computers has made possible some of the largest structures to be designed in 6 to 8 months.

Following preparatory activities are required before analysis and design can be carried out.

- Structure Geometry selection
- Geometry Simulation
5.2 Structure Geometry selection

Structure geometry shall be selected based on various requirement such as layout, water depth, environmental condition, installation methodology and topside loads etc.

5.3 Geometry Simulation

A geometric model of a structure contains a database of following information.
5.3. Geometry Simulation

- Joints or Nodes
- Members and Properties
- Foundation
- Loads

Each of the above information can be entered in a planned and systematic way so that the post processing and correlating the design drawings with analysis results becomes easier and faster.

5.3.1 Axis System

Any computer model of structure require origin and coordinate system so that the structure information such as nodes and loads can be modeled. Normally, orthogonal coordinate system with X, Y and Z will be used. The origin of such system shall be at the geometric center of the platform in plan at the work point and at Mean Sea Level (M.S.L) in elevation as shown in Figure.

Figure 5.2: Computer Model of a Jacket with Axis system
The axis origin at the center of the jacket helps in modeling and transformation of geometry for further installation analyses.

### 5.3.2 Joints or Nodes

Joints or nodes are defined as three dimensional coordinates of junctions of members in the space frame. Joints are work points (W.P.) of ends of members and the member shall be formed along the centreline of the member. Unique numbering system shall be followed so that identification of location of joint becomes easy. For example following joint numbering can be used.

<table>
<thead>
<tr>
<th>No</th>
<th>Item Description</th>
<th>Joint Numbering</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Jacket</td>
<td>J1 to J999</td>
</tr>
<tr>
<td>2</td>
<td>Deck</td>
<td>D1 to D999</td>
</tr>
<tr>
<td>3</td>
<td>Flare Boom</td>
<td>F1 to F999</td>
</tr>
<tr>
<td>4</td>
<td>Bridge</td>
<td>B1 to B999</td>
</tr>
<tr>
<td>5</td>
<td>Living Quarters</td>
<td>Q1 to Q999</td>
</tr>
</tbody>
</table>

Table 5.1: Joint Numbering scheme

Some software programs has the ability to generate automatic joint numbering based on initial number given by the user. Each panel or plane framing can be given with unique numbering. This numbering schemes will help a lot during post processing of results and identification of members or joints that require redesign.

For example, in the deck joint numbering scheme, we can split the numbering system in to blocks and use them in a systematic way. For cellar deck, D1 to D399, and for main deck D400 to D799 and for upper deck D800 to D999 and so on. This way, we can easily identify which node is lying on which deck etc. This method of modeling can only come by experience and can be improved to produce a more user friendly model. This will definitely help during quick processing of results and compiling the information from various analyses for a jacket such as inplace, fatigue, seismic, loadout, transportation and launching etc.

### 5.3.3 Members and properties

A member in structural model is a an element (usually a beam element in terms of Finite Element Terminology) connected between two nodes or joints. This member represents the presence of a structural member in the analysis. The member can be defined by two joints (one at each end of member) and a member group name (under which the member property such as diameter and wall thickness can be assigned).
Similarly, the member properties can also be given in systematic way. Same member group shall not be given to too many segments or members.

<table>
<thead>
<tr>
<th>No</th>
<th>Item Description</th>
<th>Member group</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Jacket</td>
<td>Prefix with either L or J</td>
</tr>
<tr>
<td></td>
<td>Legs</td>
<td>Prefix with H</td>
</tr>
<tr>
<td></td>
<td>Horizontal Braces</td>
<td>Prefix with V</td>
</tr>
<tr>
<td></td>
<td>Elevation braces</td>
<td>Prefix with T</td>
</tr>
<tr>
<td></td>
<td>Launch Truss</td>
<td>Prefix with BL</td>
</tr>
<tr>
<td></td>
<td>Boat Landing</td>
<td>Prefix with BL</td>
</tr>
<tr>
<td>2</td>
<td>Deck</td>
<td>Prefix with C</td>
</tr>
<tr>
<td></td>
<td>Cellar Deck</td>
<td>Prefix with M</td>
</tr>
<tr>
<td></td>
<td>Main Deck</td>
<td>Prefix with U</td>
</tr>
<tr>
<td></td>
<td>Upper Deck</td>
<td>Prefix with P</td>
</tr>
<tr>
<td></td>
<td>Crane Pedestal</td>
<td>Prefix with F</td>
</tr>
<tr>
<td>3</td>
<td>Flare Boom</td>
<td>Prefix with F</td>
</tr>
<tr>
<td>4</td>
<td>Bridge</td>
<td>Prefix with B</td>
</tr>
<tr>
<td>5</td>
<td>Living Quarters</td>
<td>Prefix with Q</td>
</tr>
</tbody>
</table>

Table 5.2: Member Group scheme

the above is list in an indication of how systematic a structural model can be made and it is upto the designer to build it the way he wanted.

5.3.4 Offsets and Eccentricities

The members connecting at one common joint can be joined to one single joint if the offset between the work points (W.P.) is less than the API RP2A allowable distance of $D/4$ where D is the diameter of the chord member at the joint. This is based some studies on tubular joints with eccentricities showed that the additional moment generated by these eccentricities have consequences on the member and joint adequacy. The shear capacities of tubular members are generally high and may not be serious issue is the eccentricity is within $D/4$.

However, this rule shall not be applied to deck truss. The column, brace and plate girder junction shall be modelled with additional joint even if the eccentricities are less than $D/4$. This is required to model the shear transfer between the brace and plate girder through the web of the girder. If a separate joint is not modelled, the vertical component of the braces forces will directly taken by the column and the shear capacity of the plate girder at the joint may be checked correctly.
5.3.5 Wish-bone Connections

The conductors are tubes inserted through series of guides in the jacket at various levels and driven in to the seabed for at least 30 to 40m. These tubes are later used to pass through the drill safely without damaging the jacket members. Further, these tubes prevent the drill pipes (normally very small diameter, typically 13 to 16 inch) from failure due to large displacement and vortex induced vibration. These conductors are not structurally connected to jacket members but just touching the conductor guides at different elevations. Total number of such conductors may vary from platform to platform, in some cases, there may as much as 30 to 40 conductors of 30 inch diameters. Wave and current loading on conductors need to be included in the structural analysis and hence they need to to modelled.

