DEVELOPMENT OF A METHODOLOGY FOR THE DESIGN OF AN OFFSHORE OIL PRODUCTION PLATFORM ON THE ALASKAN ARCTIC CONTINENTAL SHELF

Final Report
California Sea Grant Project R-OT-11

VOLUME I
(of 2)

Sanjay Sakhuja
Author

Prof. Ben C. Gerwick, Jr.
Principal Investigator

University of California, Berkeley
December, 1985
Development of a Methodology for the Design of an Offshore Oil Production Platform on the Alaskan Arctic Continental Shelf

By

Sanjay Sakhija

© 1985
Acknowledgement

This report is based on the research conducted for the California Sea Grant Project 17-OT-N. This research project was solely funded by the California Sea Grant program.

This work is a result of research sponsored in part by NOAA, National Sea Grant College Program, Department of Commerce, under grant number NA80AA-D-00120, project number R/OT-11, through the California Sea Grant College Program, and in part by the California State Resources Agency. The U.S. Government is authorized to reproduce and distribute for governmental purposes.
Development of a Methodology for the Design of an Offshore Oil Production Platform on the Alaskan Arctic Continental Shelf

By
Sanjay Sakhuja

Abstract

Hydrocarbon extraction from the oceans has been going on for the past three decades. Beginning in shallow water regions of the coastal areas, the offshore oil exploration and production process has moved successfully into the more hostile deep ocean area. More recently, the Arctic Ocean has been the frontier in the oil exploration field. Prospects of finding large reserves of gas and oil in the Beaufort Sea have initiated large exploration projects. The exploration in a few instances has led to discovery of commercially viable oil and gas reservoirs. Present lease sales in the Alaskan sector of the Beaufort Sea have opened for exploration zones in water depths as great as than 300 feet. These areas lie in the Stamukhi shear zones of the polar ice pack, where the ice action on the structures is highly dynamic. Thus, if oil is to be explored and produced safely from these areas, specially designed platforms will have to be utilized. This study presents a methodology for the design of production structures to be deployed in the Southern Beaufort Sea.

The methodology as presented is non-site specific and is intended to provide the designer as well as the evaluator with an aid to carry out the design (or evaluation) in
a logical manner. The emphasis in this study has been on technical rather than socio-economic and environmental aspects of design.

The aspects of design that have been addressed in detail in this study are as follows:

- Ice-Structure Interaction
- Structural Design
- Foundation Design
- Hydrodynamic Design Consideration
- Oceanographic Design Consideration
- Constructibility
- Risk and Reliability Aspects of Production Structure Design

Ben C. Gerwick, Jr.
Dissertation Committee Chairman
Berkeley, California
December 5, 1985
# Table of Contents

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>ABSTRACT</td>
<td></td>
</tr>
<tr>
<td>ACKNOWLEDGEMENT</td>
<td></td>
</tr>
<tr>
<td>LIST OF FIGURES</td>
<td></td>
</tr>
<tr>
<td>LIST OF TABLES</td>
<td></td>
</tr>
<tr>
<td>INTRODUCTION</td>
<td></td>
</tr>
<tr>
<td><strong>1. SEA ICE FEATURES</strong></td>
<td>1</td>
</tr>
<tr>
<td>1.1 Arctic Sea Ice</td>
<td>1</td>
</tr>
<tr>
<td>1.2 Pressure Ridges</td>
<td>2</td>
</tr>
<tr>
<td>1.3 Ice Floes</td>
<td>4</td>
</tr>
<tr>
<td>1.4 Ice Islands</td>
<td>4</td>
</tr>
<tr>
<td><strong>2. SEA ICE PROPERTIES</strong></td>
<td>10</td>
</tr>
<tr>
<td>2.1 Sea Ice Crystallography</td>
<td>10</td>
</tr>
<tr>
<td>2.2 Physical and Mechanical Properties of Sea Ice</td>
<td>12</td>
</tr>
<tr>
<td>2.2.1 Effect of Salinity</td>
<td>12</td>
</tr>
<tr>
<td>2.2.2 Effect of Temperature</td>
<td>13</td>
</tr>
<tr>
<td>2.2.3 Behavior of Ice Under Loading</td>
<td>14</td>
</tr>
<tr>
<td>2.2.3.1 Ductile Behavior</td>
<td>14</td>
</tr>
<tr>
<td>2.2.3.2 Brittle Behavior</td>
<td>16</td>
</tr>
<tr>
<td>2.2.3.3 Transition Zone</td>
<td>17</td>
</tr>
<tr>
<td>2.2.4 Strength of Ice</td>
<td>17</td>
</tr>
<tr>
<td>2.2.4.1 Compressive Strength</td>
<td>18</td>
</tr>
<tr>
<td>2.2.4.2 Tensile Strength</td>
<td>20</td>
</tr>
<tr>
<td>2.2.4.3 Flexure Strength</td>
<td>20</td>
</tr>
<tr>
<td>Section</td>
<td>Page</td>
</tr>
<tr>
<td>------------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>2.2.4.4 Shear Strength</td>
<td>22</td>
</tr>
<tr>
<td>2.2.4.5 Confined Strength of Sea Ice</td>
<td>22</td>
</tr>
<tr>
<td>2.2.5 Elastic Modulus of Sea Ice</td>
<td>23</td>
</tr>
<tr>
<td>2.2.6 Poisson’s Ratio</td>
<td>23</td>
</tr>
<tr>
<td>2.2.7 Friction Coefficient</td>
<td>24</td>
</tr>
<tr>
<td>2.2.8 Adfreeze Bond Strength</td>
<td>24</td>
</tr>
<tr>
<td>2.3 Relevance of Sea Ice Properties</td>
<td>25</td>
</tr>
<tr>
<td>3. LOADS DUE TO CRUSHING OF ICE</td>
<td>50</td>
</tr>
<tr>
<td>3.1 Analysis Technique</td>
<td>50</td>
</tr>
<tr>
<td>3.1.1 Strain Rate Effects</td>
<td>51</td>
</tr>
<tr>
<td>3.1.2 Indentation Factor</td>
<td>53</td>
</tr>
<tr>
<td>3.1.2.1 Indentation Factor for Granular Ice</td>
<td>53</td>
</tr>
<tr>
<td>3.1.2.2 Indentation Factor for Columnar Ice</td>
<td>55</td>
</tr>
<tr>
<td>3.1.3 Contact Factor</td>
<td>58</td>
</tr>
<tr>
<td>3.2 Meso Scale Look at Ice Behavior</td>
<td>60</td>
</tr>
<tr>
<td>4. LOADS ON STRUCTURES DUE TO RIDGES</td>
<td>71</td>
</tr>
<tr>
<td>4.1 Idealized Ridge Shape</td>
<td>71</td>
</tr>
<tr>
<td>4.2 Forces Due to Pressure Ridges</td>
<td>72</td>
</tr>
<tr>
<td>4.2.1 Flexure Failure</td>
<td>72</td>
</tr>
<tr>
<td>4.2.2 Effect of Partially Consolidated Ridge</td>
<td>75</td>
</tr>
<tr>
<td>4.2.3 Ridge Shearing</td>
<td>76</td>
</tr>
<tr>
<td>4.2.3.1 Multiyear Consolidated Ridge</td>
<td>76</td>
</tr>
<tr>
<td>4.2.3.2 Failure Mechanism</td>
<td>77</td>
</tr>
<tr>
<td>4.2.4 Ridge Crushing</td>
<td>79</td>
</tr>
<tr>
<td>4.2.4.1 First Year Unconsolidated Ridge</td>
<td>79</td>
</tr>
<tr>
<td>4.3 Comparison of Various Models</td>
<td>82</td>
</tr>
<tr>
<td>5. ICE FORCES ON SLOPING STRUCTURES</td>
<td>93</td>
</tr>
<tr>
<td>5.1 Introduction</td>
<td>93</td>
</tr>
<tr>
<td>Section</td>
<td>Page</td>
</tr>
<tr>
<td>---------------------------------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>5.2 Two Dimensional Analysis</td>
<td>93</td>
</tr>
<tr>
<td>5.3 Three Dimensional Analysis</td>
<td>98</td>
</tr>
<tr>
<td>5.4 Empirical Models</td>
<td>100</td>
</tr>
<tr>
<td>5.5 Comparison of Various Models</td>
<td>101</td>
</tr>
<tr>
<td>5.5.1 Narrow Structures</td>
<td>101</td>
</tr>
<tr>
<td>5.5.2 Wide Structures</td>
<td>101</td>
</tr>
<tr>
<td>5.6 Ridge Loads</td>
<td>102</td>
</tr>
<tr>
<td>5.6.1 Adfreeze Forces</td>
<td>104</td>
</tr>
<tr>
<td>5.7 Downward Breaking Conical Structures</td>
<td>105</td>
</tr>
<tr>
<td>6. Limited Driving Force Concept</td>
<td>114</td>
</tr>
<tr>
<td>6.1 Introduction</td>
<td>114</td>
</tr>
<tr>
<td>6.2 Far Field Force</td>
<td>115</td>
</tr>
<tr>
<td>6.3 Wind and Current Drag Forces</td>
<td>117</td>
</tr>
<tr>
<td>7. ICE FORCES DUE TO ICE FLOE IMPACT</td>
<td>122</td>
</tr>
<tr>
<td>7.1 Introduction</td>
<td>122</td>
</tr>
<tr>
<td>7.2 Ice Floe Impact Scenario</td>
<td>123</td>
</tr>
<tr>
<td>7.2.1 Concentric Impact</td>
<td>123</td>
</tr>
<tr>
<td>7.2.2 Eccentric Impact</td>
<td>124</td>
</tr>
<tr>
<td>7.3 Quasi-Static Analysis for Concentric Impact</td>
<td>126</td>
</tr>
<tr>
<td>7.4 Eccentric Impact</td>
<td>128</td>
</tr>
<tr>
<td>8. LOCAL ICE PRESSURES</td>
<td>143</td>
</tr>
<tr>
<td>8.1 Introduction</td>
<td>143</td>
</tr>
<tr>
<td>8.2 Method for Computing Local Ice Pressure</td>
<td>144</td>
</tr>
<tr>
<td>8.3 Local Ice Pressure for Sloping Sided Structures</td>
<td>145</td>
</tr>
<tr>
<td>8.4 Local Ice Pressure Curves</td>
<td>146</td>
</tr>
<tr>
<td>8.5 Sequential Procedure for Local Ice Pressure</td>
<td>147</td>
</tr>
<tr>
<td>9. EXTERNAL WALLS &amp; INTERNAL FRAMING SYSTEMS</td>
<td>152</td>
</tr>
<tr>
<td>9.1 Introduction</td>
<td>152</td>
</tr>
</tbody>
</table>
9.2 Internal Framing System ................................................................. 154
  9.2.1 Design Premise for Internal Framing System ......................... 154
  9.2.2 Behavior Under Global Loads ............................................. 155
  9.2.3 Global Analysis ................................................................. 155
  9.2.4 Ice Loads for Global Analysis ............................................. 158

9.3 Design Premise for Peripheral Ice Resisting Walls ...................... 159
  9.3.1 Behavior of Ice Wall Under Concentrated Loads .................... 163
    9.3.1.1 Beam and Arch Action .............................................. 163
    9.3.1.2 Catenary Action .................................................. 169
  9.3.2 Current Design Practices .................................................. 172
    9.3.2.1 Working Stress Method .......................................... 172
    9.3.2.2 Ultimate Strength Method ...................................... 173
    9.3.2.3 Probabilistic Design ............................................ 173
    9.3.2.4 Limit State Design ............................................ 174
  9.3.3 Local Ice Pressures .......................................................... 177
    9.3.3.1 Uniform Local Ice Pressures .................................... 177
    9.3.3.2 Local Ice Pressures for Sloping Sided Structures .......... 178
    9.3.3.4 Hard Spots with Uniform Pressure ............................. 178
    9.3.3.5 Wave Accelerated Small Ice Features ......................... 178
    9.3.3.6 Other Practices and Codes for Ice Pressures .............. 179
  9.4 Recommended Design Procedure for Ice Walls .......................... 180

9.5 Thermal Considerations .......................................................... 182

10. WAVE CONSIDERATIONS .............................................................. 208
    10.1 Introduction ................................................................. 208
    10.2 Wave Run-up .................................................................... 209
      10.2.1 Type A Structures (Gravel Islands) ......................... 210
      10.2.2 Type B Structures (Caisson Type) ............................. 213
    10.3 Wave & Current Induced Erosion ....................................... 216
10.4 Wave Forces

10.4.1 Procedure for Evaluating Wave Forces on Arctic Production Platforms

10.4.2 Numerical Methods

10.4.3 Design Wave Method

10.4.4 Spectral Approach

11. FOUNDATION DESIGN

11.1 Introduction

11.2 Geologic Hazards

11.2.1 Permafrost

11.2.2 Gas Hydrates

11.2.3 Ice Gouging

11.2.4 Sub-Marine Mud Slides

11.3 Foundation Analysis and Design

11.3.1 Stability Analysis

11.3.1.1 Sliding Failure

11.3.1.2 Bearing Failure

11.3.1.3 Overturning

11.3.1.4 Combined Shear and Bearing on Foundation

11.3.2 Deformation Analysis

11.3.3 Slope Stability Analysis Procedure

11.3.4 Skirt Design

11.3.5 Contact Pressures

11.3.6 Cyclic Loading Effects of Foundations

11.4 Methods for Improving Soil Stability

11.4.1 Dredging and Refilling

11.4.2 Soil Strengthening by Wick Drains

11.4.3 Artificial Soil Freezing
11.4.4 Deep In-situ Mixing of Soil .................................................. 272
11.4.5 Grouting ............................................................................. 272
11.4.6 Penetrating the Structure to the Competent Soil ................. 273
11.4.7 Mat and Spud-Pile System ................................................... 273
11.4.8 Sand and Stone Column ....................................................... 274
11.4.9 Surcharge Around Perimeter .............................................. 274

12. MARINE OPERATIONS ................................................................ 302

12.1 Introduction ........................................................................... 306

12.2 Hydrodynamic Stability Requirements .................................. 307
  12.2.1 Det Norske Veritas ............................................................. 307
  12.2.2 American Bureau of Shipping .......................................... 308
  12.2.3 FIP ...................................................................................... 309
  12.2.4 United States Geologic Survey ........................................ 310

12.3 Definitions of Naval Terms .................................................... 311
  12.3.1 Draft .................................................................................. 311
  12.3.2 Displacement .................................................................... 311
  12.3.3 Center of Floatation ........................................................... 311
  12.3.4 Center of Gravity ............................................................... 312
  12.3.5 Center of Buoyancy ............................................................ 312
  12.3.6 Tons per Inch ................................................................... 312
  12.3.7 Metacentric Height ............................................................. 312
  12.3.8 Free Surface Effects .......................................................... 313
  12.3.9 Downflooding Angle .......................................................... 314
  12.3.14 Wind Heeling Moment .................................................... 314
  12.3.11 Statical Righting Moment ............................................... 315

12.4 Motion Response Evaluation ................................................ 316
  12.4.1 Uncoupled Motion Response ............................................ 320
12.5 Tow Operation ....................................................................... 322
14.2.2.5 Stepped Pyramid Concept .................................................. 391

15. STRUCTURAL RELIABILITY AND RELIABILITY BASED DESIGN .......... 422

15.1 Introduction ........................................................................... 422
15.2 Basic Principles of Reliability ................................................. 425
  15.2.1 Structural Reliability ......................................................... 425
  15.2.2 MVFOSM Method .............................................................. 426
  15.2.3 FOSM Method ................................................................. 428
  15.2.4 FOMD & FOFD Methods ................................................... 429
15.3 Target Reliability .................................................................... 429
15.4 Reliability Based Design of Ice Wall ......................................... 430
  15.4.1 Statement of Problem ....................................................... 430
  15.4.2 Problem Formulation ....................................................... 432
  15.4.3 Ice Model ................................................................. 433
  15.4.4 Performance Function ..................................................... 434
  15.4.5 Random Variable Properties ............................................ 435
15.5 Current Format of Codes ....................................................... 437
15.6 Conclusions .......................................................................... 441

BIBLIOGRAPHY .......................................................................... 452
List of Figures