The conductors and the structural elements at the conductor guide level will be modelled through a "stiff element” called **wish bone** with only axial stiffness so that it will transfer
the lateral loads from wave and current on conductors. The length of the wishbones will be very small. This is done by creating a joint with almost same coordinate of the structural joint.
5.3.6 Dummy Structure Models

Jacket structure has many appurtenances attached to it. The wave and current loading on them shall be taken into account but the stiffness need not considered in the global analysis of jacket. This is to simplify the load path and to understand the behaviour clearly. Further the materials of such appurtenances may not be in line with the main structure, it is obvious to ignore them in the global structural stiffness. Such structures are listed below.

- Boat Landing
- Barge Bumber
- Launch Runner
- Mudmat
- Anodes
- Conductors
- Risers
- J-tube

Similarly, the deck structure has many appurtenances as listed below.

- Walkways
5.4. Foundation Simulation

- Stairways
- Platforms
- Equipment Supports
- Monorails

But these items on the deck does not have wave or current loading but may have wind loading on them. Since the wind loading on deck is normally considered as block area, this may not be the case. But these appurtenances need to be modelled to simulate the vertical COG. This is required in the sea transportation analysis and seismic analysis. Refer to relevant section for more details.

The dummy structure can be modelled in two ways. The first method is to model as a structural member and give very high Young modulus so that the stiffness is very low. This method will lead to large displacements and may not be correct.

The other method is to remove such members after the loads are calculated and summed to the nearest joint. Some of the software programs have this ability to simulate dummy structure based on this method.

5.4 Foundation Simulation

5.4.1 Pile Modelling

In an offshore structure, the piles hold them on to the sea bed. This needs to be simulated in the structural analysis involving their inplace strength and stability. There are type of pile system that can be used in the offshore structures.

- Main Pile
- Skirt Pile

As it can be seen from the figure that the skirt pile is always grouted with the skirt sleeve of the jacket. But in the case of main pile, the annulus between the pile and the jacket leg may be grouted or not grouted depending on the design water depth. Like other structural elements of the jacket structure, pile is also a structural member and shall be modelled according to the diameter, wall thickness and material properties. It is the load transfer mechanism between the jacket leg and pile that requires special care in simulation of actual load transfer.
Figure 5.7: Pile Simulation for an Offshore jacket
For the case of grouted skirt piles and main piles, the model becomes much easier by simply specifying the cross section as a "Composite Section" containing jacket leg, pile and the annulus filled with cement grout. The equivalent axial area, shear area and bending stiffness can be calculated using the equivalent section concept and used in the analysis.

But for the case of main pile, this cannot be done. The pile and jacket are two parallel members physically connected at the top of jacket by means welded connections and elsewhere no welding but spacers are placed inside the jacket leg to provide contact points for load transfer. These spacers are specially located at the horizontal framing such that the lateral loads from the wave and current can be easily transferred to the piles.

5.4.2 Soil Simulation

Piles below seabed shall be modelled in the structural analysis to reflect the vertical and lateral behaviour of pile soil system. This is very essential to simulate the jacket and deck deflections and pile stresses. This can be done in three ways.

- Equivalent Pile Stub -
- P-Y, T-Z and Q-Z Curves -
- Linearised Pile Stiffness Matrix -

5.4.3 Pile Group Effect

The skirt piles for very large jackets normally arranged in cluster at each corner to resist the forces from gravity and environmental loads.

These pile clusters can be arranged in various ways but due to construction limitations usually they will arranged in closed manner as shown in the Figures 5.10 and ?? . The distance between the jacket leg and the farthest pile shall be kept to a minimum possible for fabrication to avoid unnecessary bending on jacket legs as well on the pile sleeves.

It is a good practice to space the centre to centre of adjacent piles at a distance of 3D where D is the diameter of the pile. This will prove a clear distance between the pile face of 2D. Even with this separation, the effect of load on one pile will affect the behaviour of the adjacent pile. The issues to be looked into are two categories as listed below.

- Effect Axial Capacity
- Effect on P-Y, T-Z and Q-Z behaviour
Figure 5.8: Pile Simulation for an Offshore jacket
Figure 5.9: Pile Group arrangements for 4 legged platform

Figure 5.10: Pile Group arrangements for 8 legged platform
5.5 Load Simulation

5.5.1 Dead Loads

The dead loads of primary structural members such as deck beams, braces, jacket legs and braces, piles etc shall be calculated by the program automatically based their dimensions and unit weight of material supplied. Hence in the simulation of geometry, the structural members shall be modelled as close as possible to the actual arrangement including member offsets. Otherwise, the weight of the member may not be calculated accurately. Especially for jacket, if the offsets at the joints are not modelled, then the buoyancy will be either over estimated or under estimated depending on the member weight to buoyancy ratio. Typical K joint of a jacket is shown in Figure.

5.5.2 Equipment Loads

Generally, the equipment weight are manually entered based on VENDOR supplied information. The weight of the equipment shall be distributed to the deck beams or plating depending the load transfer method adopted for the design of the equipment skid. Equipment skids may have been designed for four corner support or continuous supports. Provision of continuous supports shall be carefully examined as it may be very difficult to achieve.

Hence these equipment loads shall be distributed to the deck beams at appropriate locations based on equipment Center of Gravity (COG) as point loads.

Only very small equipment may be placed directly on the deck plating. This can be applied as patch loads on the deck plating.

5.5.3 Fluid Loads

The fluid loads are based on equipment operating weight. This can also be obtained from the equipment manufacturer. The contents of the equipment can be calculated as below.

\[ W_{\text{fluid}} = W_{\text{oper}} - W_{\text{dry}} \]  \hspace{1cm} (5.1)

where \( W_{\text{oper}} \) and \( W_{\text{dry}} \) are the weight of equipment in operating and dry conditions. Similarly, the contents of the piping during operation can be estimated based on length of pipes, diameters etc. This can only be estimated very approximately and large contingency shall be applied on this item.
These loads shall be applied as member load on the deck beams. The distribution of these loads shall be as per the estimated overall piping COG at each deck levels.

### 5.5.4 Drilling Loads

Drilling equipment include rig, drill strings, mud tanks, etc. These equipment are also similar to the other equipment described above except that the drilling rig is not an fixed equipment.

Usually, the drilling rig is designed to be used for drilling more than one well. Each platform may have a matrix array of wells either 3x3 or 4x4 depending on the field development plan. Hence these drilling rings will be mounted on skid beams (part of deck with raised T sections as rails) so that the drilling rig can be moved longitudinally. Across the other direction, the drill dig will have a skid arrangement to move. This arrangement will produce different reaction on to the deck structure at each drilling location.

Normally, these shall be applied as point loads on the skid beams. There may be several load cases to cover all the well positions. The complication is due to application of wind loads on these drilling rig structure. Each time the drilling rig changes position, the wind load also shall be applied to the corresponding load point. This will lead to several combinations in the global inplace analysis.

### 5.5.5 Live Loads

The live loads shall be applied on open areas not occupied by equipment or facilities. This can be applied as member loads.

### 5.5.6 Wind, Wave and Current

#### Wind

Wind loads are normally calculated manually and applied to deck edge usually on nodes at the periphery. Diagonal or non-orthogonal wind load cases can be generated from loads from orthogonal cases. For example, the loads in +X, +Y, -X and -Y will be applied manually. The load case for 45 degree can be generated suitably using loads in +X and +Y. The load combination factors are discussed in the earlier Chapter “Loads”.

Wave and Current

The wave and current shall be simulated using software contained modules. Manual calculation and application of these loads will lead large errors and approximations since the number of members are very high. However, following points shall be noted in selecting various parameter for calculation.

- **Wave and Current Directions** - Normally for a 4 or eight legged jacket, at least 8 directions shall be considered. For tripod structures, at least 12 directions shall be considered. Diagonal directions shall be selected to maximize the pile loads. For example, if the platform geometry in plan is a square, 45 degree angle will produce maximum pile load. But if the geometry is a rectangle, then the angle shall be the diagonal angle joining the diagonal pile groups as shown in Figure 5.11

- **Selection of suitable Wave Theory**

- **Hydrodynamic Coefficients**
  - $C_D$ and $C_M$
  - Current Blockage Factor
  - Conductor Shielding
  - Kinematics Factor
  - Apparent Wave Period

- **Simulation of Non-structural items**
  - Boat Landing
  - Barge bumper
  - Walkways
  - Anodes
  - launch Runner
  - Skirt Sleeve Connections
  - Risers and J-tubes
  - Mudmat
Figure 5.11: Wave attack angles for platform with square and rectangular base
Chapter 6

In-place Analyses

6.1 Principle

In-place analysis can be carried out in two ways.

- Pseudo Static Analysis
- Wave Response Analysis

The pseudo static in-place analysis can be carried out as below.

- Geometry Simulation - Simulate the geometry of the jacket as per the drawings and Basis of Design (B.O.D)
- **Load Simulation** - Simulate the gravity and environmental loads as per weight control report and meta-ocean report.
- **Modal Analysis** - Evaluate the dynamic characteristics of the deck/jacket system and determine the dynamic amplification factor (DAF).
- **Foundation Simulation** - Simulate the foundation pile and soil as per the geological report
- **Pile/Soil Interaction** - Carry out static analysis with pile/soil interaction
- **Post processing of results.** - Post process the results to obtain member forces, joint deflections, pile loads, etc.
6.2. Geometry Simulation

The wave response in-place analysis can be carried out as per the steps below.

- **Geometry Simulation** - Simulate the geometry of the jacket as per the drawings and Basis of Design (B.O.D)
- **Load Simulation** - Simulate the gravity loads as per the weight control report and generate results of the analysis.
- **Foundation Simulation** - Simulate the foundation pile and soil as per the geotechnical report
- **Modal Analysis** - Establish the dynamic characteristics of the deck/jacket system and generate modal masses, modal frequencies and mode shapes
- **Wave Response Analysis** - Carry out time series wave response analysis together with the pile/soil interaction.
- **Combine Gravity and Wave Response** - Combine the results of the gravity static analysis due to gravity loads and wave response analysis with appropriate load combination factors.
- **Post processing of results.** - Post process the results to obtain member forces, joint deflections, pile loads, etc.

The analysis will be performed on a 3D space-frame computer model representing the jacket. All primary structural members, caissons and appurtenances will be modelled.

### 6.2 Geometry Simulation

A 3-dimensional rigid spaced frame structural computer model with all members contributing to its stiffness will be generated. This consist of:

- **Legs** - Jacket legs normally consists of normal sections and can section. Member between two adjacent horizontal frames shall be defined using member segments rather than additional joints. Care shall be taken to provide correct length of the CAN at the joints.
- **Braces** - Brace members shall be modelled with offsets from the jacket legs. Any eccentricities more than \( D/4 \) shall be modelled by adding additional joints.
- **Appurtenances** - These include conductors, risers, walkways, mudmat, anodes, etc. These needs to accounted for both their air weight and wave load purposes.
6.2. Geometry Simulation

6.2.1 Simplified Topside Model

The primary frame members within the deck shall be included in the sub-structure models in order to correctly represent the stiffness of the deck and the load transfer from the topsides to the jacket. The other structures, living quarters, drilling rig, drilling support module and vent boom will be explicitly modelled. The loads on these modules and structures will be applied accurately within the structure itself to reflect the actual COG of the loaded structure. Stiff members to appropriate locations within the integrated deck model will connect the individual structural units for the representation of load transfer and stiffness simulation.

6.2.2 Load Simulation

Dead Load

Program SACS using element areas and densities computes the dead weight of all jacket and topside structural elements. The weight of non-modelled components, such as leg diaphragms, pile sleeve guides and appurtenance steel will be input as additional member or joint loads, at appropriate points of application on the structure. The non-structural appurtenance dead weights will be applied as point loads at their points of attachment to the jacket. Where it is impractical to input these as individual point load, to reflect the best current estimate of the structure dead weight, the member densities will be factored up to include the weight of those items. These include miscellaneous pipes, joint and ring stiffeners, conductor casing program, etc. Upper and lower values of jacket weight will be evaluated for use in the load combinations. The lower value of jacket weight will be the base weight. This will be used to check for maximum tension uplift force in the piles. A factor of 1.00 shall be used. The upper value of the jacket weight will include all design allowances.