Figure 1.1 Ice Features in the Beaufort Sea ......................................................... 5
Figure 1.2 Ice Zones in the Beaufort Sea ............................................................... 6
Figure 1.3 Seasonal Development of Ice Zonation .................................................. 7
Figure 1.4 Multi-year pressure ridge model ........................................................... 8
Figure 1.5 Ice condition map of Arctic Ocean ......................................................... 9
Figure 2.1 Tetrahedral arrangement of oxygen atoms in ice .................................. 27
Figure 2.2 Molecular structure of ice ................................................................. 27
Figure 2.3 Schematic drawing showing various layers of sea ice ......................... 28
Figure 2.4 Section through sea ice sheet ............................................................... 29
Figure 2.5(a) A cross polaroid photograph of sea ice sheet ................................. 30
Figure 2.5(b) Crystal structure of landfast ice at Notsuke-Odaita ......................... 31
Figure 2.6 Brine channels in sea ice ................................................................. 32
Figure 2.7 Salinity distribution through the thickness of ice sheet ....................... 32
Figure 2.8 Universal curve for uniaxial crushing and indentation S2 ice ............... 33
Figure 2.9 Unconfined compressive strength as a function of strain rate for different types of fresh water ice ................................................................. 34
Figure 2.10 Compressive strength of Baltic Sea ice as a function of strain rate, ice temperature, and orientation of the force .................................................. 35
Figure 2.11 Compressive strength of granular ice as a function of strain rate .......... 35
Figure 2.12 Compressive strength of unoriented columnar sea ice as a function of strain rate ................................................................. 36
Figure 2.13 Compressive strength of oriented columnar sea ice as a function of strain rate ................................................................. 37
Figure 2.14 Compressive strength of oriented columnar sea ice as a function of strain rate ................................................................. 38
Figure 2.15 σR from compressive strength tests vs. brine volume ......................... 38
Figure 2.16 Temperature dependence of compressive strength in Transition zone ........................................................................................................... 39
Figure 2.17 Compressive strength of Baltic Sea ice and fresh water ice as a function of temperature ................................................................. 39
Figure 2.18 Tensile strength of various types of polycrystalline ice as a function of temperature ................................................................. 40
Figure 2.19 Tensile strength of sea ice as a function of brine volume .................... 41
Figure 2.20 Flexure strength of sea ice as a function of brine volume, from cantilever tests ................................................................. 41
Figure 2.21 Flexure strength measured by beam tests
Figure 2.22  Shear strength as a function of square root of brine volume...........................................42
Figure 2.23  Confined compressive strength of sea ice.................................................................43
Figure 2.24  Triaxial strength of fine grained polycrystalline ice.....................................................44
Figure 2.25  Elastic Modulus of sea ice as determined by small specimen tests vs. brine volume........45
Figure 2.26  Elastic Modulus of sea ice as determined by seismic techniques vs. brine volume.....45
Figure 2.27  Apparent elastic modulus as a function of brine volume..........................................46
Figure 2.28  Elastic modulus for fresh water ice as a function of strain rate....................................47
Figure 2.29  Static and kinematic friction coefficients for ice..........................................................48
Figure 2.30  Adfreeze bond strength for sea ice.................................................................................49
Figure 3.1    Physical model of indentation process.................................................................61
Figure 3.2    Indentation factors.......................................................................................................62
Figure 3.3    Assumed slip planes for crushing model.................................................................63
Figure 3.4    Indentation factors.......................................................................................................64
Figure 3.5    Parabolic yield criteria for ice......................................................................................65
Figure 3.6    Indentation factors.......................................................................................................66
Figure 3.7    Contact factor as a function of strain rate.................................................................66
Figure 3.8    Contact factors............................................................................................................67
Figure 3.9    Pressure distribution for cold and warm ice...............................................................68
Figure 3.10  Failure zones in crushing............................................................................................69
Figure 3.11  Meso-scale look at ice pressures.................................................................................70
Figure 4.1    Consolidated ridge model...........................................................................................84
Figure 4.2    Unconsolidated ridge model.......................................................................................84
Figure 4.3    In-plane flexure failure model for ice ridge.................................................................85
Figure 4.4    Failure mechanism for ice ridge................................................................................86
Figure 4.5    Ridge flexure failure - initial crack formation.............................................................87
Figure 4.6    Ridge flexure failure - hinge crack formation..............................................................87
Figure 4.7    Anisotropic yield function for sea ice.........................................................................88
Figure 4.8    Consolidated ridge shearing mechanism.................................................................89
Figure 4.9    First year ridge crushing mechanism.........................................................................90
Figure 4.10  Consolidated Ridge force............................................................................................91
Figure 4.11  Rubble Pile force.........................................................................................................91
Figure 4.12  Unconsolidated Ridge bearing.....................................................................................92
Figure 5.1    Initial interaction between ice and sloping structures.................................................108
Figure 5.2    Interaction between ice and sloping structure.............................................................109
Figure 5.3    Coefficient C1 as a function of slope angle and
friction coefficient

Figure 5.4 Coefficient C2 as a function of slope angle and friction coefficient

Figure 5.5 A three dimensional representation of ice structure interaction

Figure 5.6 Ice force coefficients for plastic analysis

Figure 5.7 Formation of initial crack and final hinge crack during ridge structure interaction

Figure 6.1 Concept of limit force and limit stress

Figure 6.2 Stages in limit force interaction

Figure 7.1 Schematic model of the system including the foundation, the structure and the ice feature

Figure 7.2 An example of ice force time history

Figure 7.3 Summer collision ice loading from Tarsuit Island

Figure 7.4 Winter ice loading form Tarsuit Loading

Figure 7.5 Two cases of ice floe impact

Figure 7.6 Concentric impact scenario

Figure 7.7 Eccentric impact scenario

Figure 7.8 Flow chart for concentric impact analysis algorithm

Figure 7.9 An example calculation for concentric impact

Figure 7.10 Plot showing time history of impact force

Figure 7.11 Schematic diagram of non-sliding contact of ship with fender in berthing operation

Figure 7.12 Example for force vs. penetration and energy vs. penetration

Figure 8.1 Local ice pressure curve

Figure 8.2 Local ice pressure curve

Figure 8.3 Typical Ice Wall system

Figure 9.1 A typical load path

Figure 9.2 Pressure versus area curves of equal probability

Figure 9.3 Portion of structure carrying ice load

Figure 9.4 Ovalling of structure under global ice loads

Figure 9.5 Elements of global analysis

Figure 9.6 Definition of deflection ductility

Figure 9.7 Reinforced concrete member for ice wall

Figure 9.8 Arch action in an idealized beam

Figure 9.9 Inclined compression stress trajectories in a deep beam

Figure 9.10 Triaxial stress model for concrete

Figure 9.11 Idealized beam with inclined shear crack

Figure 9.12 Constitutive model of concrete under various levels
of confinement

Figure 9.13 Various stages of a beam as it deflects under load

Figure 9.14 Shear capacity for varying percentages of shear reinforcement

Figure 9.15 Typical ice wall configuration for reinforced concrete and hybrid steel-concrete construction

Figure 9.16 Local ice pressure curve

Figure 9.17 Critical area for punching shear

Figure 9.18 Some concepts for hard spot local pressure loading

Figure 9.19 Design procedure for ice wall

Figure 9.20 Typical winter temperature regime

Figure 9.21 Global thermal straining of a typical structure

Figure 10.1 Wave action around Tarsuit Island

Figure 10.2 Definition sketch, wave run-up and overtopping

Figure 10.3 Wave run-up on smooth impermeable slopes

Figure 10.4 Wave run-up on smooth impermeable slopes

Figure 10.5 Wave run-up on smooth impermeable slopes

Figure 10.6a Wave run-up on smooth impermeable slopes

Figure 10.6b Wave run-up on smooth impermeable slopes

Figure 10.7 Run-up correction for scale effects

Figure 10.8 Gravel island model studied for wave run-up at the University of California, Berkeley

Figure 10.9 Wave action observed in model tests

Figure 10.10 R/H versus \( \eta_{w}/H \) for nonbreaking waves

Figure 10.11 R/H versus \( \eta_{w}/H \) for nonbreaking waves

Figure 10.12 Definition of Clapotis wave on vertical walls

Figure 10.13 Three cases of reflection as defined by Wiegel

Figure 10.14 Oblique reflection of a solitary wave

Figure 10.15 Significant wave run-up on slope above a caisson for example problem

Figure 10.16 Predicted runup profiles for various \( k_{a} \) and \( L \)

Figure 10.17 Two-dimensional model BICOC

Figure 10.18 Erosion pattern observed in model tests conducted at the University of California, Berkeley

Figure 10.19 Variation of inertia coefficient and phase angle

Figure 10.20 \( f_{A} \) and \( f_{B} \) versus \( d/L \)

Figure 10.21 Wave force coefficients for a square body

Figure 10.22 Spectrum of overturning moments

Figure 11.1 Holocene marine sediment thickness on the
Figure 11.2  Shallow free gas concentration................................................. 280
Figure 11.3  Ice gouging............................................................................. 281
Figure 11.4  A major ice gouge mark.......................................................... 282
Figure 11.5  Some statistical data from USGS Ice Gouge
Investigation Program...................................................................... 283
Figure 11.6  Foundation design logic diagram............................................. 284
Figure 11.7  Potential modes of failure for production structures............... 285
Figure 11.8  The resistance to a deep seated sliding failure is
supplemented by passive resistance...................................................... 286
Figure 11.9  For a doughnut shaped structure the sliding failure mode........ 287
Figure 11.10 Effective foundation area for eccentric loading....................... 288
Figure 11.11 Suggested N values................................................................. 289
Figure 11.12 Relationship between foundation width, SPT value N,
and allowable bearing pressure............................................................... 290
Figure 11.13 Correction factor for SPT value N to allow for the effect
of overburden pressure........................................................................... 291
Figure 11.14 Criteria for foundation stability (cohesive soils)...................... 292
Figure 11.15 Criteria for foundation stability (granular soils)...................... 293
Figure 11.16 In the event of an eccentric impact the foundation
will be subjected to direct shear and moment........................................... 294
Figure 11.17 Variation of $I_z$ with $z$.............................................................. 295
Figure 11.18 Passive failure of soil between skirts...................................... 296
Figure 11.19 Structural implications of seating structure on
uneven seafloor....................................................................................... 297
Figure 11.20 Stability implication of seating structures on
uneven seafloor....................................................................................... 298
Figure 11.21 Local contact pressure due to uneven seafloor........................ 299
Figure 11.22 Field measurements of live loads shows the
cyclic nature of ice load.......................................................................... 300
Figure 11.23 Longterm variation in ice load................................................ 301
Figure 11.24 Reduction in material coefficient as a function of
increase in wave force............................................................................ 302
Figure 11.25 Mobilized maximum shear stress ratio with
current practice....................................................................................... 303
Figure 11.26 Wick drain system of improving soil stability......................... 304
Figure 11.27 Deep in-place mixing stabilization patterns for
deep-cement mixing and similar techniques.......................................... 305
Figure 11.28 Mix-in place columns made by jet grouting............................ 306
Figure 11.29 Schematic of jet eductor system............................................. 307
Figure 12.1  Logic diagram for marine considerations of production structure design.........................................................331
Figure 12.2  Stability curve for establishing stability under impulsive load............................................................................332
Figure 12.3  Relationship between centers of Buoyancy, Center of Gravity and Metacenter.................................................................333
Figure 12.4  Definition of positive and negative stability.................................334
Figure 12.5  Illustration showing effect of stability with liquid free surface..................................................................................335
Figure 12.6  Wind healing moment..........................................................................................336
Figure 12.7  The six degrees of freedom of a floating body..............................337
Figure 12.8  Added mass and damping coefficients..............................................338
Figure 12.9  Added mass and damping coefficients..............................................339
Figure 12.10  Added mass and damping coefficients...........................................340
Figure 12.11  Added mass and damping coefficients...........................................341
Figure 12.12  Added mass and damping coefficients...........................................342
Figure 12.13  Added mass and damping coefficients...........................................343
Figure 12.14  Added mass and damping coefficients...........................................344
Figure 12.15  Added mass and damping coefficients...........................................345
Figure 12.16  Added mass and damping coefficients...........................................346
Figure 12.17  Definition of terms for a cylindrical structure for use in figure 12.8 to 12.16.................................................................347
Figure 12.18  Suggested values of Scott-Wiegel spectrum parameters.................348
Figure 12.19  A typical spectral density function...................................................349
Figure 12.20  RAO and response spectra for heave.............................................350
Figure 12.21  Spectral density of heave acceleration........................................351
Figure 12.22  Spectral density function of heave................................................352
Figure 12.22  $R_D^H$ and $C_t$ curve for Barge A..................................................353
Figure 12.23  $R_D^H$ and $C_t$ curve for Barge B..................................................354
Figure 12.24  $R_D^H$ and $C_t$ curve for Barge C..................................................355
Figure 12.25  $R_D^H$ and $C_t$ curve for Barge D..................................................356
Figure 12.26  $R_D^H$ and $C_t$ curve for Barge E..................................................357
Figure 12.27  $R_D^H$ and $C_t$ curve for Barge F..................................................358
Figure 12.28  Residual resistance coefficient versus length-beam ratio for bare hull of barge...............................................................359
Figure 12.29  Residual resistance coefficient versus length-beam ratio for bare hull of barge...............................................................360
Figure 12.30  Residual resistance coefficient versus length-beam ratio for bare hull of barge...............................................................361
<table>
<thead>
<tr>
<th>Figure 13.1</th>
<th>Simplified process flow diagram for oil production</th>
<th>381</th>
</tr>
</thead>
<tbody>
<tr>
<td>Figure 13.2</td>
<td>Block diagram for topside facilities and equipment</td>
<td>382</td>
</tr>
<tr>
<td>Figure 13.3</td>
<td>Topside facilities breakdown on component basis</td>
<td>383</td>
</tr>
<tr>
<td>Figure 14.1</td>
<td>Typical well spacing pattern for a gravity structure</td>
<td>393</td>
</tr>
<tr>
<td>Figure 14.2</td>
<td>Ice feature interacting with a berm under a caisson retained island</td>
<td>394</td>
</tr>
<tr>
<td>Figure 14.3</td>
<td>Typical scenario for ice floe structure interaction</td>
<td>395</td>
</tr>
<tr>
<td>Figure 14.4</td>
<td>Ice jamming in multi-tower concept</td>
<td>396</td>
</tr>
<tr>
<td>Figure 15.1</td>
<td>Geometric interpretation of ice failure probability</td>
<td>446</td>
</tr>
<tr>
<td>Figure 15.2</td>
<td>Probability hill for joint distribution of load and resistance</td>
<td>446</td>
</tr>
<tr>
<td>Figure 15.3</td>
<td>Probability of failure</td>
<td>447</td>
</tr>
<tr>
<td>Figure 15.4</td>
<td>Probability density function Z</td>
<td>448</td>
</tr>
<tr>
<td>Figure 15.5</td>
<td>Probability density function of standard variate U</td>
<td>448</td>
</tr>
<tr>
<td>Figure 15.6</td>
<td>FOSM method for linear performance function and uncorrelated basic variables</td>
<td>449</td>
</tr>
<tr>
<td>Figure 15.7</td>
<td>A typical ice wall loading scenario</td>
<td>450</td>
</tr>
<tr>
<td>Figure 15.8</td>
<td>Definition of terms used in ice wall loading model</td>
<td>451</td>
</tr>
</tbody>
</table>
List of Tables

Table 5.1  Comparison of models for computing ice forces on wide conical structures .............................................. 106
Table 5.2  Comparison of models for computing ice forces on narrow conical structures ......................................... 107
Table 6.1  Ridge building force .................................................................................................................. 119
Table 9.1  Loading cases for ice wall ........................................................................................................... 183
Table 9.2  Summary of load factors ............................................................................................................. 184
Table 9.3  Capacity reduction factors & material coefficients ........................................................................... 185
Table 10.1 Site conditions for example island .............................................................................................. 225
Table 10.2 Wave run-up on slope above caisson for the example ................................................................. 226
Table 10.3 Wave loads computed by design spectrum approach and design wave approach .......................... 227
Table 11.1 Relationship between cone resistance and internal angle of friction ......................................... 275
Table 11.2 Numerical values of coefficients kp and kf for sand and clay ......................................................... 276
Table 11.3 Typical seabed surface conditions in area of active ice gouging .................................................... 277
Table 12.1 North Pacific design criteria for tow ............................................................................................ 330
Table 13.1 Main data for North Sea platforms .............................................................................................. 378
Table 13.2 Comparison of estimated and final costs of Ekofisk platform ...................................................... 379
Table 13.3 Topside data for some North Sea platforms ..................................................................................... 380
Table 13.4 Man hour data for fabrication & hook-up ..................................................................................... 380
Table 15.1 Summary of current reliability indices .......................................................................................... 443
Table 15.2 Basic statistical data for concrete and steel .................................................................................... 444
Table 15.3 Random parameter values at failure point ..................................................................................... 445
INTRODUCTION

Hydrocarbon fuel extraction from the oceans has been going on for the past three decades. Beginning in shallow water regions of the coastal areas, the offshore oil exploration and production process has moved successfully into the more hostile deep ocean areas, such as, the Gulf of Mexico, the North Sea, and the North West Coast of Australia. More recently the Arctic Ocean has been the frontier in the oil exploration field. Prospects of finding large, or perhaps giant reserves of gas and oil in the Beaufort Sea has initiated large exploration projects. The exploration in some instances has led to the discovery of viable oil and gas reservoirs. Future lease sales in the Alaskan sector of the offshore continental shelf areas will open for exploration zones in water depths greater than 300 feet. These areas lie in the dynamic Stamukhi shear zone of the polar ice pack, where the ice action on a structure can be very aggressive. Thus, if oil is to be explored and produced safely from these areas, specially designed platforms will have to be utilized. The challenges of the inhospitable Arctic environment are numerous and designs demanding.