Wave and Current Loading

Drag and inertia forces on individual members will be calculated using Morison’s Equation. Shielding or interaction effects within the structure will be considered.

The water particle velocities and accelerations for the design waves will be computed using Stream Function Theory in accordance with API RP 2A. For any given position along the wave profile, the specified current velocity profile will be from seabed to the free water surface. Current and wave directions will always be assumed parallel and of the same sense; resultant particle velocities being the vector sum of these components.

For the initial stage of the analysis, the structure will be modelled using a combination of
tubular structural and wave force elements. Structural elements are those elements that attract wave forces and contribute to the stiffness of the structure. Wave force elements attract wave force but are considered to possess zero stiffness. Wave and current forces will be computed using Morison’s equation.

Pile sleeve bottle sections will be modelled as tubular members with effective section properties equal to the pile pitch circle diameter plus sleeve diameter. Pile stick-up of 9.0 m will be allowed on all skirt sleeves. Elements with attachments will have wave loading calculated based on the nominal member section with modified Cm and Cd values.

Similarly, where two or more members are combined into one for the purposes of the wave load analysis, the hydrodynamic coefficients will be modified to give the correct total drag and inertia response.

Drag and inertia coefficients for non-tubular and/or complex geometry will be calculated using an equivalent diameter. The equivalent diameter will be based on the circumscribing circle.

In the calculation of all effective drag and inertia coefficients, the increase in diameter due to marine growth of both the true structural members and the equivalent wave force members will be taken into account where appropriate.

### 6.2.3 Topsides Load COG Shift Envelope

The topsides operating weight and COG, including all allowances and factors will be taken from the latest topside model available at the time of the analysis and used in the jacket in-place analysis. This, by implication implies, that the values used in the detailed design analyses may well differ later on when topside detail engineering becomes more advanced and information more accurate. The load contingency percentages are included to take care of possible weight growth. The load to be used in the Detailed Design analysis will be applied as described below.

An envelope within which the topsides centre of gravity is shifted will be included in the in-place analysis. A load case shifting the COG to each of the four extreme corners of the envelope as well as in between the extreme corners will be included in order to maximise leg loads/frame loads. A COG shift of 1.0 m between north and south, 1.5 m east to west will be envisaged (rectangle box).
6.2.4 Minimum and Maximum Water Depth

In-place analysis shall be carried out for minimum and maximum water depth cases. The minimum design water depth shall be the water depth to LAT and the maximum water depth shall include storm tide (100 year) and any surge shall also be included.

6.2.5 Maximum Wave Loads

- Maximum Base Shear
- Maximum Overturning Moment

6.2.6 Allowable Stresses

Allowable stresses for members and joints shall be taken as per API RP 2A. One-third increase in allowable stresses is allowed for storm load cases.
Chapter 7

Dynamic Analysis

7.1 Principle

Basically, the dynamic analysis is carried out to determine the natural periods, mode shapes etc for further use in the seismic analysis, spectral fatigue analysis. Further, the natural period will be used for the calculation of Dynamic Amplification Factor (DAF) for both inplace storm analysis and fatigue analysis.

7.2 Geometry Simulation

The stiffness model for the jacket will be developed from the in-place model with the specific modifications detailed below:

The model incorporates plan framing and vertical trusses, providing overall stiffness simulation.

Additional joints required for mass modelling are provided at the centre of gravity positions of the drilling, drilling support module, LQ and other deck equipment for mass modelling. These joints will be connected by triangulated (pyramid form) stiff links to the deck model. Stiff links will be created by applying large section properties to the link members, however densities should be set to zero to ensure that there are no effects due to mass of these members.

The jacket model used in the natural frequency analysis is similar to the one used in the inplace analyses. Modifications will comprise alteration to the Foundation Model as described below, preparation of a complete mass model and any alterations to appurtenance simulation necessary to minimise the number of potential spurious local vibration modes.
Stiffness matrices developed for extreme storm condition, corresponding to the 100-year wave height will be used to generate the linearized foundation stiffness.

7.3 Load Simulation

The mass model comprises the structural mass, water added mass, contained mass and marine growth. Added mass is the mass of water assumed to move in unison with the structural member as it deflects. For tubular, a value of mass numerically equal to the mass of water displaced by the submerged member is used including marine growth where applicable. Contained mass is the water contained or enclosed by the submerged members.

In this analysis the water depth is taken at mean sea level to be the fatigue design water depth (maximum still water depth). All members below this depth will therefore have an added mass value. The four corner legs will be flooded to top of leg, while all other members will be considered non-flooded.

Pile sleeves, their added masses and contained grout mass will be computed manually and lumped proportionately at the nodes of the members defining the elements.

7.3.1 Computer Generated Mass

The SACS Program will internally compute the structural mass, water added mass, flooded mass and mass of marine growth for all active members (only) of the structural model. Members will be temporarily designated active or inactive depending on whether computer mass generation or direct input mass is required.

For steel mass calculation, active members of the jacket will consist of elements in the corner legs, plan levels and frames. Additional jacket mass due to the nodes will be generated by SACS based on variable sectional properties input for each member so that the total ”stick mass” and node mass add up to the overall steel mass of the jacket. SACS will also calculate the water added mass of each active structural member and the mass of marine growth on it. Each mass will be calculated for a structural member and assigned equally to the joints bounding the member.

7.3.2 Direct Input Mass

The masses that are not generated by program will be calculated by hand and allocated to their appropriate centre of gravity positions at each level. Masses will be based on the weight reports for jackets and topsides and will include the following:
7.4 Foundation Simulation

The dynamic analysis cannot be performed together with the iterative nature of soil-pile interaction.

The non-linear soil-pile system of the jacket foundation is replaced by linear foundation model obtained through Pile Soil Interaction (PSI) analysis of the SACS Program. The foundation model will comprise a 6 x 6 stiffness matrix, representing each pile to provide a linear elastic approximation to the soil stiffness.

7.5 Solution

The solution to the following equation will give the eigen values and eigen vectors.

\[
[M]X + [K]X = 0
\]  
(7.1)

An eigen value analysis will be performed for the jacket model using computer program. The program uses the standard Householder-Guyans extraction technique in solving for the eigen values and eigen vectors of the reduced sets of equations for the model.

The consistent mass approach is adopted to generate the mass matrix. The program first assembles the overall stiffness and mass matrices corresponding to all degrees of freedom of the model. Subsequently, both the stiffness and mass matrices are reduced to have only master or retained degrees of freedom using Guyan reduction procedure.

Matrix decomposition and solution yields eigen values and eigen vectors from which the natural periods are extracted and the mode shapes of the structure plotted. At least twenty-five modes will be considered in this analysis.
7.5.1 Dynamic Amplification factor

From the structural periods derived, the ratio of structural period to wave period will be computed for the relevant waves.

The dynamic amplification factors (DAF’S) will then be derived using the relationship for a single degree-of-freedom system, i.e.

\[
DAF = \frac{1}{\sqrt{\left(1 - \frac{T_n^2}{T^2}\right)^2 + \left(\frac{2\zeta T_n^2}{T}\right)^2}}
\]  

(7.2)

where

- \( T \) = Wave Period
- \( T_n \) = Natural period of structure (first mode)
- \( \zeta \) = Damping factor (5% for steel structures in water)

These DAF’s will be used in the fatigue analyses and in-place analyses if necessary, when the natural period exceeds three seconds.
Chapter 8

Fatigue Analysis

8.1 General

Fatigue analysis can be carried out using the following two methods.

- **Deterministic Method** - In the deterministic method, the seastate energy is simulated using discrete frequencies and wave heights with corresponding number of occurrences. Structural responses and hot spot stresses are generated for each of these discrete waves. The summation of fatigue damages due to these discrete wave load cases are then summed up to obtain the total damage during the life of the structure.

- **Spectral Method** - Spectral method uses the seastate energy spectra used to generate the transfer function for the structural response. This transfer function is then used to generate the hot spot stresses in the joints.

8.2 Deterministic Fatigue Analysis

Deterministic Fatigue analysis of jacket involves the following steps.

- Establish a wave scatter diagram for the field location including ranges of wave height, wave period and occurrences

- Simulation of jacket structure stiffness and deck stiffness accurately and make the model simple enough to understand the behaviour.

- Simulate deck and jacket mass accurately including vertical COG to determine the dynamic characteristics of the platform
8.3 Wave Scatter Data

The wave scatter data contains the sea state information such as wave height, wave period and their distribution over a certain period of time. This information may be available for a 1 year period based on measurements at the site or may be based on numerical simulation validated with bench mark measurements. These data will give a indication of each set of waves with (definitive height and period) will occur how many times in a given period of time. Further, this will also define the direction of approach to the platform. Normally, at least 8 direction sectors (45 degrees each) shall be used for fatigue analysis.

A typical wave scatter data for the middle east (Persian Gulf)is given in Table 8.1.
Table 8.1: Wave Scatter Data

<table>
<thead>
<tr>
<th>T, sec</th>
<th>Tm, sec</th>
<th>H, m</th>
<th>Hm, m</th>
<th>Directional Distribution Percent (%)</th>
<th>W</th>
<th>Total</th>
<th>Percent</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
It should be noted that sometime such information may be only available with significant wave height and zero crossing periods. This shall be converted into individual maximum wave heights and peak period using appropriate factors as shown below the table.

8.4 Spectral Fatigue Analysis

8.4.1 Principle

The spectral fatigue analysis will consider the dynamic response of the structure to a range of wave heights covering a frequency range that includes all sea-states contributing significantly in terms of fatigue damage. The response transfer functions and the sea-state spectra will be used to generate stress spectra according to the well-known power spectral density approach.

Assuming that the joint stress histories conform to a Rayleigh distribution, the fatigue damage will be calculated using Miner’s Rule.

8.4.2 Wave Spectra

Methods of fatigue analysis recognise the probabilistic nature of sea states. The random sea surface can be represented in a deterministic domain by superimposing a finite number of sinusoidal components of amplitude and phase. An energy spectral density function is constructed from the mean square of the amplitude components for each narrow band of frequency, \( f \), and plotted against the appropriate wave frequency.

The two-parameter ISSC modified Pierson-Moskowitz wave spectrum will be used to represent sea surface elevation and is given by:

\[
S_h(f) = \frac{H_s^2T_z}{4\pi(fT_z)^5} \exp\left[\frac{-1}{\pi}(fT_z)^{-4}\right]
\]  

(8.1)

where

- \( S_h(f) \) = Single sided spectral density of wave amplitude \((m^2 \text{unit frequency})\)
- \( H_s \) = Significant wave height \((m)\)
- \( T_z \) = Zero up-crossing period \((\text{sec})\)
- \( f \) = Wave frequency \((Hz)\)
8.4.3 Wave Theory Selection

Water particle velocities and accelerations will be calculated using stream function.

In determining the water depth to use in the deterministic fatigue analysis, care will be taken to recognize that higher water particle velocities, and hence higher local member wave loads, result from shallower water depths. To this end, the Mean Sea Level (MSL) will be used which reflects the long-term average of water depth. Current will be excluded from the analysis.

8.4.4 Wave Steepness

In a fatigue analysis, the forces are assumed to be proportional to wave height. If this assumption were always true, the selection of wave height (wave steepness) would be immaterial to the analysis. However, non-linear effects are present in wave force calculation due to drag influences.

Wave steepness is defined by the ratio of wave height to wavelength. The wavelength is obtained from the appropriate wave theory. For small amplitude deepwater waves, the wavelength in metres is given by:

\[
\text{wavesteepness} = \frac{H_d}{L} \quad \text{where} \quad L = 1.56T^2
\]

(8.2)

where \( T \) is the wave period and \( H_d \) is the design wave height.

From experience a wave steepness of 1:16 will be used for generation of wave forces.