This study presents a methodology for the design of production structures to be deployed in the Southern Beaufort Sea. The methodology as presented is non-site specific and is intended to provide the designer as well as the evaluator with an aid to carry out the design (or evaluation) in a logical manner.
Design Basis

The issues facing the designer of an offshore oil production platform are three fold:

(1) Socio-Economic Issues
(2) Environmental Issues, and
(3) Technical Issues.

The first two, though of importance, have not been addressed in this study; this study primarily addresses the technical issues. Logic diagrams A & B present briefly the Socio-Economic and Environmental issues and concerns. The technical issues as presented in this document are logically represented in Logic Diagram C. A brief outline of topics discussed in this study is presented below.

Production Structure Requirements

The Arctic environment is highlighted by the severe ice conditions – dynamic interaction with icefloe...
the Fatigue Limit State Design. To avoid catastrophic failure the structure’s performance should also comply with the requirements of *Progressive Collapse Limit State*.

The design requirements of a production platform are as follows:

**Resistance to Global Ice Loads**

The production structure in the Beaufort Sea will be designed to withstand forces from ice features such as:

a. Summer Ice Floe Impact  
b. Winter Embedded Ridges  
c. Ice Island Fragments

The selection of design feature is based on an appropriate return period of the event to ensure an acceptable encounter frequency and risk. The global ice loads will govern the overall sizing of the structure. Logic diagrams D & E show the procedure for calculating ice forces on vertical and conical structures, respectively. Chapters 1 through 7 discuss in detail the various method for estimating the ice forces.

**Resistance to Local Ice Pressure**

The ability of the structure to withstand the global ice pressures is limited by the ability of the structure to efficiently transfer load through structural load paths to the foundation. Thus the ice load has to be carried by individual member, such as the ice wall, bracing walls, floor slabs etc. and eventually transferred to the foundation elements. The design of these components of the structure is governed by the local ice pressures, which are much higher than the pressures corresponding to the maximum global ice forces (chapter 8 discusses the local ice pressure issue). The local ice pressures from hard spots
can produce high loads over relatively small areas for small time duration during ice-structure interaction. To resist these high pressures, the components have to be not only individually strong but the overall configuration of the bracing system has to be such that there is efficient load sharing by the system as a whole. This aspect of design reflects on the overall shape as well as the configuration of the internal bracing system. Since the design of the platform is also checked for *Ultimate Limit State* it is desirable to have a ductile mode of failure, hence, the design should ensure ductile behavior of the components and the complete system. Progressive Collapse Limit State analysis is also required to ensure that failure of any one element does not induce a progressive collapse. In chapter 9 the details of designing the ice walls and framing systems is discussed; Logic diagram F presents the procedure for ice wall design.

*Load Transfer to Foundation*

The load from the ice-structure interaction must eventually be transferred to the foundation soil without causing excessive movement of the structure. The most likely modes of failure for gravity based structures are sliding (shear) failure, and bearing failure. Sliding seems to be the dominant mode of failure since the Arctic seafloor sediments have lenses of weak silts, which are potential failure planes, and the ice loads have a large horizontal component. The design of the production structure must be such that there is an adequate factor of safety for a sliding failure through all possible planes, both below the seafloor and directly under the structure. For structures in deeper waters, bearing failure due to high overturning moments is also possible. Another form of foundation failure can arise from an oblique impact of an ice feature causing the structure to develop a rotational shear in the foundation. Thus, to
ensure a safe foundation the structural design must consider methods for providing foundation strength, either by having larger foundation areas or by alternate means such as foundation preparation by predredging and backfilling, spud foundations, strengthening by artificial means such as wick drains or artificially freezing soil, or by sinking the structure into deeper formations (as is the practice with bridge pier caissons).

Foundation design methods are discussed in detail in chapter 11 and logically shown in diagram G.

**Hydrodynamic Considerations**

If the structure is designed to be towed to the site, then it is extremely important that the floating characteristics of the structure be acceptable. The structures being towed from the ports of U.S. West Coast or Japan will have to pass through the North Pacific in summer and hence the structure should be stable enough so as not to cause excessive motion which could damage the structure or the equipment on board. Maximum draft limitations are also imposed by the shallow water region around Pt. Barrow. Section 12 discusses the naval architecture considerations of the platform design which is logically represented in diagram H.

**Summer Wave Considerations**

Besides resisting the ice loads the structure in the Beaufort Sea will be subjected to the open ocean environment in the brief summer season. During this time the structure will have to withstand the wave action around it. If the structure is not properly designed for wave action, heavy damage can occur. The various aspects to be considered in designing for the wave
environment are:

(a) Wave run-up on the structure
(b) Wave erosion
(c) Spray
(d) Wave refraction and mach-stem effects
(e) Local and global wave forces
(f) Wave acceleration of ice fragments

To resist these, the structure must have adequate freeboard, adequate drainage (even in below freezing temperatures) and appropriate external configuration, such as wave and spray deflectors. Chapter 10 discusses the wave consideration in detail.

Reliability Based Design

Due to the inherent high randomness in the ice loads and lack of understanding of ice as an engineering material it essential to rely only on reliability based design procedures for designing safe and economical production structures. Purely deterministic design methods will yield uneconomical or unreliable designs. Currently this aspect of design is a "hot" topic in the industry since it is universally agreed that only through such an approach can all the requirements, both legal and technical, be met. In practice, reliability is achieved through the use of limit state design, in which the loads and effects are factored and the resistance and/or capacity also factored, so as to achieve a predetermined degree of reliability.

Chapter 15 discusses in fair detail the methodology for carrying out reliability based design for various components of the structure.
Constructibility and Economics of Design

The feasibility of a production structure concept is contingent upon its constructibility. Constructibility is part of the design and influences the configuration, internal sub-division, mechanical systems, and sizing etc. Constructibility considerations are directed towards making the structure practicable to build. Constructibility and economic issues pertinent to the production structure design are discussed in chapter 13.
SOCIO-ECONOMIC ISSUES AND CONCERNS

**ISSUE**

**EMPLOYMENT OF REGIONAL RESIDENTS**
- In drilling, construction, and production
- In indirect and induced labor requirements
- In the Community
- In the business sector
- In the government sector

**DEVELOPMENT OF BUSINESS SECTOR**
- Drilling, construction and production supply and service needs
- Community population growth
- Rising levels of income
- Changing lifestyles

**NATIVE TRADITIONS & HARVESTING**
- Construction of shore bases, pipelines, etc.
- Population of community growth induced by development
- Increased vessel and vehicle movement

**CONCERNS**

**EMPLOYMENT OF REGIONAL RESIDENTS**
- Migration of growth centers
- Short duration of major projects
- Effect of cash earning on family life
- Interference with traditional trades

**DEVELOPMENT OF BUSINESS SECTOR**
- Limited availability of capital
- Increasing competition from non-residents
- Lack of business support services
- Insufficient managerial expertise

**NATIVE TRADITIONS & HARVESTING**
- Decline in traditional resource base
- Competition by non-native harvesters
- Decline in native culture
- Overharvesting near some communities
- Increased access to harvest areas

---

**Logic Diagram A** Socio-Economic Issues and Concerns (adopted from Environmental Impact Statement for Canadian Beaufort Sea, 1982)
<table>
<thead>
<tr>
<th>Environmental Concerns</th>
<th>Concerns</th>
</tr>
</thead>
<tbody>
<tr>
<td>MAJOR OIL SPILL</td>
<td>• Adequacy of prevention&lt;br&gt;• Adequacy of cleanup capability&lt;br&gt;• Impact on Wildlife</td>
</tr>
<tr>
<td>MARINE STRUCTURES</td>
<td>• Extension of landfast ice&lt;br&gt;• Effluent discharge&lt;br&gt;• Attraction to wildlife&lt;br&gt;• Coastal fisheries effects</td>
</tr>
<tr>
<td>STRUCTURES ON LAND</td>
<td>• Terrain disturbance&lt;br&gt;• Fish habitat disturbance&lt;br&gt;• Mammal habitat disturbance&lt;br&gt;• Bird habitat disturbance</td>
</tr>
<tr>
<td>DREDGING</td>
<td>• Effects on Crustacean community&lt;br&gt;• Removal and burial of sea-bottom life&lt;br&gt;• Creation of turbid waters&lt;br&gt;• Disturbance of migratory whales</td>
</tr>
<tr>
<td>ICEBREAKING</td>
<td>• Alteration of ice pattern&lt;br&gt;• Killing of seal pups&lt;br&gt;• Trapping of whales in artificial leads&lt;br&gt;• Interruption of access for native hunters</td>
</tr>
<tr>
<td>NOISE</td>
<td>• Aircraft noise disturbance to wildlife&lt;br&gt;• Underwater noise disturbance to marine mammals</td>
</tr>
<tr>
<td>WASTE</td>
<td>• Proliferation of solid wastes&lt;br&gt;• Effects of air emissions&lt;br&gt;• Effects of the discharge of drill muds, sewage, and produced water</td>
</tr>
</tbody>
</table>

Logic Diagram B Environmental Issues and Concerns (adopted from Environmental Impact Study)
TECHNICAL ISSUES

GLOBAL ICE LOADS
Sections 1-7
- Ridge Loads
- Ice Floe Impact Load
- Far Field Force

LOCAL ICE LOADS
Section 8
- Design Loads for Ice Walls and Framing System
- Ice Abrasion

STRUCTURAL DESIGN OF FRAMING SYSTEM
Section 9
- Design Methods for Framing System Design

WAVE CONSIDERATIONS
Section 10
- Wave Run-Up
- Wave Erosion
- Wave Loads

FOUNDATION DESIGN
Section 11
- Global Stability
- Foundation Element Design
- Contact Stresses
- Soil Improvement Methods

HYDRODYNAMIC CONSIDERATIONS
Section 12
- Towing Characteristics
- Towing Resistance
- Towing Configuration

CONSTRUCTIBILITY & ECONOMICS
Section 13
- Integration of design and construction methodology
- Economic issues

RISK AND RELIABILITY
Section 15
- Reliability Based Design
- Semi-Probabilistic Methods

Logic Diagram C  Technical Issues discussed in this study
Logic Diagram D  Logic Diagram for Computing Ice Forces on Vertical Structures
Logic Diagram for Computing Ice Forces on Conical Structures

1. **Season**
   - **Summer**
     - Identify Ice Features
       - Ice Floes
       - Ice Islands
     - Ice Feature Properties
       - Velocity, Mass, Size, Thickness
     - Ice-Material Properties
       - Unconfined Compressive Strength, Flexure Strength, Shear Strength, Temperature, Friction Coefficients, Density
       - Identify Possible Failure Modes
       - Select Empirical/Semi-Empirical Parameters
       - Use Available Models to Evaluate Failure Force
       - Dynamic Analysis
       - Run Dynamic Analysis to Determine the Maximum Force
       - Determine the Maximum
     - Limit Stress
     - Limit Force
   - **Winter**
     - Identify Ice Features
       - Ridges
     - Ridges Properties
       - Sail Height, Keel Height, Shape
     - Core Profile
     - Size of Feature
       - Wind Drag, Current Drag, Far Field Force

2. **Failure Modes**
   - Indentation Factor
   - Contact Factor
   - Added Mass Coefficient
   - Dynamic Analysis
   - Croasdale (1975)
   - Maximum
   - Compare with Driving Force and Take Least

**Logic Diagram E** Logic Diagram for Computing Ice Forces on Conical Structures
Logic Diagram F  Design Procedure for Ice Walls
GEOTECHNICAL DESIGN LOGIC DIAGRAM

GEOTECHNICAL PARAMETERS

- Soil Strength
- Deformation Properties
- Soil Profile

STRUCTURE DESIGN

LOADS ON FOUNDATION

Geotechnical Analysis

Material/Load Factors

Stability Analysis

- Sliding
- Bearing
- Overturning
- Slope Stability

(ULS) (SLS)

Design Acceptable

Deformation Analysis

Design Acceptable

Modify Design

Logic Diagram G  Foundation Design Logic Diagram
Logic Diagram II  Logic Diagrams for Marine Considerations of Production Structure Design
SEA ICE FEATURES

1. Sea Ice Features

1.1. Arctic Sea Ice

The entire Arctic ice canopy can be divided into three distinct zones, the Polar Pack, the Stamukhi Shear Zone, and the Land-Fast Zone, Fig. 1.1. The polar pack, which constitutes about 70% of the entire Arctic Ocean surface, is a floating mass of ice which moves in clockwise direction around the North Pole due to the action of the winds and the currents of the Arctic Gyre. The motion at the periphery of the ice pack is at the rate of about 2 km/day (Wright et al 1978). The polar pack typically extends to water depths of 100 feet and beyond (Figure 1.2). The ice of the polar pack is not smooth surfaced but is characterized by regions with protruding uneven masses of ice called Hummock Fields. Occasionally, huge embedded fresh water Ice Islands of extreme thickness are found in the polar pack.

In summer the polar pack recedes further into the sea, rendering the narrow annulus between the polar ice pack and the land ice free. Sometimes, due to wind and current drag forces, this open water channel is invaded by the polar pack.

In early winter the freeze-up begins at the shore and progresses seaward forming the landfast ice, Fig. 1.3. The landfast ice sheet grows up to a thickness of 6 ft. (2m) in one season. The first 15 miles (25 Km.) of the landfast ice is relatively smooth surfaced, whereas the remaining ice, which extends to water depths of about 60 ft. (18 m.), is ridged and very uneven. Due to wind and current forces, the ice cracks and deforms in this region to form ice ridges. Ice features that are able to survive two melt season develop stronger
cores and are called multi-year ice features. Sometimes ice floes and ice island fragments drift into the landfast ice.

A narrow zone exists between the interface of the Landfast ice and the Polar Pack where the energy is transferred from the fast moving Polar Pack to the Landfast ice, which is relatively stationary. The ice in this region experiences high shear forces: thus this zone is appropriately called the Stamukhi Shear Zone. As the Polar Pack interacts with the Landfast ice sheet, it causes it to crack and ride up, producing pressure ridges and rubble fields.

The zones of current interest for petroleum exploitation are in the Landfast and Shear zones of the Arctic Sea ice. Thus any structure deployed in any portion of the Arctic Ocean other than very shallow zones (less that 25 feet) must be designed to withstand forces generated by: 1) interaction of the structure with multiyear ice features, and 2) rare features, such as ice floes with embedded ridges and ice island fragments. The following section gives a brief description of these ice features.

1.2. Pressure Ridges

The sheet ice cracks due to the motion induced by wind, currents, and tide; these cracks widen to form leads. Such leads can become wide enough for the ships to navigate through them, but eventually, wind action and currents cause the leads to close up. As these leads close, ice sheets impinge against each other and the resulting impact forces cause the ice to deform under the high contact pressures. As the ice fails, broken pieces of ice pile up on the sheet. Such a pile-up leads to the formation of features called Pressure Ridges. Based on the mode of formation, the Pressure Ridges are classified into two categories: 1)p-type, and 2)s-type. The p-ridges are formed due to closure of leads by movement normal to the lead, causing the sheet ice to fail
in crushing or buckling. Such ridges are composed of ice slabs of various sizes. The highest free floating ridge observed to date had a sail height of 42 ft. above the sea level and a keel depth of 150 ft. (Wright et al 1978). The size of a ridge in any region depends upon the water depth, since the larger ridges become grounded when they are driven into shallower water.

The s-ridge, also called the shear ridge, is formed due to closure of leads by motion parallel to the edge of the lead. This type of motion results in high shear forces, which grind the ice into fine particles. These then accumulate into ice blocks and pile-up to form a pressure ridge.

A first year ridge has an accumulation of broken ice pieces with large voids in its cores. If the first year ridge survives a melt season, the voids in the core of the ridge get filled up with melt water, which eventually refreezes to form a solid core. The core of the multiyear ridge also exhibits higher strength since the brine drains from the ice in summer while fresh melt water from the surface snow fills the voids and refreezes to form a solid core with low salinity ice.

From an engineering standpoint, the important properties of a ridge are its dimensions and the strength characteristics of the ice itself. In 1971 APOA sponsored a field investigation to study the pressure ridges in the Southern Beaufort Sea. As a result of this study and some other independent studies (Wright et al 1978), an idealized model for the multiyear ridge was proposed by Kovacs (Fig. 1.4).

Sometimes, instead of having a well-defined pressure ridge, the ice sheet deforms at a number of points and a large area of sheet ice develops heaps of ice blocks. This is called a rubble field.
1.3. Ice Floes

Ice floes are large masses of ice which break off the polar pack and enter the open water of the shear zone. Figure 1.5 shows a map prepared by NOAA showing the ice conditions in the Beaufort Sea, in August 1983. The ice map shows a very large ice feature just off the North Slope of Alaska. This ice feature was identified as an ice floe. Such floes attain velocities of up to 1.5 m/sec. due to wind and currents. Since these floes have large masses, they possess extremely large kinetic energy. Thus any structure which is impacted by an ice floe has to convert this energy to alternate forms of energy without sustaining significant damage. Multi-year floes have been sighted in the southern Beaufort Sea with embedded pressure ridges,

1.4. Ice Islands and Ice Island Fragments

Ice Island fragments have been sighted in the Stamukhi shear zone at water depths greater than 100 ft. Ice Islands can be 100-150 feet in thickness and several hundred square miles in area. Ice islands are formed when large pieces of ice break off the ice shelf on the north side of Ellesmere Island and enter the Arctic gyre. The ice islands then get encapsulated in the polar pack and eventually break up into fragments, typically less than 100 m. in diameter. During summer these ice island fragments enter the open water and start to travel with higher speeds due to wind and currents. Structures must be evaluated for survivability, i.e. in the PLS (Progressive Collapse Limit State), to ensure against catastrophic failure in event of encounter with ice island fragments.
Figure 1.1 Ice Features in the Beaufort Sea
Figure 1.2 Ice Zones in the Beaufort Sea
Figure 1.3 Seasonal Development of Ice Zonation
Figure 1.4 Multi-Year Pressure Ridge Model (After Kovacs unpublished)
Figure 1.5 Ice Condition Map of Arctic Ocean (Note the Ice Floe)
2. Sea Ice Properties

Although it is not the intention of this report to present a detailed discussion of the properties of sea ice, it is considered important to review the physical and mechanical properties of ice to better understand sea ice as an engineering material. This section outlines briefly the physical characteristics of sea ice and discusses the engineering properties of ice relevant to the design process.

2.1. Sea Ice Crystallography

For pure fresh water ice-crystals the tetrahedral configuration is the simplest type of crystallographic orientation. Tetrahedral crystals have one oxygen atom surrounded by four equally spaced oxygen atoms (Figure 2.1). One hydrogen atom between every two oxygen atoms provides the necessary bond forces for the stability of the molecules. Due to the tetrahedral shape of the crystals the molecular configuration has a hexagonal symmetry (Figure 2.2). These molecules of ice are concentrated in a series of parallel planes called the Basal planes. The normal to the Basal plane is referred to as the C-axis of the crystal. The crystal growth in columnar ice is such that the C-axis is horizontal and the ice is isotropic through the thickness. Growth pattern of its physical ice crystals affects the properties of sea ice. Since the lattice of ice is very open, i.e. the spacing of the oxygen atoms is very large compared to the size of the atoms themselves, the density of ice is less than that of water.

The above given description is for freshwater ice; the sea ice does is highly saline and hence does not show any isotropy through the thickness.
The sea ice sheet has three distinct zones. Each zone has unique crystallographic configuration (Figure 2.3). The topmost layer is called the skim and its thickness varies from a few millimeters to about 20 cm. The skim layer is the first layer of ice to be formed - when cold air comes in contact with relatively calm water, a thin skim layer of ice is formed. However, in rough waters, continuous mixing due to wind and waves does not allow the water in the top few inches to freeze, but instead makes it super-cooled. With the aid of nuclei from the atmosphere or the ocean this super-cooled water eventually freezes to form frazile ice. Being lighter than water, frazile ice floats to the surface and refreezes with other ice to form a thin skim layer. In the skim, crystals do not have any preferred axis of orientation, hence such ice is called Unoriented Granular Ice.

The ice layer below the skim is called the transition layer, for in this layer the ice crystals begin to develop a definite orientation of growth. The crystals in this zone also begin to elongate (Figure 2.4). The average crystal size in this layer is 1x2x4mm. These top two layers, skim and transition, melt during the summer leaving behind only the third layer - the columnar layer. From an engineering standpoint the top two layers are not considered important, since their thickness compared to the columnar layer is very small.

In the third zone, or Columnar zone, ice has a preferred axis of vertical growth and has a horizontal C-axis. The crystals in the zone have an elongated shape since the growth is very rapid in the vertical direction. The average crystal size in Columnar ice is 6x12x24 mm (Figure 2.4).

Salinity of the ice is by far the most important factor influencing the mechanical properties of ice. As the sea water begins to freeze, brine concentration in the unfrozen regions begins to increase due to brine migration. The
brine is eventually drained out due to gravity and concentration gradients, leaving behind brine channels. These channels run down through the ice like a river system. Figure 2.5 shows a picture of a 15cm thick ice sample with distinctly visible brine channels. As the freezing continues, the ice crystals bridge across the brine channels and trap brine to form brine pockets. Near the bottom of the ice sheet the main drainage channels, spaced about 15 to 20cm apart, are approximately 1cm in diameter (Schwarz and Weeks, 1977).

A typical salinity profile for ice is shown in figure 2.7. Note that for multi-year ice the salinity reduces at the top and increases toward the bottom, this is due to the downward migration of brine under gravity.

2.2. Physical and Mechanical Properties of Sea Ice

Sea ice is an anisotropic and inhomogeneous material, whose properties depend upon a number of factors such as temperature and salinity. Most of the engineering properties of sea ice are obtained by laboratory tests on samples obtained in the field. Since the size of sea ice crystals is of the same order of magnitude as the size of the sample core the scale effects become important. Thus validity of the laboratory results is questionable.

2.2.1. Effect of Salinity

Physical properties of sea ice are greatly dependent upon the salinity or the brine content of the ice (salinity is defined as brine volume in parts per thousand). Brine channels in the ice affect the strength characteristics of the ice. Assur (1958) developed a relationship for the strength and the salinity of the ice by assuming that the brine pockets reduce the effective contact area between ice platelets, thus causing the ice to have a weak plane along which the failure can take place. The idealized model for the sea ice is shown in
The strength of sea ice can be written as:

\[ f = f_0 (1 - p) \]  \hspace{1cm} (2.1)

where \( f_0 \) is the fictitious strength of ice corresponding to zero porosity, \( p \). Assur also suggested that the porosity of ice can be calculated by assuming that the pores of ice are cylindrical in shape: therefore the porosity is proportional to the brine volume. Thus:

\[ f = f_0 \left[ 1 - \sqrt{\frac{v}{v_0}} \right] \]  \hspace{1cm} (2.2)

where \( v_0 \) is the reference brine volume (%) and \( f \) is the strength of ice with brine volume \( v \).

This equation is only valid for brine volume \( v < 0.5 \% \), for brine content greater than this the strength of ice has been observed to be constant (figure 2.15).

The salinity of ice changes rather systematically with ice thickness, temperature, and age. Figure 2.7 shows one such relationship.

2.2.2. Effect of Temperature

Temperature plays a key role in the behavior of ice. The physical properties of ice are highly dependent upon the temperature. The dependence of ice strength on temperature is a crucial factor in design since the behavior of ice dramatically changes with temperature. Referring to figure 2.16, it can be seen that the ice becomes a brittle material at colder temperatures and exhibits ductility at higher temperatures. During summer the strength of the ice could be lower. However due to the ductile behavior it may have a high
contact factor (figure 3.8, and 3.9), approaching 1.0. In winter although the
strength is higher, the brittle behavior will produce non-simultaneous failure,
resulting in a relatively low contact factor. Therefore the global force is
influenced greatly by the temperature.

2.2.3. **Behavior of Ice Under Loading**

Michel (1978) suggested that ice behaves like a pack of cards under load-
ing, the cards representing the basal planes of the crystalline structure of the
ice. When this pack of ice crystals is subjected to a load parallel to the basal
planes, it deforms without offering much resistance, however when loaded
normal to the basal plain, ice exhibits higher strength. The behavior of ice
when loaded normal to the basal plane, has been observed to be highly related
to the rate of loading. At high strain rates the ice behaves as a *brittle* material
while at slower strain rates the behavior is *ductile*. Under sustained loads and
low strain rates, the ice undergoes substantial creep.

2.2.3.1. **Ductile Behavior**

Michel and Toussaint (1977) conducted model tests to study the behavior
of ice sheet at various loading rates. They observed that the ice failure mode
is highly related to the rate of loading. At slow rates of loading the ice yields
like a ductile material, the yield strength being the maximum sustainable load.
From model tests the ice was observed to behave in a ductile manner for the
following strain rates:

\[ 10^{-8} \, \text{s}^{-1} < \varepsilon < 5 \times 10^{-4} \, \text{s}^{-1} \]  \hspace{2cm} (2.3)

(note the units of strain rate is cm/sec/cm, which is equivalent to sec \(^{-1}\))
Michel and Toussaint (1977) combined their data with other available indentation data to form a Universal Indentation Curve for S2 Ice (Polycrystalline Ice) (Figure 2.8). The data was also fitted to Glen's secondary creep equation to yield the following relationship:

\[
\dot{\varepsilon} = A \exp \left[ -\frac{Q}{RT} \right] \sigma^n
\]  

(2.4)

\[
\dot{\varepsilon} = 470.76 \exp \left[ -\frac{10.907}{T} \right] \sigma^{3.097}
\]  

(2.5)

where A = 470.76

Q is the activation energy = 21,662 cal/mole (90,699 J/mole)

R is the universal Gas constant = 1.986 cal/mole K (8.316 J/mole K)

n = 3.097 for S2 ice (This is the slope of the Stress vs Strain Rate Curve when plotted on log-log plot)

T is the absolute temperature in °K

From equation 2.4, the strength of ice at any strain rate (\(\dot{\varepsilon}\)), in the ductile range and at a fixed temperature, can be written as follows:

\[
\sigma = \sigma_0 \left[ \frac{\dot{\varepsilon}}{\dot{\varepsilon}_0} \right]^{1/3}
\]  

(2.6)

where \(\sigma_0\) is the reference strength of ice at a reference strain rate of \(\dot{\varepsilon}_0\) in the ductile range, and \(\sigma\) is the yield strength corresponding to the strain rate \(\dot{\varepsilon}\).
2.2.3.2. Brittle Behavior

Under high strain rates the ice deforms elastically and is followed by a sudden brittle failure. There is no plastic deformation of the ice and most of the deformation is due to the changes in the molecular distances within the crystal of the ice. The brittle range, as defined by Michel and Toussaint's model tests (1977), is $e > 10^{-2}$/sec. The uniaxial compressive strength of S2 ice is independent of the strain rate in the brittle zone, however, it is related to both temperature and size of the ice crystals (Michel 1978). The strength of ice in the brittle range is given by the following equation:

$$\sigma_0' = 9.4 \times 10^3 \left( \frac{1}{\sqrt{d^*}} + 3T^{-0.78} \right)$$  \hspace{1cm} (2.7)

where $T$ is the temperature in °C, and $d^*$ is the equivalent crystal diameter in millimeters and is related to the average grain size, $d$, by the following relation:

$$d^* = \frac{3\pi d}{8}$$  \hspace{1cm} (2.8)

The Tensile strength of ice in the brittle range can also be expressed in a similar form (after Michel, 1978):

$$\sigma_t = 0.079 \sqrt{\frac{1-9 \times 10^{-3} T}{d^*}}$$  \hspace{1cm} (2.9)

2.2.3.3. Transition Zone

For strain rates greater than $5 \times 10^{-4}$/sec and less than $10^{-2}$/sec the
behavior of ice is unstable. The strength of ice as observed by Michel and Toussaint (1977) increases to a value higher than the maximum yield strength of ice in the ductile range before it drops to a constant value, which corresponds to the strength of ice in the brittle zone (Fig. 2.8). The data from laboratory tests obtained by Michel and Toussaint (1977), when fitted to Glen's creep equation, yields the following expression for strength of ice in the transition zone:

\[
\sigma = \sigma_0 \left[ \frac{\dot{\varepsilon}}{\dot{\varepsilon}_0} \right]^{-0.126}
\]  

(2.10)

For engineering purposes this increase in strength in the transition zone is ignored and it is assumed that it remains constant.

Within the transition zone the mode of failure of ice is not defined. The ice can fail either in ductile or brittle mode. The main factor determining whether the ice will fail in a ductile mode or a brittle mode is the number of dislocations present in the ice (Michel 1978). If there are a small number of dislocations present, then the ice deforms primarily as an elastic solid and fails in a brittle mode. On the other hand, a large number of dislocations make the ice deform plastically.

2.2.4. Strength of Ice

So far we have discussed the behavior of ice under different conditions of loading, but have not addressed the strength of the ice as such. From an engineering standpoint the uniaxial compressive strength, confined compressive strength, flexure strength, and shear strength are important. The effects of salinity and temperature on these properties are also important and these are therefore discussed in this section.
2.2.4.1. Compressive Strength

Michel (1978) obtained strength data for different types of polycrystalline ice for a wide range of strain rates (see section 3.1.1 for definition of strain rate) and temperatures. After reducing this data to a standard temperature of -10 °C the unconfined compressive strength of ice was plotted as shown in Fig. 2.9. Here T1 is the snow ice, S1 is the Frazil ice, S2 is the Polycrystalline ice, and S4 is the Pseudo-Mono crystalline ice. It can be seen here that the ice strength is independent of the strain rate in the Brittle zone.

Fig. 2.10 shows the compressive strength as a function of strain rate for Baltic Sea ice. The plots shown are of two types, one for the axis of loading along the C-axis and the other perpendicular to the C-axis. It is clear from this plot that the strength of ice is higher for loading along the C-axis.

Exxon conducted extensive field and laboratory investigations to determine the properties of sea ice (Wang, 1979). The unconfined compressive strength of three different types of sea ice, granular, columnar, and unoriented columnar ice, is shown in Figs. 2.11, 2.12, and 2.13, respectively. Exxon’s data has been widely used and has also been recommended by the API Bulletin 2N.

Vaudrey’s (1977) field investigations of sea ice properties revealed compressive strength data shown in Fig. 2.14.

**Effect of Brine Volume on the Compressive Strength**

The variation of compressive strength with a change in salinity is shown in Fig. 2.15 (Peyton, 1966). In this plot \( \sigma_r \) is the index strength, which is expressed as:

\[
\sigma_r = \frac{\sigma_c}{\phi}
\]  
(2.11)
where,

\[ \sigma_c \] is the compressive strength of ice

\[ \dot{\sigma} \] is the rate of stress application

\( b \) is an experimental constant = 0.22

The broken line in Fig. 2.15 is by Peyton (1966) and the solid line is by Weeks and Assur (1967) and is defined by the following expression:

\[
\sigma_r = 16.5 \times 10^5 (1 - \frac{v_b}{0.275})^{1/2} \text{ N/sq.M}
\]  \hspace{1cm} (2.12)

for \( 0.2 < v_b < 0.5 \)

where \( v_b \) is the brine volume.

For brine volume \( v_b > 0.25 \) or \( v_b > 0.5 \), \( \sigma_r \) is considered to be constant, i.e. strength is independent of the brine volume.

**Effect of Temperature on the Compressive Strength**

The uniaxial compressive strengths shown in Figures 2.11, 2.12 & 2.13 are all for a standard temperature of -10 °C. For any other temperature the strength of ice is different. The variation in the strength of ice with temperature is actually due to the change in brine volume associated with the change in temperature. Schwarz (1970) conducted tests on Baltic Sea ice to determine the variation in strength with temperature, figure 2.17 shows his results. Figure 2.16 also shows compressive strength as a function of strain rate and temperature (after Wu et al 1976).

**2.2.4.2. Tensile Strength**

Direct tension is the only unambiguous way of measuring uniaxial tensile strength, However, a number of investigators have chosen other indirect
means of determining tensile strength, such as ring tensile strength, beam test or brazil test. The indirect tests complicate the stress state in the material and the results are at best ambiguous.

Michel's (1978) results for pure tensile strength of various types of polycrystalline ice are shown in Fig. 2.18. The tensile strength is shown as function of temperature. Dykins (1971) also conducted tests to determine the tensile sea ice. His results are shown in Fig. 2.19 for two cases of loading, one for loading along the C-axis and the other for loading perpendicular to the C-axis. The tensile strength is shown as a function of brine volume. Weeks and Schwarz derived the following expression from Dykin's data:

\[
f_t \, (vertical) = 154 \times 10^5 \left(1 - \frac{v_b}{0.31}\right)^{0.5} \text{ N/sq.M} \tag{2.13}
\]

\[
f_t \, (horizontal) = 8.2 \times 10^5 (1 - \frac{v_b}{0.142})^{0.5} \text{ N/sq.m} \tag{2.14}
\]

where \(v_b\) is the brine volume.

2.2.4.3. Flexure Strength

Flexure strength is not a basic material property, but it is a useful parameter in engineering applications. The flexure strength is used in determining the forces required to fail the ice features in bending.