8.4.5 Transfer Functions

In order to perform a fatigue analysis, the wave height spectra must be transformed into a series of member-end hot-spot stress-amplitude response spectra at each of 8 equally-spaced circumferential points around each end of each member for each sea state. This transformation is accomplished by multiplying the appropriate frequency ordinate of the spectrum for the sea state under consideration, by the square of the corresponding ordinate of the hot spot transfer function. The transfer function ordinates are defined as the ratio of hot spot stress amplitude at a particular member-end circumferential point divided by the wave amplitude that generates the stress amplitude at that point as a function of frequency. Consequently, units of the transfer function will be, for example, MP a/m of wave amplitude.
8.4.6 Selection of Frequencies

The inaccuracy introduced when a transfer function is used to convert a wave spectrum is dependent on the number of frequencies for which transfer function ordinates are available. If an infinite number of ordinates were available, the response spectrum would be “exact”. This, however, is obviously not practical. All that is really required is a sufficient number of ordinates to describe adequately the variation of transfer function over the frequency range of interest. For this to be done, the frequencies at which transfer function peaks and troughs can be expected must first be anticipated. Ordinates are then computed at these points and at other intermediate points. A minimum of 8 ordinates will be considered.

To generate a transfer function ordinate for a particular fatigue wave direction, selective waves of various heights but constant steepness are used to load the structure. These waves need not necessarily be the waves from the fatigue environment, but waves chosen based on the following criteria:

**Dynamics Criterion**: The waves with frequencies corresponding to first three modal frequencies will be considered.

**Leg Spacing Criterion**: Certain frequencies are chosen such that the corresponding wavelengths are approximately integral (n) or half integral (n/2) multiples of the distance between legs. This criterion is used to locate the crests and troughs in the transfer function.

**Minimum and Maximum Wave Height Criterion**: Waves with a minimum height of 0.25 m and a maximum wave height obtained among the fatigue wave climate will also be considered for generating transfer function.

8.4.7 Stress Amplitudes

As the wave profile should be a pure sinusoid for the response to be interpreted as a transfer function, the Stream Function theory does result in wave profiles that are pure sinusoids. The stress is calculated at various wave positions and the difference between the maximum and minimum stress called the stress range is determined for each wave. Dividing these stress ranges by one-half of the corresponding wave height produces stress ranges for waves of unit amplitude (for sinusoidal waves, wave height equals twice the wave amplitude). The relationship between the stress ranges of unit amplitude and the corresponding wave frequency for all waves considered is the transfer function.
8.4.8 Fatigue Life Calculation

Calculation of Stress Ranges

If for the direction being considered, the wave height spectral density $S_h(f)$ of a particular sea-state is known and the transfer function $H_i(f)$ for the point can be calculated, then the statistical cyclic stress range (RMS cyclic stress range) at that point for this particular sea-state is given as:

$$
\sigma_{RMS_i} = \sqrt{\int_0^\infty H_i^2(f)S_h(f)df}
$$  \hspace{1cm} (8.3)

Cumulative Damage calculation

For every RMS stress, there exist an average time, $T_z$, between zero crossings with a positive slope for a stationary Gaussian process with zero mean. This period called the zero crossing period is given by:

$$
T_z = \frac{\sigma_{RMS_i}}{\sqrt{\int_0^\infty f^2H_i^2(f)S_h(f)df}}
$$  \hspace{1cm} (8.4)

For a narrow band process, this is the average period or the reciprocal of the average frequency of the process. The expected number of cycles associated with this sea-state during the design life of the structure is:

$$
n(s) = \frac{mL}{T_z}
$$  \hspace{1cm} (8.5)

where, $L$ is the design life of the structure and $m$ is the fraction of the design life that this sea-state prevails.

For a given stress range $s$, the number of cycles to failure, $N(s)$, can be found from the S-N curve used. Then, the expected damage from the given sea-state is given as:

$$
D = \frac{n(s)}{\sigma_{RMS_i}^2} \int_0^\infty \frac{s}{N(s)} \exp\left(\frac{s^2}{\sigma_{RMS_i}^2}\right) ds
$$  \hspace{1cm} (8.6)

The total expected damage for all sea-states during the life of the structure is the sum of the damage for each individual sea-state. The expected fatigue life is equal to the design life divided by the expected damage.
8.4.9  S-N Curves

For a particular stress range $s$, there exists a theoretical number of cycles $N(s)$ at which fatigue failure may occur. The relationship between this number of allowable cycles and the stress range is usually expressed as an S-N curve. The API RP 2A-WSD curve $X$ applicable for normal welding and curve $X'$ applicable for profile control welding.

8.4.10  Stress Concentration Factors

In most tubular joints, local stresses on both the stub and chord side of the stub-to-chord weld will be considerably higher than the nominal stub and chord stresses resulting from a global frame analysis of the structure. The magnitudes of these stress concentrations are dependent upon the weld profile and the joint geometry. Weld profile stress concentrations result from notches in the weld and are difficult to quantify explicitly using full-scale test results or finite element techniques. Consequently, they are incorporated into the empirically derived S-N curves.

Tubular Joints

Several researchers for various unstiffened joint geometry have developed geometric stress concentrations, but those developed by Efthymiou is preferred.

Hot spot stresses on the chord and brace sides of the chord-to-brace weld are determined by multiplying the nominal stub stress by chord and stub stress concentration factors (SCFs) respectively. The manner in which each SCF is computed is dependent on the details of the geometry of the joint under consideration. The following joint configurations exist.