The flexure strength is measured by bending tests on beams or on cantilevers. The cantilever test can be carried out in the field by cutting a cantilever in the ice feature and the loading it in situ. For fresh water ice, the flexure strength measured by beam test is about 50% higher than the results from cantilever test. This variation in strength is thought to be due to the
stress concentration at the supports. Fortunately, sea ice is more plastic than fresh water ice; thus the stress concentration is not as severe. Therefore the test results by both methods are reasonably in agreement for sea ice. In the bending tests it is assumed that the deformation through the thickness of the beam is elastic and that the ice is isotropic and homogeneous. Since this is so, the flexure strength of the cantilever can be written as:

\[ \sigma_f = \frac{6PL}{b^2t^2} \]  

(2.15)

where \( P \) is the load causing the failure, \( L \) is the span of the cantilever, \( b \) is the width of the cantilever, and \( t \) is the thickness of the cantilever.

The above stated assumptions are certainly not representative of the actual conditions, since sea ice is highly anisotropic and non-homogeneous. Some researchers have developed equations for the flexure strength of ice by considering the anisotropy and the inelastic behavior of ice (Hutter, 1973). However, for the purposes of obtaining flexure strength as an index property, the assumptions are adequate (Schwarz and Weeks, 1977).

Flexure strength varies with a change in brine volume in a manner similar to compressive strength. Schwarz and Weeks (1977) developed the following relationship for the flexure strength and brine volume from in situ tests on cantilevers (Fig. 2.20):

\[ \sigma_f = 7.5 \times 10^5 \left(1 - \frac{v_b}{0.202}\right)^{0.5} \text{ N/sq.m for } \sqrt{v_b} < 0.33 \]  

(2.16)

\[ \sigma_f = \text{ constant for } \sqrt{v_b} \geq 0.33 \]  

(2.17)
Test data gathered by Dykins (1971) led to the development of the following expression for flexure strength and brine volume (figure 2.21):

$$\sigma_f = 10.3 \times 10^5 (1 - \frac{v_b}{0.209})^{0.5} \text{ N/sq.m}$$ (2.18)

Vaudrey (1977) also developed an expression for the flexure strength and brine volume from in situ cantilever tests.

$$\sigma_f = 0.96(1-\sqrt[2]{\frac{v_b}{0.250}}) \text{ N/sq. m}$$ (2.19)

2.2.4.4. Shear Strength

Only a few tests have been conducted to evaluate the shear strength of sea ice. The test results so far indicate that the shear strength is strain rate dependent, yet this dependency has not been completely resolved (Schwarz and Weeks, 1977). Fig. 2.22 shows shear strength vs brine volume (after Paige and Lee, 1967).

2.2.4.5. Confined Strength of Sea Ice

In certain conditions the ice impinging against the structure might be confined due to the structure and/or the adjoining ice. In such a case, the stress state in the ice is not uniaxial but is rather multi-axial. Figure 2.23 shows results obtained from biaxial compression testing of sea ice at -10° C. The test results shown are for two types of loading, one for confinement along the c-axis and the other with confinement perpendicular to the c-axis. For comparison purposes the unconfined compressive strength is also plotted. The effect of confinement is clearly visible; the confined compressive strength
is about twice the unconfined compressive strength at the same strain rate.

Triaxial tests have also been conducted on sea ice. Figure 2.24 shows some triaxial test results by Jones (1982). The triaxial strength is about four times the unconfined compressive strength.

2.2.5. Elastic Modulus for Sea Ice

At high rates of loading the ice behaves as a brittle material and deforms elastically before failing. In the brittle zone the elastic modulus for ice can be defined, whereas in the ductile behavior zone the elastic modulus is not definable. The Elastic Modulus for sea ice is obtained by either dynamic or static testing. In dynamic testing, seismic waves are propagated through the ice and extremely small displacements are measured along the sea ice surface, from which the Elastic Modulus is computed. Static testing gives more meaningful results for engineering purposes.

The Elastic Modulus is highly dependent on both the brine volume and the temperature of the ice. The temperature dependency is explained by the fact that the brine volume changes with temperature and that some salts precipitate at low temperatures. Fig. 2.25 and 2.26 show Elastic Modulus as a function of the brine volume. The Elastic Modulus is also dependent on the strain rate as can be seen in Fig. 2.28.

2.2.6. Poisson’s Ratio

Weeks and Assur (1967) analysed the data from Kap Shmit, Siberia and concluded that the Poisson’s Ratio is independent of the brine volume for porosity up to 30 %, and that its value is "0.33" This value of Poisson’s Ratio is valid as long as the ice undergoes elastic deformation. If the ice is plastically deformed, then the Poisson’s Ratio increases to as high as 0.8 (Schwarz
2.2.7. Friction Coefficient

For sloping sided structures, the friction of the ice contributes to the total force on the structure. Thus it is important to know the friction coefficients of ice for various materials. Fig. 2.29 summarizes the coefficients of friction between ice and steel, and concrete and ice. As can be seen from this data, the friction coefficient is highly dependent upon the temperature and surface roughness of the material.

2.2.8. Adfreeze Bond Strength

If the ice around the structures does not move for a period of time, then it is possible that it will freeze to the exterior walls of the structure. As a result of the adfreeze, the failure mode of the ice feature could be altered. For example, in the case of a conical structure the ice sheet will fail in bending if there is no adfreeze. However in the case that the ice sheet is adfrozen to a conical structure with high bond strength, the failure could be in crushing. Sackinger (1977) reported adfreeze strengths of ice on steel. He obtained a maximum value of 227 psi. Saeki (1981) has also reported adfreeze bond strengths for ice with various materials (figure 2.30).

Low friction adfreeze coatings can be used on the external surface of the structure to reduce the adfreeze forces. Of the various materials available Zebron (a 100% solid polyurethane) and Inerta 160 (a high solids pure epoxy) have been used on the exploratory structures. Gulf Canada's Kulluk and Mollikpak were both coated with Zebron. The coefficient of friction for these materials range from 0.12 to 0.3.
Adfreeze bond strengths of 5 to 10 psi for Zebron and 25 to 31 psi for epoxy based material have been reported (Alliston, 1985). It is believed that values for Inerta 160 are comparable for Zebron.

2.3. Relevance of Sea Ice Properties

From the engineering standpoint, it is important to understand the relevance of various factors that influence the sea ice properties on the design process for production structures.

Based on the above presented data, it can be seen that the engineering properties of sea ice are highly influenced by the following factors:

1) Temperature
2) Salinity of Ice
3) Rate of Loading (strain rate)

Temperature dependence of sea ice properties, particularly strength, is very dramatic. The strength of ice is much increased at lower temperatures. Equally important is the change in behavior of ice under loading at various temperatures. At very low temperatures, ice behaves as a brittle material. At higher temperatures, ice shows ductile behavior. This aspect of ice behavior is of great importance to the designer, for at low temperatures, although the ice has higher strengths, the contact factor is reduced due to brittle behavior (figure 3.8). At higher temperatures the strength is lower but the failure is ductile and therefore the contact factor is also higher. Warm ice also penetrates further into the structure, increasing the interaction area.

In terms of design it is important to study a number of possible cases with various temperatures to determine the worst-case loading scenario.

Salinity plays an important role in determining the properties of sea ice. Since the salinity of first year ice is higher than that of multi-year ice, its
strength is lower. Ice features that survive a few melt seasons have reduced salinity and consequently higher strength.

Strain rate dependence of sea ice properties greatly influences the ice strength. Thus during design it is crucial to determine the effect of varying the strain rate on the design ice force.

The design engineer, therefore, will be concerned with the three parameters of temperature, salinity, and strain rate, as he develops his loading criteria.
Figure 2.1 Tetrahedral Arrangement of Oxygen Atoms in Ice (after Michel, 1978)

Figure 2.2 Molecular Structure of Ice. a) View Perpendicular to C-axis b) View Along C-axis (after Michel, 1978)
Figure 2.3 Schematic Drawing Showing Various Layers Of Sea Ice (after Schwarz and Weeks, 1977)
Figure 2.4 Section Through Sea Ice Sheet (Wang, 1979)
Figure 2.5 (a) A Cross Polaroid Photograph of Sea Ice Sheet. (after Eide and Martin. 1975)
Figure 2.5 (b) Crystal Structure of Landfast Ice at Notsuke-Odaita
Figure 2.6 Brine Channels in Sea Ice (after Assur, 1958)

Figure 2.7 Salinity Distribution Through the Thickness of Ice Sheet. (after Weeks and Assur, 1967)
Figure 2.8 Universal Curve for Uniaxial Crushing and Indentation of S2 Ice (after Michel and Toussaint, 1977)
Figure 2.9 Unconfined Compressive Strength as a Function of Strain Rate for Different Types of Fresh Water Ice. (after Michel, 1978).
Figure 2.10 Compessive Strength of Baltic Sea Ice as a Function of Strain rate, Ice Temperature, and Orientation of the Force (after Schwarz, 1971)

Figure 2.11 Compressive Strength of Granular Ice as a Function of Strain Rate (after Wang, 1979)
Figure 2.12 Compressive strength of unoriented columnar sea ice as a function of strain rate (Wang, 1979)
Figure 2.13 Compressive strength of oriented columnar sea ice as a function of strain rate (Wang, 1979)
Figure 2.14 Compressive strength of unoriented columnar sea ice as a function of strain rate (Wang, 1979)

Figure 2.15 $\sigma_R$ from compressive strength tests vs. brine volume (Peyton, 1966)
Figure 2.16 Temperature Dependence of Compressive Strength in Transition Zone. (after Wu et al, 1976)

Figure 2.17 Compressive strength of Baltic Sea ice and fresh water ice as a function of temperature. (Schwarz, 1971)
Figure 2.18 Tensile strength of various types of polycrystalline ice as functions of temperature (Dykins, 1970)
Figure 2.19 Tensile strength of sea ice as a function of brine volume. (Dykins, 1970)

Figure 2.20 Flexure strength of sea ice as a function of brine volume, from insitu cantilever tests. (Weeks & Schwarz, 1977)
Figure 2.21 Flexure strength measured by beam tests vs. brine volume. (Dykins, 1971)

Figure 2.22 Shear strength as a function of square root of brine volume. (Paige & Lee, 1967)
Carter and Michel's Unconfined Strength Tests

Frederking's Plane Strain Strength Tests

Unconfined and plane strain ice strength tests for columnar-grained ice.

Figure 2.23 Confined compressive strength of sea ice. (Ralston, 1977)
Figure 2.24 Triaxial compressive strength of fine grained polycrystalline ice. (Jones, 1982)
Figure 2.25 Elastic Modulus of sea ice as determined by small specimen tests vs. brine volume. (Langleben and Pounder, 1963)

Figure 2.26 Elastic modulus of sea ice as determined by seismic techniques vs. brine volume. (Dykins, 1971)
Figure 2.27 Apparent elastic modulus as a function of brine volume. (Vaudrey, 1977)
Figure 2.28 Elastic modulus for fresh water ice as a function of strain rate at -10 °C and at low stress level. (Traetterberg et al, 1975)
Figure 2.29 Static and kinematic friction coefficients for ice (Saeki et al, 1979)
<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>ADFREEZE BOND STRENGTH (PSI)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Painted Steel (Saeki, 1981))</td>
<td>23</td>
</tr>
<tr>
<td>Steel (Saeki, 1981)</td>
<td>27</td>
</tr>
<tr>
<td>Concrete (Saeki, 1981)</td>
<td>44</td>
</tr>
<tr>
<td>Wood (Saeki, 1981)</td>
<td>54</td>
</tr>
<tr>
<td>Corroded Steel (Saeki, 1981)</td>
<td>66.5</td>
</tr>
<tr>
<td>Steel (Sackinger, 1977)</td>
<td>227</td>
</tr>
</tbody>
</table>

**Figure 2.30**  Adfreeze Bond Strength for Sea Ice
3. Loads Due to Crushing of Ice

If a structure is impinged by an ice feature, the interaction force is limited by the failure force of the ice feature, i.e. the minimum force required to fail the feature will not be exceeded during the interaction. The failure mode of ice features is dependent upon the type of feature and the shape of the structure it interacts with. Thick ice features, such as ice floes, ice island fragments, and multiyear ridges are likely to fail in crushing. Other features such as ridges can fail in crushing or in shear. Ice features interacting with sloping sided structures may fail in bending.

The following sections describe the analysis technique for evaluating forces due to crushing of ice.

3.1. Analysis Technique

The force exerted on a structure by an ice feature due to pure crushing type failure is defined by the following equation:

\[ F = p^*D^t \]  

(3.1)

where \( p \) is the effective ice pressure, \( D \) is the interaction width of the indenter, and \( t \) is the thickness of the ice. Here, effective pressure is used to account for the confined strength of ice. Korzhavin (1962) related the effective pressure to the unconfined compressive strength of ice by the following relationship:
Here, $I$ is the indentation coefficient (to account for confining effects), $m$ is the shape factor (to account for the shape of the indentor), $k$ is the contact factor, (accounting for the incomplete simultaneous contact of ice over the whole area),

$\sigma$ is the unconfined compressive strength of the ice, and $V$ is the velocity of ice sheet relative to $V_0$; (a reference velocity). Korzhavin's expression is valid for deformation rates of $10^{-3}$ to $10^{-4}$ per second. The generalized form of Korzavin's equation for all rates of deformation is as follows:

$$F = I f_c m \sigma$$

Where $I$ is the Indentation Factor, $f_c$ is the contact factor, and $m$ is the shape factor. The value of $\sigma$ is selected corresponding to the strain rate (see figures 2.11, 2.12, & 2.13 for strength vs. strain rate). This form of the crushing equation is most widely used and has been adopted by the API Bulletin 2N also.

Now we examine the individual variables in the equation and see their influence on the crushing force.

3.1.1. Strain Rate Effects

Michel and Toussaint (1977) conducted model tests in the laboratory on the indentation of ice plates. They observed that the ice plate's mode of failure depended on the rate which the load was applied, i.e. strain rate.
The ice failed in a brittle manner at high rates of indentations and showed ductile behavior at lower indentation rates. They also observed that the area of microcracking did not extend beyond 2.5 times the width of the indentor. This value was observed to be constant for all widths of the indentor, thicknesses of the plate and the rate of indentation in the ductile range. With these observations they set up a model for the indentation of the ice sheet which is reproduced in Fig. 3.1. Since most of the deformation was shown to occur in the plastification zone only (area adjacent to the indentor), the strain rate was defined as:

\[ \dot{\varepsilon} = \frac{V}{4b} \]  

(3.4)

where \( V \) is the velocity of the indentor and \( b \) is the width of the indentor.

Ralston (1978) proposed a different expression for strain rate, claiming it better fits the existing data.

\[ \dot{\varepsilon} = \frac{V}{2b} \]  

(3.5)

Vivatrat (1982) studied the problem by considering different deformation patterns and concluded that both equations 3.4 & 3.5, are reasonable. API 2N has adopted Ralston’s expression. The width of the indentor as referred to in the strain rate equation pertains to the width in contact with the ice. For large structures, the width of indentor may or may not be the diameter of the structure but rather the width in contact.
3.1.2. Indentation Factor

Experiments have revealed that when an indentor is forced into an ice sheet the crushing pressures vary with the rate of the indentation and that such pressures are higher than the uniaxial compressive strength of ice (Michel and Toussaint, 1977). The indentation factor $I$ in eq. 3.2 & 3.3 is thus defined as the ratio of the indentation pressure and the uniaxial compressive strength of the ice. The increase in ice strength during indentation is due to the confinement provided by the adjoining ice and the face of the indentor. Since the confining effects are more pronounced in the case of a narrow indentor, indentation factor is higher for narrow indentors; in comparison wide indentors have less confining effects resulting in lower strengths. The strength of ice is usually defined in terms of uniaxial compressive strength obtained in the laboratory from tests on small samples, hence the modification of this strength to account for the real life situation with appropriate confinement is done by the use of the indentation factor.

3.1.2.1. Indentation Factor for Granular Ice

Granular ice is considered to be an isotropic material and thus the yield criteria for isotropic material can be used to determine the indentation factors. A number of researchers have made attempts to find the solution for the indentation problem, including Croasdale et al 1977, Rojansky & Gerwick 1981, Morgenstern and Nuttal 1977, and, Ralston 1977. The following are the Indentation factors suggested by the various researchers:

*CROASDALE ET AL*

Croasdale et al (1977) obtained the following upper bounds for a rectangular
indentor shown in Fig. 3.2 by assuming the ice to be a perfectly plastic Teresca solid:

\[ I = 1.0 \quad \text{for} \quad D/t = \infty \]  \hspace{1cm} (3.6)

\[ I = 1.0 + \frac{\pi}{2} = 2.57 \quad \text{for} \quad D/t = 0 \]  \hspace{1cm} (3.7)

Rough Contact

\[ I = 1.45 + \frac{0.35}{D/t} \leq 2.57 \]  \hspace{1cm} (3.8)

Smooth Contact

\[ I = 1.15 + \frac{0.37}{D/t} \]  \hspace{1cm} (3.9)

ROJANSKY AND GERWICK (1981)

\[ I = \frac{\cos \theta}{2 \sin^3 \theta} \quad \text{with} \quad 2D/t = \frac{\sin^3 \theta}{\cos 2 \theta} \]  \hspace{1cm} (3.10)

refer to Fig. 3.3 for definition of the terms.

RALSTON (1977)

Rough Contact

\[ I = 1.67 + \frac{0.43}{D/t} \leq 2.97 \]  \hspace{1cm} (3.11)
Smooth Contact

\[ I = 1.15 + \frac{0.37}{D/t} \]

(TOSSAINT)

For Prandtl's Yield Criteria

\[ I = 1.0 + \frac{\pi}{2.0} = 2.57 \]  \hspace{1cm} (3.13)

For Mises Type Material

\[ I = 1.15(1.0 + \frac{\pi}{2.0}) = 2.97 \]  \hspace{1cm} (3.14)

For purposes of design it is recommended that the indentation factor developed by Ralston (eqs. 3.11 & 3.12) be used.

3.1.2.2. Indentation Factor for Columnar Ice

The above-mentioned expressions for the indentation pressure have been developed for the Granular Ice. The granular ice is considered to be isotropic, unlike the Columnar ice which is highly anisotropic. Assur (1971) pointed out that in columnar ice the inclined flaking slip plane would have to form diagonally across the columnar formation of ice crystals and hence will experience more resistance. Therefore for the columnar ice it is important to consider the anisotropic yield criteria. Ralston (1978) developed upper and lower bound solutions for indentation of a columnar sheet of ice using Prandtl's yield criteria. Fig. 3.4 shows the Indentation Factor developed by Ralston.

Michel and Toussaint (1977) also developed Indentation Factors for the
columnar ice. They defined the indentation factor based on the strain rate as follows:

**Brittle Zone**

\[ I = 2.97 \] (3.15)

for

\[ 10^{-8} \text{ s}^{-1} < \dot{\varepsilon} < 10^{-4} \text{ s}^{-1} \]

where

\[ C_x = \left[ \frac{\dot{\varepsilon}}{\dot{\varepsilon}_0} \right]^{0.32} \sigma_0 \]

\( C_x \) is the empirical strength based on experimental results.

\( \sigma_0 \) is the uniaxial strength at strain rate \( \dot{\varepsilon} \)

\( \dot{\varepsilon} \) is the strain rate.

**Ductile Zone**

\[ I = 1.57 \] (3.16)

for \( \dot{\varepsilon} > 10^{-2} \text{ s}^{-1} \)

where, \( C_x = \sigma_0 \)

**Transition Zone**

\[ I = 2.97 \] (3.17)
for \(0.5 \times 10^{-4}s^{-1} \leq \dot{\varepsilon} \leq 10^{-2}s^{-1}\)

where, \(C_x = \left( \frac{\dot{\varepsilon}}{\dot{\varepsilon}_0} \right)^{-0.126}\)

Ralston and Reinieke (1978) used plasticity theory and considering ice to be anisotropic plastic solid with a parabolic yield criteria (Figure 3.5), i.e. the ice is stronger in compression than in tension, obtained upper and lower bounds for the Indentation Factor, see in Fig. 3.4.

Croteau (1983) compared the various models mentioned above with the experimental results of Michel (1978) and adopted the following relationship (Fig. 3.6) for Indentation Factor:

\[ I = 3.0 + \frac{0.80}{D/t} \leq 4.5 \]  
\[ (3.18) \]

\[ I = 1.2 + \frac{0.32}{D/t} \leq 3.0 \]  
\[ (3.19) \]

The above given indentation factors developed by Croteau (eqn. 3.18 and 3.19) are recommended for use in design since they were developed after comparing various proposed indentation factors. However, the mode of failure and actual confinement conditions should be carefully considered while adopting an Indentation Factor. Particular attention should be paid to the value of aspect ratio \((D/t)\), since an error in selecting the proper values for the aspect ratio can lead to a very large change in the value of indentation factor.
3.1.3. Contact Factor

The contact factor is introduced to the equation to account for stress concentration due to incomplete contact. The microcracking and fracturing of the ice at its interface with the indentor causes irregularities which produce variation in local stresses along the interaction width. Korzhavin (1962) suggests that the contact factor is temperature dependent, whereas Michel and Toussaint (1977) purport the contact factor to be dependent upon the strain rate. They defined the contact factor to be the ratio of the residual force and the peak force, the initial peak force being due to the first perfect contact.

The following values were suggested for contact factor:

*Pure Ductile Failure*

\[ 10^{-8} \text{s}^{-1} < \dot{\varepsilon} < 5 \times 10^{-4} \text{s}^{-1} \quad f_c = 1.0 \] (3.20)

*Transition Zone*

\[ 5 \times 10^{-4} \text{s}^{-1} < \dot{\varepsilon} < 10^{-2} \text{s}^{-1} \quad f_c = 0.25 \] (3.21)

*Brittle Failure*

\[ \dot{\varepsilon} > 10^{-2} \text{s}^{-1} \quad f_c = 0.30 \] (3.22)

Ralston (1979) suggested higher values than those recommended by Toussaint and Michel; these are shown in Fig. 3.7. Bruen et al (1982) state that the contact factor is related to the temperature, and they have presented a
relationship for the strain rate and temperature (Fig. 3.8) based on the observations from Cook Inlet. According to their observations the ice forces increase with a rise in temperature, this is in contradiction with the indentation theory, since the ice strength is said to be greater at lower temperatures. The apparent change in observed forces is explained in terms of the brittle behavior of the ice. At lower temperatures the ice behaves in a brittle fashion causing peak loads to develop non-simultaneously over the interaction width, (a contact factor less than 1.0 would account for this behavior). On the other hand, at higher temperatures the ice will behave in a ductile manner (figure 3.9). Because ductile type failures are thought to have a more uniform stress distribution, a contact factor close to 1.0 is considered appropriate.

Kry (1978) claims that the contact factor should also be related to the interaction width. Kry (1978) presented a method of calculating the ice pressures on a stochastic basis, by postulating that the interaction width can be divided into a number of zones and the effective pressure on the entire width is the average of the simultaneous pressures for each zone. Hence, the effective pressure on the entire width is less than the peak zonal pressures for a given ice movement. A log normal distribution of probability for the zonal peak pressures was adopted by Kry, from which the design pressure is calculated based on the acceptable risk for the structure. Fig. 3.10 illustrates this concept. The contact factor then is the ratio of the stress for a single zone to the stress value for a selected number of zones or the entire width.
3.2. Meso - Scale look at Ice Behavior

The above presented theories for computing ice pressures under crushing conditions are widely accepted, though field measurements have proven that the actual ice pressures are markedly lower than the predicted values. We present here meso-scale (meso being intermediate scale between large and micro scale) look at ice behavior. The arguments presented here are not yet backed by field data: they are meant to present the concepts only.

The ice forces are presently calculated by a modified Krozavin’s equation

\[ F = \sigma_u \cdot I_f \cdot c \cdot \text{Area} \] (3.23)

In this equation the effective ice pressure is considered to be higher that the uniaxial strength of ice, as measured in the laboratory by testing small scale measurements. The ice strength is modified by using the Indentation factor to account for the confining effects in the large scale ice feature. The indentation factor is a semi-empirical relationship based on small scale lab and field tests. The indentation factor is generally defined as a function of the aspect ratio of the indentation member. The other factor in the equation is the contact factor, which accounts for the fact that the entire ice face does not experience the same pressure at the same time, i.e. non-simultaneous failure of ice over the contact face. Figure 3.11 shows schematically the ice pressures on the face of an ice piece being indented. As can be seen the high pressures are limited to only some zones. Therefore it is argued that the confining effect on ice is limited to these zones only and are not present over the entire face of the ice. It is therefore logical to recommend that the indentation factor should be computed for the high pressure zones only and not for the entire ice face. Actual field measurements will be necessary to assess the extent of the high pressure zones.
Figure 3.1 Physical model of indentation process: [1] Indentor, [2] Zone of plastic flow, [3] Zone of ductile deformation with micro-cracks, [4] Zone of deformation without cracks (Michel and Toussaint, 1977)
Figure 3.2 Indentation factors (Crossdale et al, 1977)
Figure 3.3 Assumed slip planes for crushing model
( Rojansky & Gerwick, 1981 )
Figure 3.4 Indentation Factors (Ralston, 1978)
Figure 3.5 Parabolic Yield Criteria for Ice (after Ralston & Reinieke 1978)
1.0

Ductile Crushing

$$C_I = 3.0 + \frac{0.80}{D/t}, \leq 4.5$$

Transition and Brittle Crushing

$$C_I = 1.2 + \frac{0.32}{D/t}, \leq 3.0$$

Lower-bound Solution

Figure 3.6 Indentation Factors (Croteau, 1983)

---

Figure 3.7 Contact factor as a function of strain rate (Ralston, 1976)
Figure 3.8 Contact factor (Breun et al 1982)
Figure 3.9 Pressure distribution for cold and warm ice (Bruen et al, 1982)
ICE MOVEMENT

TOTAL ZONES

ICE MOVEMENT

a) illustration of the concept

b) typical force penetration variation for a single zone

c) influence of the number of zones on the maximum lobal stress level for different probabilities of occurrence P (Kry, 1978 & 1980)

Figure 3.10 Failure zones in crushing (Kry, 1978)
Ice Floe

Structure

Ice Pressure Along Plane A-A

\( P_U \) is uniaxial strength of ice

\( D_t \) is typical dimension of zone where ice is stressed beyond \( P_U \)

Figure 3.11 Meso-scale look at Ice Pressures
4. Loads on Structures Due to Ridges

Pressure Ridges represent a severe condition of loading for Arctic structures. Pressure ridges are formed as a result of ice sheets cracking and broken pieces of ice being pushed up to form piles of ice. As these pieces continue to accumulate, the ridge grows both above and below water. Subsequent melting and refreezing of ice causes the ridge to become stronger. A first year ridge consists of thick sections of ice embedded in an ice sheet with relatively unconsolidated core. By definition, multiyear ridges are those ridges that have survived at least one melt season. Cores in multiyear ridges are consolidated. During summer the ice melts and fills the voids in the ridge with water which subsequently refreezes, consolidating the core.

Unlike the sloping sided structures, where the ridge fails in flexure (out of plane), (as shown in section 5) for vertical sided structures the failure of the ridge is either in-plane flexure, pure crushing or a shear failure.

4.1. Idealized Ridge Shape

Kovacs (1971) proposed an idealized geometry for consolidated first year ridge (figure 4.1) based on observed profiles of ridges in the Beaufort Sea, where $S$, the sail height, is the vertical distance between waterline and the top of the ridge, and the keel height $K$ is the distance between the water line and the lowest part of the ridge.

For the multiyear ridges Kovacs proposed the following factors:

\[ K = 3.3 \times S \]

\[ B = 5.5 \times K \]

Sail angle $= 20^\circ$
Keel angle = 30°

Figure 4.2 shows an idealized profile for a multi-year ridge (after Prodanovic, 1981)

The keel depth is the principal characteristic, which is selected for a particular site; and the other parameters are derived from the above given relations.

The cores of first year ridges are considered to be of thickness equal to the thickness of the parent ice sheet.

4.2. Forces Due to Pressure Ridges

As stated earlier, for vertical sided structures the failure of a ridge could be either in-plane flexure failure, shear failure or crushing. These modes of failure require different analytic techniques which are discussed in the following sections.

4.2.1. Flexure Failure

Observations at Cook Inlet (Blenkarn 1970) and the Baltic Sea have shown that for a pressure ridge impinging against a vertical sided structure the failure mode is not necessarily pure crushing, but rather, a multimodal type failure (Rojansky & Gerwick, 1981). At Cook Inlet, flexure failure was in fact the most frequent mode of failure (Blenkarn, 1970).

The behavior of a ridge under in-plane flexure forces is analyzed in a manner similar to the classical beam on elastic foundation problem, figure 4.3. For an out of plane failure the foundation modulus corresponds to the buoyancy characteristics of the ice ridge, whereas for an in-plane failure the foundation modulus is associated with the sheet ice in which the ridge is embedded.
Rojansky & Gerwick (1981) describe the flexure failure mode analysis as follows:

After the initial contact the force exerted on the structure by the ridge continues to increase until this force is enough to cause a flexure failure (assuming that the structure does not fail during the interaction, and that the flexure failure is the only likely mode of failure for the ridge). This first crack is called the Initial Crack, see figure 4.4. The point of failure in the ridge is dependent upon the length of the ridge, the position of interaction, and the stiffness of the parent ice sheet. After the initial crack has formed, the driving force attempts to clear the cracked ridge around the structure and as a result, the forces again begin to build up until it finally is large enough to again cause the ridge to fail in flexure at the point of maximum moment. The ridge should ideally develop two Hinge Cracks which will enable it to clear the cracked ridge.

In order to analyze the force required to produce a flexure failure, the ridge is considered to be a beam of finite length on elastic foundation (Fig. 4.5). The ice sheet in which the ridge is embedded is the foundation of the beam. Due to the restraint offered by the parent ice sheet, the beam is assumed to be fixed at the supports. For this condition of loading the maximum positive moment is at the point of interaction and is given by,

\[ M^* = \frac{P}{4\lambda} \frac{\cosh \lambda L - \cos \lambda L}{\sinh \lambda L + \sin \lambda L} \]  

(4.1)

The location of the maximum negative moment varies with the length of the ridge, for short ridges it occurs at the supports, and is given by the following expression:
\[ M^* = -\frac{P \sinh \lambda L/2 - \sin \lambda L/2}{\lambda \sinh \lambda L + \sin \lambda L} \]  

where,

P is the interaction force

L is the length of the ridge

\( \lambda \) is the damping coefficient (also called the characteristic length)

\[ \lambda = 4\sqrt{\frac{K}{4EI}} \]  

where,

K is the foundation modulus

E is the Young's elastic modulus of ice

I is the sectional moment of inertia of the ridge

The moment capacity of the ridge is obtained by assuming an elastic stress distribution through the thickness of the ridge and is expressed as:

\[ M_{\text{max}} = \frac{\sigma_f l}{C} \]  

where,

\( \sigma_f \) is the flexure strength of ice and C is the distance to extreme fiber of the ridge from the centroid.

The failure force P is then computed from equations 4.1 & 4.2. This gives the force corresponding to the formation of the Initial Crack.
Analysis of the hinge crack is done similarly by considering two beams on an elastic foundation, with loading as shown in Fig. 4.6. The maximum negative moment corresponding to this loading case is given by the following equation:

\[
M^* = \frac{P'}{2\lambda} \frac{2\sinh \lambda L/2 \cos \lambda L/2 + \cosh \lambda L/2 \sin \lambda L/2 (1 + 2 \cos \lambda L)}{\cosh \lambda L + \cos \lambda L + 2}
\]  

(4.5)

Here, again, the corresponding force can be computed by substituting the moment capacity of the section at the point of maximum moment in the equation 4.5. The higher of the two forces, (Initial Cracking force and Hinge Cracking force) is adopted for design purposes.

In the above-given analysis, the foundation modulus corresponds to the in-plane stiffness of the ice parent ice sheet, since the ice ridge is assumed to be supported by the sheet in which it is embedded. There is no general agreement in the published literature as to what value for the stiffness should be adopted. Rojansky and Gerwick (1981) adopted shear strength of the ice as the foundation modulus, i.e. \( K = \sigma / 2 \) (\( \sigma \) is the uniaxial compressive strength of the ice sheet).

4.2.2. Effect of Partially Consolidated Ridges

Most of the ridges, both first year and multiyear, do not have an isotropic section but rather, have cores which are relatively more consolidated than the keel or the sail. To account for this property of ridges, Rojansky and Gerwick (1981), proposed using a transformed section: this concept is similar to the transformed section in reinforced concrete.

Using Griffith’s Theory of Brittle Fracture the flexure strength can be
written as:

\[ \sigma^*_{f} = \sqrt{\frac{2\gamma_c E}{\pi(1-\mu^2)}} \]  

(4.6)

where \( \gamma_c \) is the critical strain rate energy released, \( C \) is the crack length, and \( \mu \) is the Poisson's Ratio.

Thus, the flexure strength is proportional to the square root of the elastic modulus. Hence:

\[ \frac{\sigma^*_{fu}}{\sigma^*_{fc}} = \sqrt{\frac{E_u}{E_c}} \]  

(4.7)

The transformed section of the unconsolidated section can be written as

\[ \sigma^*_{fu} = \sigma^*_{fc} \sqrt{\frac{E_u}{E_c}} \]  

(4.8)

\[ \sigma^*_{fu} Z_u = \sigma^*_{fc} Z_c \]  

(4.9)

where \( Z_c \) and \( Z_u \) are the section modulus of the consolidated and unconsolidated sections respectively.