Unstiffened Non-overlapping Joints

SCFs will be derived from the formulae of Efthymiou for all joints. These formulae include parameter ranges within which they are said to be valid. In many cases there is no alternative but to use the formulae outside these ranges, but in these cases the joint will be the subject of special consideration. Normally, SCFs will be computed for the actual parameter values, and again using values on the limit of exceeded ranges. The maximum of those two values will be used.
### 8.4. Spectral Fatigue Analysis

#### Chapter 8. Fatigue Analysis

Figure 8.1: S-N Curve [API RP2A 2000]

Permissible Cycles of Load $N$

<table>
<thead>
<tr>
<th>Curve</th>
<th>$\Delta_{\text{ref}}$ Stress Range at 2 Million Cycles</th>
<th>$m$ Inverse Log-Log Slope</th>
<th>Endurance Limit at 200 Million Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>$X$</td>
<td>14.5 ksi (100 MPa)</td>
<td>4.38</td>
<td>5.07 ksi (35 MPa)</td>
</tr>
<tr>
<td>$X'$</td>
<td>11.4 ksi (79 MPa)</td>
<td>3.74</td>
<td>3.33 ksi (23 MPa)</td>
</tr>
</tbody>
</table>

Note: These curves may be represented mathematically as

$$N = 2 \times 10^6 \left( \frac{\Delta_a}{\Delta_{\text{ref}}} \right)^m$$

where $N$ is the permissible number of cycles for applied cyclic stress range $\Delta_a$, with $\Delta_{\text{ref}}$, and $m$ as listed below.
8.4. Spectral Fatigue Analysis

Unstiffened Overlapping Joints

SCFs will be derived from the Efthymiou’s formulae. However, overlapping joints shall be avoided wherever possible.

iv) Internally Stiffened Joints

The SCF’s for internally stiffened joints will be calculated based on the Lloyd’s Register formulae.

Cone Transitions

The geometrical SCF’s for conical transitions will be determined as below:

$$SCF_{geom} = \frac{(fa + fb + fb1)}{(fa + fb)}$$

where:

$fa =$ Axial stress  $fb =$ Bending stress  $fb1 =$ Secondary bending stress as defined in API RP2A Section 2.5.

Fabrications Misalignments / Thickness Transitions

The Fabrication Specification will require that all plate misalignments in excess of 10% of the minimum thickness of the plates being joined must be profiled to a slope of 1:4 and that smaller misalignments will be joined with a tapering weld. Thickness transitions will be effected with a 1:4 taper on the thicker plate.

Minimum Stress Concentration Factors

In general, for tubular node joints, the minimum axial and bending SCF for all stubs and all chords is taken as 2.0. SCFs less than 2.0 may be used only for fully backed-up leg nodes when justified by finite element or acrylic model test results. Similarly, the minimum SCF for stiffened joints is taken as 2.5 unless otherwise justified.

8.4.11 Foundation Linearisation

The fatigue analysis involves series of structural analysis for each direction and each wave set. It will be very time consuming if every analysis has to be carried out with pile/soil
interaction. Further, the modal analysis cannot be carried out with pile/soil interaction. Hence the nonlinear foundation shall be linearised and a stiffness matrix that corresponds to six degrees of freedom at the pile head shall be used for the analyses. But to generate this spring stiffness, a representative wave height and direction shall be selected that represents the fatigue seastate.

The foundation spring stiffness will be derived using loads corresponding to an equivalent wave height, $H_e$. From the wave scatter diagram, it is possible to derive a fatigue damage scatter diagram on the basis of the following:

$$D_i = H_{si}^{bm} \frac{P_i}{T_{zi}}$$  \hspace{1cm} (8.7)

where

- $D_i$ = Damage of the $i_{th}$ sea-state
- $m$ = Slope of the S-N curve
- $b$ = Slope of log-linear wave height versus stress (Assumed = 1.80)
- $H_{si}$ = Significant wave height of the $i_{th}$ sea-state
- $T_{zi}$ = Zero up-crossing period of the $i_{th}$ sea-state
- $P_i$ = Probability of occurrence of the $i_{th}$ sea-state

The above assumes a log-linear wave height versus stress relationship.

After constructing the fatigue damage scatter diagram using the above formula, its centroid will be determined using the following equations to identify the sea-state causing the most damage:

$$\bar{H}_s = \frac{\sum D_i H_{si}}{D_i}$$  \hspace{1cm} (8.8)

$$\bar{T}_z = \frac{\sum D_i T_{zi}}{D_i}$$  \hspace{1cm} (8.9)

The deterministic design wave $(H_d, T_d)$ which represents the seastate at the centre of the damage scatter diagram is usually taken to be:

$$H_d = 1.86 \bar{H}_s$$  \hspace{1cm} (8.10)

$$T_d = 1.27 \bar{T}_z$$  \hspace{1cm} (8.11)
Chapter 9

Ship Impact Analysis

9.1 Impact Vessels

In an offshore development, often service boats and supply vessels have to serve the offshore operation. During their trips, due to harsh weather conditions, it may some time drift and hit the the jacket legs or braces. These vessels during their normal approach to the platform may arrive in with normal operating speed or may arrive at accidental speed depending on the weather conditions at the time of arrival. API RP2A specifies a operating a speed of 0.5 m/sec and accidental speed of 2 m/sec.

The jacket legs and braces in the splash zone shall be designed of such loads to avoid premature failure and collapse of the platform. Where such impacts are not allowed, a properly designed boat impact guard (sacrificial) shall be provided. For example, the risers located outside the jacket perimeter shall be protected with riser guard or riser protector and this kind of riser guard shall be located sufficiently away (at least a 1m) so that during vessel impact, risers does not experience large deflection.

The purpose of the boat impact analysis is:

- **Normal Impact** - To ensure the adequacy of the jacket leg and brace members in the splash zone such that they can absorb the energy imparted by a design vessel traveling at normal operating velocity.

- **Accidental Impact** To ensure the adequacy of the jacket leg and brace members in splash zone such that they can absorb the energy imparted vessel traveling at accidental velocity.

- **Post Impact Strength** - To ensure the compliance of the damaged platform for operating (1-year wave) design requirements after the boat impact.
9.2 Principle

The general methodology for ship impact analysis involves the following three primary steps:

- **Impact Local** Impact analysis to estimate the damage to the members which are directly hit by the ship and the impact force.

- **Impact Global Analysis** - Static analysis to ensure that the jacket will be able to withstand the impact force safely. In this analysis the member and joint stresses are allowed up to yield limit as this force is temporary.

- **Post Impact Analysis** Static analysis to verify if the damaged platform can sustain the 1 year operating environmental loads.

9.3 Method of Computing Impact Energy Dissipation

As the boat hits any member, equal and opposite forces are applied to the member and to the boat. These forces cause the following effects:

- **Local denting** of the member under the point of impact, which causes a local reduction in the effective cross-sectional area and section modulus of the member. The relationship between dent depth and lateral forces causing the dent is given by the DNV curve (Furnes and Amdahl) for knife-edge contact.