4.2.3. Ridge Shearing

4.2.3.1. Multiyear Consolidated Ridges

Prodanovic (1981), developed an analysis technique for computing forces due to ridges failing in crushing. His analysis accepts the idealized shape of the ridge as given by Kovacs (1971) (Fig. 4.1), and the strength of the ice is expressed by Von Mises yield function for an anisotropic material modified for isotropy in the xy plane (Reinicke and Ralston, 1978). The yield function
\[ f(\sigma) = a_1 \left[ (\sigma_y - \sigma_z)^2 + (\sigma_z - \sigma_x)^2 \right] + a_3 (\sigma_x - \rho y)^2 + a_4 (\tau_{yx}^2 + \tau_{zx}^2) + a_6 \tau_{xy}^2 + a_7 (\sigma_x + \sigma_y + \sigma_z) - 1 \] (4.10)

with \( a_6 = 2( a_1 + 2 a_3 ) \), where \( a_1 \), \( a_3 \), \( a_4 \), and \( a_7 \) can be determined from measured ice properties. Four test results are required to define the function. Simple tensile strength, compression test along with Frederking type A and confined compressive strength tests are enough to define the yield function. See Fig. 4.7 for an example of an anisotropic yield function for ice.

Unlike an ice sheet, the ridge does not possess global strength anisotropy, hence the yield function for the ridge can be approximated by including another plane of isotropy in the yield function. Originally, this 3-parameter yield surface was proposed by Reinicke and Remer (1978) for modelling granular ice.

\[ f(\sigma) = a_1' \left[ (\sigma_y - \sigma_z)^2 + (\sigma_z - \sigma_x)^2 + 6\tau_{xy}^2 + \tau_{yx}^2 + \tau_{zy}^2 \right] + a_7' (\sigma_x + \sigma_y + \sigma_z) - 1 \] (4.11)

where \( a_1' \) and \( a_7' \) can be obtained by using two known strength values.

4.2.3.2. Failure Mechanism

Observations from Cook Inlet (Blenkarn 1970) indicate that the in-plane crushing is not likely to be the mode of failure: instead the ridge will fail in shear and the adjoining ice sheet will crush. Figure 4.8 shows the failure mechanism proposed by Prodanovic (1981). Here the ridge is shown to shear along a plane at an angle \( \psi_o \). This type of failure is called a plug type failure. After formation the plug interacts with the sheet behind the ridge as a flat indentor: as the interaction continues the sheet fails in crushing. A no slip
condition exists between the structure-ridge interface and a freeslip (no shear) condition is assumed between the interface of the sheet and the plug.

Maximum force occurs (figure 4.8) when the ridge penetrates enough to allow the structure width, $D$, to interact in its entirety with the ridge. For flat indentors normal to the ice flow, the maximum force is reached when the structure starts to penetrate, whereas for narrow cylinders the peak is reached when the cylinder is half penetrated.

Using the Upper Bound Theorem of Plasticity, Prodanovic (1981) was able to get the total shearing force by equating the total energy dissipated to the external work done as follows:

$$ F = 2A_o(C_1 \tan \psi_o + C_2 \cot \psi_o) + T D_1 \left[ \frac{g_1}{\sin \phi} + \frac{T}{2D_1} \left( \frac{g_1 \tan \phi_0}{\sin \phi \tan \phi} + \frac{g_2}{\tan \phi \cos \phi} \right) \right] $$

(4.12)

where,

$$ C_1 = \frac{1}{3} a_\gamma + \frac{a_\gamma}{6a_1} $$

(4.13)

$$ C_2 = \frac{a_\gamma}{8a_1} $$

(4.14)

$A_o$ (area of consolidated ridge) = 4.02 $K^2 + 0.38 KT - 1.69 T^2$

$$ D_1 = D + 2 B_0 \tan \tau_o $$

$$ B_0 = 5.5K + 3.12T $$

(4.15)

4.2.4. Ridge Crushing

A ridge crushing model is shown in Fig. 4.9. The logarithmic spiral-shaped discontinuous surfaces are adopted to find the upper bound crushing
forces for the ridges. Ralston (1978) and Prodanovic (1978) used the following equation to define the crushing of sheet ice:

\[
F = \frac{D}{2\sin 2\psi_o} (C_1 \tan \phi + C_2 \cos \phi) \int_{\psi_o}^{\pi} \exp(Z(\psi - \psi_o)) \tan \phi \, dz \, d\psi
\]  

(4.16)

in which \( F \) is the crushing force. The integration is performed between the limits corresponding to the thickness of the consolidated part of the ridge.

In the actual analysis of the forces required to fail the ridges the following steps are involved:

1. Define the ridge geometry
2. Define the yield function of the ice sheet and the ridge
3. Assume a penetration distance and find the crushing and shearing force by optimizing the values of \( \psi_o \) and \( \phi \). The minimum of the two forces is then the design force for that particular feature. Figure 4.10 shows the plot of shear and crushing forces for ridges with various aspect ratios.

4.2.4.1 First Year Unconsolidated Ridges

As explained earlier, the major portion of the first year ridges is composed of ice rubble in an unconsolidated form. The core is the only portion of the ridge which is relatively consolidated. The idealized shape of the ridge is shown in Fig. 4.1. The maximum force exerted by first year ridges is the summation of the forces imposed by the consolidated section of the ridge and the force due to the unconsolidated rubble.

\[
F = f_s + f_R
\]  

(4.17)

According to Prodanovic (1981), the consolidated zone fails in crushing.
Following this logic, the force from this section can be computed by the standard crushing analysis for sheet ice.

Failure mechanisms for the rubble (unconsolidated keel and sail) are of two types: a gate type crushing failure, or a plug type shearing failure. For larger structures, the shear failure mechanism (plug type) is most likely to occur. This mechanism is the same as for the multiyear ridge (Figure 4.8). The forces from such a failure are also computed in the same manner as for the multiyear ridges.

For narrow structures the failure mechanism is a crushing failure (gate type). The crushing surface is defined by Prodanovic (1981) as the classical Prandtl Velocity Field - a logarithmic spiral. Again, this analysis technique is the same as that defined earlier for a multiyear ridge.

The yield functions for rubble piles are different than for consolidated ridges. Model test data on the strength of rubble piles reported by Prodanovic (1981) and Weiss et al (1981) indicates that the rubble pile, which constitutes the major portion of the first year ridge, can be considered to be isotropic. In order to determine the strength of the rubble pile Prodanovic assumed it behaved as a linear cohesive Mohr-Coloumb material with the following yield criteria,

\[ \tau = C + \sigma \tan \phi \]  \hspace{1cm} (4.18)

where, \( \sigma \) = normal stress

\( C \) = effective cohesion

and, \( \phi \) = internal angle of friction
Using the extrapolation of the Mohr-Coloumb yield function by Drucker and Parger (1952), the upper bound rubble pile forces were obtained. The upper bound shear and crushing failure forces for the rubble are:

**Shearing Forces**

\[ F_S = 2A_0c \]  \hspace{1cm} (4.19)

where \( A_0 \) is the cross-sectional area of the ridge and sail, and \( c \) is the cohesion.

**Crushing Forces**

\[ F_C = \frac{CD}{\sin 2\psi_o} \int \int EXP2(\psi - \psi_o) \tan \phi \, dz \, d\psi \]  \hspace{1cm} (4.20)

Once again, as for multiyear ridges, the force can be computed by assuming a penetration distance for the ridge and evaluating the shearing and crushing forces. The failure force corresponds to the lower of the two, figure 4.11 shows the rubble crushing and rubble shearing forces.

Croasdale (1978) proposed a different model for an unconsolidated ridge by assuming that the unconsolidated ridge is essentially made up of ice blocks, held together by buoyancy, gravity and friction. Referring to Fig. 4.12 the intergranular force is given by:

\[ f_y = (\rho_o - \rho)(\gamma_o - \gamma) \]  \hspace{1cm} (4.21)

\[ F = 2B \frac{l^2}{3} (\rho_o - \rho)g \tan \phi \]  \hspace{1cm} (4.22)
where, \( B \) is the ridge width,

\( t \) is the ridge thickness,

\( (\rho_w - \rho) \) is the buoyant density of ice, and

\( \phi \) is the internal angle of friction of submerged blocks of ice.

If the ridge is partially consolidated then:

\[
F = \frac{1}{3}(\rho_w - \rho)Bt^2\tan\phi + \frac{1}{2}cBt
\]  

(4.24)

4.3. Comparison of Various Models

Solutions to Prodanovic's equations have been presented in nondimensional form in Fig. 4.10 & 4.11. The figure gives the forces required to fail a ridge given certain dimensionless parameters of the ridge geometry. The failure modes considered by Prodanovic are only crushing and shearing. Since the length of the ridge is not considered to be a variable, the forces given are independent of the ridge length.

In figure 4.10 the failure mode for \( D/t \) values less than 10 is always crushing and for \( D/t \) greater than 40 it is always shearing. Within the range \( 10 < d/t < 30 \) the failure mechanism is determined by the ratio of the keel depth and the ice sheet thickness, \( K/t \). It is interesting to note that most structures are designed with a \( d/t > 50 \) (\( D > 300 \) ft. and \( t = 6 \) ft), hence, for most structures the ridges are likely to fail in shearing. The crushing forces are about 2-3 times the shearing forces. (As the ice ridge interacts with the structure the crushing forces develop at the interface and continue to build up, until the ridge fails in shear. Therefore the maximum force is limited by the shear failure mechanism.)
There is no field data available for the verification of the above-mentioned model. Thus, so far, the above-mentioned analysis techniques are the only means available to computed forces from ridges.

To calculate the forces from ridges Prodanovic’s plots shown in Fig. 4.10 & 4.11 can be used. For first year ridges the force obtained from Fig. 4.11, gives the rubble force only, the force due to the consolidated part should be added to this to get the net force.

Prodanovic’s model does not include the bending mode of failure. Very long ridges may fail in bending. It should be noted here that in the event the ridge fails in bending (eg. in the case of a conical structure) then the failure force is highly dependent upon the length of the ridge. The shorter the ridge length the larger the theoretical failure force. This concept is further explored in chapter 5.

The maximum force exerted by ridges thus is the minimum of the the failure force for the three modes of failure: crushing, shear and bending.
Figure 4.1 Consolidated Ridge Model
(Pradanovic 1981)

Figure 4.2 Unconsolidated Ridge Model
(Kovacs, 1971)
Figure 4.3 In-plane Flexure Failure Model for Ice Ridge
Figure 4.4 Failure Mechanism for Ice Ridge
Figure 4.5 Ridge flexure failure - initial crack formation (Rojansky, 1981)

Figure 4.6 Ridge flexure failure - hinge crack formation (Rojansky, 1981)
Figure 4.7 Anisotropic yield function for sea ice (Ralston, 1978)
Figure 4.8 Consolidated ridge shearing mechanism (Pradanovic, 1981)
Figure 4.9 First year ridge crushing mechanism
(Pradanovic, 1981)
Figure 4.10 Consolidated Ridge Force (Pradanovic, 1981)

\[ F = F_c + F_r \]

- **\( F \)** - Rubble Force
- **\( F_c \)** - Force Due to Consolidated Section
- **\( C \)** - Effective Cohesion (~5-10 psi)
- **\( D \)** - Structure Diameter
- **\( K \)** - Keel Height

\[ \sigma_c \] - Unconfined Compressive Strength

**D** - Structure Diameter

**T** - Thickness of Parent Ice Sheet

\( K \) - Keel Depth (Fig. 4.9)

Figure 4.11 Rubble Pile Force (Pradanovic, 1981)
Figure 4.12 Unconsolidated ridge bearing
(Croasdale, 1978)
5. Ice Forces on Sloping Structures

5.1. Introduction

As the ice sheet approaches a sloping structure such as a cone, at first contact the ice begins to crush at the interface and an interaction force is generated. Assuming that unlimited driving force is available, the ice continues to crush locally and the magnitude of the force increases. Figure 5.1 shows the interaction forces between an ice sheet and a structure with sloping sides. The interaction force, acting normal to the face of the structure, has a vertical and a horizontal component. The vertical component produces bending stresses in the ice sheet. The ice force continues to increase until the vertical component of the force is large enough to cause the ice to fail in bending. After its failure the smaller pieces of ice are pushed by the approaching ice sheet and begin to ride up the face of the structure. Subsequent to the first ride up, a larger force is generated since additional force is required to push the broken pieces of ice up the slope of the structure. During such an interaction the structure experiences both Horizontal and Vertical ice forces. The following sections discuss in detail the various methods for analyzing this interaction.

5.2. Two Dimensional Analysis

Croasdale (1978) presented a two dimensional analysis model for the sloping structure-ice interaction. The interaction forces as shown in Figure 5.1 are as follows:

\[ H = N \sin \alpha + \mu N \cos \alpha \]  

(5.1)
\[ V = N \cos \alpha + \mu N \sin \alpha \]  \hspace{1cm} (5.2)

\[ N = \frac{H}{\sin \alpha + \mu \cos \alpha} \]  \hspace{1cm} (5.3)

\[ V = H \left[ \frac{\cos \alpha - \mu \sin \alpha}{\sin \alpha + \mu \cos \alpha} \right] \]  \hspace{1cm} (5.4)

or \( H = V \left[ \frac{\sin \alpha + \mu \cos \alpha}{\cos \alpha - \mu \sin \alpha} \right] \)  \hspace{1cm} (5.5)

The next step in the analysis is to determine the maximum value of \( V \), the vertical force, that can be generated for a particular ice feature. The vertical force is limited by the bending capacity of the ice feature, \( M_0 \). Most investigators do not measure the moment capacity directly, but instead relate the extreme fiber tensile strength to the moment capacity (Ralston, 1977). Assuming an elastic stress distribution through the thickness of the ice feature and also ignoring any variations in the elastic modulus, the moment capacity can be written in terms of the tensile strength as:

\[ M_0 = \sigma_f b t^2 / 6 \]  \hspace{1cm} (5.6)

For a beam on an elastic foundation it can be shown (Hetenyi, 1947) that the maximum bending capacity \( M_0 \) due to edge load \( V \) is given by:

\[ M_0 = \frac{V}{\beta e^{\pi/4}} \sin \pi/4 \]  \hspace{1cm} (5.7)

where \( 1/ \beta \) is the characteristic length of the beam (ice sheet in this case) defined by,

\[ \beta = \left[ \frac{K}{4Et} \right]^{1/4} \]  \hspace{1cm} (5.8)
$K$ is the foundation constant given by $(\rho g b)$ for a floating beam,

$\rho_w$ is the density of sea water,

$E$ is the modulus of ice,

$I$ is the moment of inertia of ice feature,

$t$ is the thickness of ice feature.

From the equations 5.6, 5.7 & 5.8, the vertical force required to cause failure of the ice sheet can be written as:

$$V = \frac{M_0 \beta e^{\pi/4}}{\sin \pi/4}$$  \hspace{1cm} (5.9)

$$V = 0.68\sigma_f b \left[ \frac{\rho_w g t^3}{E} \right]$$

Now combining eq. 5.6 & 5.9, the horizontal force per unit width of the structure is given by,

$$\frac{H}{b} = 0.68\sigma_f \left[ \frac{\rho_w g t^3}{E} \right] \left[ \frac{\sin \alpha + \mu \cos \alpha}{\cos \alpha - \mu \sin \alpha} \right]$$  \hspace{1cm} (5.10)

$H$, given by EQ. 5.10, gives the horizontal force generated at the instance of the first failure of ice. Once the ice has failed, the broken pieces start to ride up the face of the structure. As a result of friction, an additional force is experienced by the structure. The force system for such a case is shown in Figure 5.2. It should be noted that this ride up force, also called the Clearing Force is not negligible. In fact, for ice breakers the clearing force is thought to
be greater than the breaking force.

As shown in Figure 5.2, \( P \), the force required to push the ice up the slope, can be quite easily written as:

\[
P = \left[ \frac{ztb}{\sin \alpha} \right] \rho g (\sin \alpha + \mu \cos \alpha) \tag{5.11}
\]

where \( z \) is the maximum height the ice can ride up to, (usually limited to the level of the throat of the cone),

and \( \rho_i \) is the unit weight of ice in air.