- **Bending of the member**, initially elastic and subsequently, elasto-plastic until a plastic mechanism form.

- **Denting of the boat** - The force/indentation curves contained in DnV TN A202 document is used for this purpose.

- **Global deformation** - Elastic deformation of the rest of the structure.

9.4 Energy dissipation by member

The total energy dissipation is computed in four stages as described below by iterating on the dent depth.

- **Stage 1 : Elastic Beam Bending** - Stage 1 of the energy absorption process covers the period from the moment of impact until the section starts yielding due to the axial
load and moments at the dented section. The impact is conservatively assumed to occur at midspan of the member. For a given dent depth, X, the lateral force, \( P_d \), causing the dent depth is calculated in accordance with the DNV curve for \( B/D = 0 \), where, \( B \) is the central length of the dent and \( D \) is the mean diameter of the member. The equation given below approximates the DNV curve for \( B/D = 0 \):

\[
P_d = 15m_p(D/t)(X/R)
\]  

(9.1)

where

- \( m_p \) = is the plastic moment capacity of tube wall equal to \( F_{yt}24 \)
- \( D \) = is the mean diameter of the tubular member
- \( R \) = is the mean radius of the tubular member
- \( t \) = is the thickness of the tubular member
- \( X \) = is the dent depth

The energy absorbed, \( E_d \), by a dent of given depth, \( X \), is found by integration of the force over the dent and is given as,

\[
E_d = 14.14m_p\frac{X^{3/2}}{t}
\]  

(9.2)

The lateral force, \( P_d \), causes a change in the bending moment distribution of the member. The moments at the ends and at the middle depend on the original moments and the rotational stiffness of the ends of the member. The energy absorbed by the bending deformation is given by,

\[
E_b = 0.5P_d\delta
\]  

(9.3)

Where, \( \delta \) is the lateral displacement of the member centre line. The other energy absorbing components are energy due to the longitudinal strain, global structural transitional energy and ship distortion energy. Stage 1 of the energy absorption mechanism ends when the sum of the axial and bending stresses at the centre of the tube is equal to the plate yields stress. This point is found by iterating on the dent depth.

- **Stage 2 : Elasto-Plastic Beam Bending** -

The behaviour of the member in Stage 2 is similar to that in Stage 1. Stage 2 ends at the formation of full plasticity at the dented section. By iterating on the depth of dent until the formation of first plastic hinge, the end of Stage 2 is found. The energy associated with the formation of the first plastic hinge is calculated in the same way as in Stage 1.
9.5 Energy Dissipation by Overall jacket deflection

- **Stage 3 : After Formation of First Hinge**
  
  This phase of energy absorption process continues until a 3-hinge mechanism is formed. For a given dent depth, the lateral force is found. The deflection at the centre of the beam is calculated assuming that the dent force is carried by the two cantilevers. If the energy absorbed at the end of Stage 3 is less than the impact energy of the ship, then the absorption mechanism enters Stage 4.

- **Stage 4 : After Formation of Three-Hinge Mechanism**
  
  At this stage, it is assumed that no further dent growth occurs and the dent has absorbed all the energy it is capable of. The energy absorption at this stage is due to the triangulation effect of longitudinal strain, structure distortion and ship deformation energy. Rupture of the brace being impacted and consequently penetration of the boat within the jacket shall be prevented by limited the tension strain to 10%. The displacement of the braces on gridline 1 shall be limited to prevent damage of the conductees.

  All the computations involved in the above four stages of energy absorption process are incorporated in the IMPACT program. Starting from Stage 1, the energy computations are continued over the four stages till the full ship impact energy is absorbed.

### 9.5 Energy Dissipation by Overall jacket deflection

Energy absorption by global deflection of the structure shall be included in the calculation of total energy of absorption.

### 9.6 Structural Strength During Impact

The strength of jacket immediately after impact is a static inplace analysis without any environmental loads. But the impact load at the point of impact shall be considered and the global strength of the jacket including piles shall be evaluated.

### 9.7 Push Over Analysis

Post impact analysis can be classified in to following three categories.

- Post Impact Strength Analysis
• Post Impact Redundancy Analysis

Post impact strength analysis is carried out to check the structure against 1 year environmental loads with all gravity loads. This is to ensure the structure is able to function as normal for some period of time so that any repair needs to be carried out due to the impact. However, at this stage, the structure need not be designed for 100 year storm loads. Obviously, the structure is checked for stresses against allowable stresses as per API RP 2A.

Push Over analysis is carried out to check the global integrity in terms of collapse behaviour.

The analysis will be performed using the full non-linear structural model described in Section 3.8. The finite element program ABAQUS 4.9 (Ref 3.0-2) will be used to carry out the analyses. ABAQUS uses full Newton-Raphson iteration to determine the non-linear response of the platform. Geometric non-linearity will be included and the load-displacement curves will be obtained using the automatic increment options, based on the modified Riks method (see Ref 3.0-2).

The analyses will be conducted by first applying the dead loads and the buoyancy loads, and then incrementally increasing a concentrated force at the point of impact until either instability in the model is achieved or the resisting strain energy of the structure greatly exceeds the specified impact energy.

Energies associated with the two non-linear springs (representing the energy absorptions due to local deformation of the vessel and jacket), the elements used to model the impacted member, and the remainder of the structure will be computed at each incremental step of the analysis which will effectively produce a history of the energy distribution as a function of the impact force.

The energy response from the structure will be used in an energy balance relationship to proportion the levels of energy dissipated by the substructure and the vessel. The impact load will be determined at which the sum of the strain energies of the structure and vessel equals the kinetic energy of the supply ship. Rebound (unloading) analysis will be carried out as this impact load is removed; this will give the final displaced shape of the structure which could be used as the initial point for the post-impact pushover analysis, if required.

The ultimate axial and bending moment capacity of the jacket tubular joints will be checked to see that they have sufficient capacity to carry the maximum brace loads from the abnormal environmental condition and the accidental ship impact conditions.

The lower bound ultimate capacity formula for axial load, inplace and out-of-plane bending moment in API RP2A (Ref 3.1-2) will be adopted for the joint strength checks. The ultimate capacity formulas are the same as the API RP2A nominal load formula except that the factor of safety of 1.7 is removed.