The corresponding total horizontal force experienced by the structure is,

\[
H = (V + P \sin \alpha) \left[ \frac{\sin \alpha + \mu \sin \alpha}{\cos \alpha - \mu \sin \alpha} \right] + P \cos \alpha \tag{5.12}
\]

substituting the value of \( P \) and \( V \) from equations 5.11 & 5.4 in eq. 5.12,

\[
\frac{H}{b} = C_1 \sigma_f \left[ \rho g \frac{t^5}{E} \right]^{1/4} + ztg \rho_i C_2 \tag{5.13}
\]

where

\[
C_1 = \left[ \frac{\sin \alpha + \mu \cos \alpha}{\cos \alpha - \mu \cos \alpha} \right] \tag{5.14}
\]

\[
C_2 = \left[ \frac{\cos \alpha + \mu \cos \alpha}{\cos \alpha - \mu \sin \alpha} \right] + \left[ \frac{\sin \alpha + \mu \cos \alpha}{\tan \alpha} \right]
\]

\( C_1 \) and \( C_2 \) are functions of slope angle and friction coefficient only and
have been plotted in figure (5.3 and 5.4)

The above given analysis is a two dimensional approximation of a three dimensional problem. For very wide structures, such as conical caissons, this analysis gives good results; for narrow structures this is not a good approximation, since the ice does not fail in a width equal to the width of the structure rather the failure is dispersed over a much wider area. (see figure 5.5). The 2-D analysis assumes the failure to be limited to a zone equal to the width of the structure.

Croasdale (1978) introduced a modified model for narrow structures. In this corrected model, correction factor $C_f$ is introduced. $C_f$ is the ratio of the length of the circumferential crack and the structure diameter. Afanasev (1971) proposed the following expression for the characteristic length of the ice sheet:

$$ l = \left[ \frac{E l^3}{12 \rho g (1-\nu^2)} \right]^{1/4} \quad (5.15) $$

The circumferential length of the crack is derived from the characteristic length by the following relationship:

$$ l' = 0.25 \pi^2 l $$

hence

$$ C_f = \frac{\left[ \frac{E l^3}{12 \rho g (1-\nu^2)} \right]^{1/4}}{0.25 \pi^2} \quad (5.16) $$
where $\nu$ is Poisson's ratio,

and $W$ is the width of the structure.

Croasdale (1978) reports some comparisons of the modified 2-D model with other models. The results are reproduced in Table 5.1 where they demonstrate rather close agreement.

5.3. 3-D Analysis

For a narrow structure, the failure zone of the ice is extended beyond the width of the structure. Not all broken pieces of ice ride up the slope of the structure, as some pieces may clear around without even touching the structure. As stated above, two dimensional approximation is not realistic. Ralston (1977) presented a method for analyzing the problem in three dimensions. Ralston considered the failure of an advancing ice sheet against a conical structure and used 3-D plate theory instead of simple beam bending theory. In order to compute the forces, Plastic Limit State analysis was employed.

In the Plastic Limit State analysis the rate of work done by external forces is equated to the rate of energy dissipation. The energy dissipation sources being:

1. Formation of plastic hinges in the ice along circumference and side of the structure
2. Work done against the foundation reaction
3. Continuous deformation of ice
4. Frictional dissipation

Assuming elastic stress distribution through the thickness of the ice, the bend-
ing moment capacity of ice was derived. Ralston performed a closed form integration of energy dissipation equation. His results are as follows: form:

\[ H = \left[ A_1 \sigma_f t^2 + A_2 \rho_w gt D^2 + A_3 \rho_w gt (D^2 - D^2_T) \right] A_4 \] (5.17)

\[ V = B_1 H + B_2 \rho_w gt (D^2 - D^2_T) \] (5.18)

where, \( H \) is the horizontal force,

\( V \) is the Vertical force,

\( \rho_w \) is density of sea water,

\( \sigma_f \) is the flexure strength of ice,

\( D \) is the waterline diameter of the structure,

\( D_T \) is the top diameter of the structure (throat),

\( t \) is the thickness of ice,

and \( \mu \) is the friction coefficient for ice and structure,

The coefficients \( A_1, A_2, A_3, A_4, B_1 \) & \( B_2 \) are given in Figure 5.6.

In equation 5.17 the first term is due to ice strength, i.e. the force required to break the ice feature, the second is for buoyancy and the third is for clearing.
5.4. Empirical Models

Edwards and Croasdale (1976), conducted model tests to determine the ice forces in ARCTEC model basin on a 45° cone, up to 100 cm. in diameter with ice of 7 cm. thickness. They derived the following expression from their model tests:

\[ H = 1.6\sigma_I t^2 + 6.0\rho g Dt^2 \]  \hspace{1cm} (5.19)

This equation is valid for cones with a 45° slope with friction coefficient \( \mu = 0.5 \).

The first term in this expression is the contribution of the ice breaking force and the second is for the friction or clearing force. Afanasev et al (1971) conducted tests on a cone having a 28 cm. diameter and an ice thickness of 3.5 cm. Their empirical relation is as follows:

\[ H = \frac{\sigma_I t^2 S_x \tan \alpha}{1.93l} \]  \hspace{1cm} (5.20)

where \( S_x \) is the length of circumferential crack, given by:

\[ S_x = 1.76(r + \pi/4 - 1) \]

where \( r \) is the cone radius at the waterline,

\( l \) is the characteristic length, given by:

\[ l = \left[ \frac{E_I^3}{12\rho g (1-\nu^2)} \right]^{1/4} \]  \hspace{1cm} (5.21)
E is the elastic modulus of ice

and \( \nu \) is the Poissons Ratio.

5.5. Comparison of Various Models

5.5.1. Narrow Structures

Croasdale (1978), compared results obtained for a slender conical structure using the various models stated above. These results are reproduced in Table 5.2. It is clear from the results that 2-D model greatly under estimates the forces for narrow structures, whereas the modified 2-D model gives results fairly close to Ralston's 3-D model.

5.5.2. Wide Structure

Croasdale's study also compared various models; these results are summarized in Table 5.1. The comparison shows that the 2-D and 3-D models are in good agreement. The forces computed for the wide structures indicate that the ride-up forces are of a magnitude equal to the ice breaking forces.

It should be pointed out that the above mentioned models assume that there are no ice pieces existing around the structure in the form of rubble pile or adfrozen ice before the ridge interacts with the structure. The existence of rubble pile or adfreeze will change the scenario and may increase the forces.

As mentioned in section 2.2.7, adfreeze is an important element to be considered in design. The conical structures are able to reduce the ice forces because the failure mode for most ice features is changed from crushing to bending by the sloping face of the structure. However, if the ice is frozen
around the structure (also called the break-out condition) then the ice is not able to ride-up the slope to generate the bending stresses. In such a case the maximum force generated corresponds to the minimum of adfreeze bond failure force and the crushing failure of ice at the interface. Once the break-out has occurred and the ice is no longer fixed to the structure, the ice features can now ride-up the slope of the structure and fail in bending. Thus in design the adfrozen condition should be considered in addition to the regular global failure force (refer to section 5.6.1 for details of adfreeze forces).

5.6. Ridge Loads on Sloping Sided Structures

Multiyear ridges are frequent features in the deep waters of Beaufort Sea. These ridges represent a severe case of loading on offshore structures. It is quite likely that forces from multiyear ridges will govern the design of the structure.

Croasdale (1975), presented a model for finding the forces exerted by a ridge on a sloping sided structure. The first approximation in the analysis assumes the multiyear ridge to be an infinitely long floating beam. The beam is then analyzed using classical solutions for beams on elastic foundations. The beam itself is assumed to behave elastically too. Lewis and Croasdale (1978), did model tests and concluded that the failure mechanism for a ridge is two-fold. The ridge fails first at the center, i.e., at the point of contact with the structure and subsequently, hinge cracks form at the two sides, as shown in the Figure 5.7. The load is computed with the assumption that the initial failure is analogous to the loading of an infinitely long beam on an elastic foundation with a vertical load. The subsequent formation of hinge cracks are considered parallel to those which arise when a semi-infinite floating beam subjected to a vertical loading at one of the ends.
The vertical force required to cause the initial crack is given by:

\[ V_1 = \frac{4I\sigma_f}{y_l} \]  

(5.22)

I is the second moment of inertia of the ridge at the point of contact,

\( \sigma_f \) is the flexure strength of ice,

\( y_l \) is the distance of the neutral axis to the top surface of the ridge,

and \( l \) is the characteristic length given by:

\[ l = 4\sqrt{\frac{\sigma f}{\rho g b}} \]

where \( b \) is the width of ridge.

After initial crack formation there must be further breakage before the ridge can pass around the structure. Hinge cracks form to facilitate the ridge clearance around the structure. These hinge crack forces are evaluated by analysis of a semi-infinite beam on elastic foundation, with vertical loading as follows:

The vertical load at the end of the beam required to cause bending failure is:

\[ V_2 = \frac{6.17I\sigma_f}{yb^3} \]  

(5.23)

where \( y_b \) is the distance of the neutral axis to the bottom fiber of the ridge.

From the vertical force computed above for the initial and hinge cracks the horizontal force can be computed as follows:
\[ H_1 = \frac{V_1 (\sin \alpha + \mu \cos \alpha)}{(\cos \alpha - \mu \sin \alpha)} = V_1 C_1 \]  
(5.24)

\[ H_2 = \frac{V_2 (\sin \alpha + \mu \cos \alpha)}{(\cos \alpha - \mu \sin \alpha)} = V_2 C_1 \]  
(5.25)

The values of \( C_1 \) are given in Figure 5.3.

Ralston (1977), pointed out that the vertical force required to cause failure of a ridge, as given by equation 5.24, will increase with decreasing ridge length. Hence, for shorter ridges, higher force will be generated to cause failure. The shorter ridges are likely to produce higher forces because of the 3-D effects. The longer ridges are more likely to fail in bending, whereas shorter ones in shear.

5.6.1. Adfreeze Forces

In the event the ice freezes to the structure, the structure will experience forces due to adfreeze when the ice is pushed by either the ice pack or the wind. The horizontal component of the adfreeze forces can be computed by the following equation:

\[ H = \frac{\pi t q D}{\tan \alpha} \]  
(5.26)

where, \( t \) is ice thickness, \( \alpha \) is the slope angle of the cone, \( D \) is the structure diameter at water line, and \( q \) is the adfreeze bond strength.

Although very limited data on the adfreeze strength of ice is available, Croasdale (1978) reports that adfreeze strengths of ice have been measured in the range 20 to 150 psi. More adfreeze bond data is given in section 2.2.7.

The adfreeze forces should be computed for the break-out condition as mentioned in section 5.5.2.
5.7. Downward Breaking Cones

It is important to note that the above stated analysis is for upward breaking cones only. For downward breaking cones the situation is quite different. Forces acting on downward breaking cones can be computed using the models developed above after making modifications to account for the fact that the ice does not ride up the face and that the work is not done against gravity. The ice feature is pushed into the water instead of being lifted, hence, work is done to submerge the ice. As a result, the density term, i.e. $0.9 \rho_w g$, in Ralston's equation is replaced by buoyant density, i.e. $0.1 \rho_w g$ (or simply replace $\rho_w g$ by $\rho_w g/9$).
COMPARISON OF MODELS FOR
COMPUTING ICE FORCES ON CONICAL STRUCTURES

<table>
<thead>
<tr>
<th>Model</th>
<th>Ice Thickness (M)</th>
<th>Breaking Force (KN)</th>
<th>Rideup Force (KN)</th>
<th>Total Force (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ralston</td>
<td>0.5</td>
<td>1574</td>
<td>4385</td>
<td>5959</td>
</tr>
<tr>
<td>Simple 2D</td>
<td>0.5</td>
<td>559</td>
<td>5625</td>
<td>6184</td>
</tr>
<tr>
<td>Ralston</td>
<td>1.0</td>
<td>4822</td>
<td>8770</td>
<td>13590</td>
</tr>
<tr>
<td>Simple 2D</td>
<td>1.0</td>
<td>1330</td>
<td>11250</td>
<td>12580</td>
</tr>
<tr>
<td>Ralston</td>
<td>2.0</td>
<td>14855</td>
<td>17540</td>
<td>32395</td>
</tr>
<tr>
<td>Simple 2D</td>
<td>2.0</td>
<td>3165</td>
<td>22500</td>
<td>25662</td>
</tr>
</tbody>
</table>

Adfreeze forces are not included.

EXAMPLE DATA: Diameter of Cone at Water Line = 60.0 m., Cone, Slope Angle = 45°, Friction Coefficient \( \mu = 0.15 \), Freeboard = 10.0 m., Elastic Modulus = \( 7 \times 10^6 \) KPa, Flexure Strength \( \sigma_f = 700 \) KPa.

TABLE 5.1 Comparison of Models For Computing Ice Forces on Wide Conical Structures (Crossdale, 1978)
## COMPARISON OF VARIOUS MODELS FOR ICE FORCES ON CONICAL STRUCTURES

<table>
<thead>
<tr>
<th>Model</th>
<th>Breaking Force (KN)</th>
<th>Ride-Up Force (KN)</th>
<th>Total Force (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ralston $\mu = 0.15$</td>
<td>1964</td>
<td>30</td>
<td>1994</td>
</tr>
<tr>
<td>Edwards &amp; Crossdale</td>
<td>1384</td>
<td>150</td>
<td>1534</td>
</tr>
<tr>
<td>Simple 2D</td>
<td>84</td>
<td>70</td>
<td>154</td>
</tr>
<tr>
<td>Modified 2D</td>
<td>1200</td>
<td>70</td>
<td>1270</td>
</tr>
<tr>
<td>Afanasev et al</td>
<td>694</td>
<td>-</td>
<td>694</td>
</tr>
</tbody>
</table>

*Adfreeze forces are not included*

**EXAMPLE DATA:** Cone Diameter = 3.0 M., Ice Thickness = 1.0m., Free Board = 1.5 m., Slope Angle = 45°, Flexure Strength $\sigma_f = 1050$ kPa, Poissons Ratio $\nu = 0.33$, Elastic Modulus $E = 700 \times 10^6$ kPa

Table 5.2 Comparison of Models for Computing Ice Forces on Narrow Conical Structures (Crossdale, 1978)
Figure 5.1 Initial Interaction Between Ice and Sloping Structure (after Croasdale 1978)
Figure 5.2 Interaction Between Ice and Sloping Structure (after Croasdale 1978)

Figure 5.3 Coefficient $C_1$ as a Function of Slope Angle and Friction Coefficient (after Croasdale 1978)
Figure 5.4 Coefficient C2 as a Function of Slope Angle and Friction Coefficient (after Croasdale 1978)
Figure 5.5 A Three Dimensional Representation of the Ice Structure Interaction (after Ralston 1977)
Figure 5.6 Ice Force Coefficients for Plastic Analysis (after Ralston 1977)
Figure 5.7 Formation of Initial Crack and Final Hinge Crack During Ice Ridge Structure Interaction
6. Limited Driving Force Concept

6.1. Introduction

The ice force analyses presented earlier computed the ice forces on a structure by considering the failure of the ice feature. The design force from such an analysis is selected to correspond to a failure mode which requires the least force. In this analysis it is implicitly assumed that the far field ice exerts enough force on the impinging ice feature to enable it to fail, in other words the driving force is greater than the failure force. Croasdale and Marcellus (1981) were first to point out that the force required to fail an ice feature can be up to 30 times greater than the maximum available far field force. This concept of evaluating the environmental forces on the ice feature is called Limit Force concept, as opposed to Limit Stress concept of evaluating failure forces from an ice feature.

As an ice feature, traveling at relatively slow velocity, in winter, comes in contact with the structure, after first contact, the ice in the contact zone begins to get stressed due to the interaction force. This dissipates the energy of the floe in the form of strain energy. As a result of the energy dissipation, the ice feature comes to a stop in contact with the structure. Until this point, the maximum force experienced by the structure corresponds to the failure force of the ice feature.

The surrounding ice pack, water and air continue to move even after the ice feature has stopped in front of the structure. These elements of the environment continue to exert forces on the stopped ice feature, which eventually get transferred to the structure. The ability of the ice pack to transfer force to the ice feature is limited by the strength of the ice pack sheet (figures
6.1 & 6.2). Beyond a certain stress level the ice sheet fails and forms rubble. Thus the ice pack force can be assessed as the maximum far field force; in other words, the ridge building force.

After the formation of the rubble, the force transmitted to the structure is limited by the shear capacity of the rubble. This is also referred to as the pack ice drag force (see figure 6.2).

In a Limit Stress approach the forces are computed irrespective of the far field force. An ice feature, say a ridge, is assumed to interact with the structure and the minimum force to fail the feature is calculated. It is possible in summer events, that the ice feature has enough kinetic energy to develop failure forces, but, for winter events, the velocities are low and it is probable that the interaction force will be less than the driving force.

For an ice feature in the ocean the driving force is the sum of the current drag, the wind drag force and the far field average driving force at the edge of the ice feature. Current and wind drag are estimated relatively easily by using the existing equations, but the estimation of the far field force is a non-trivial problem.

The total driving force on the structure is as follows:

\[
F_{D,F} = F_{CURRENT\ DRAG} + F_{WIND\ DRAG} + F_{FAR\ FIELD} \tag{6.1}
\]

Now we will examine each of the three driving force elements.

6.2. Far Field Force

As stated earlier we are interested in the maximum force the far field ice pack can transmit. The maximum force transmittable through the ice pack is assessed by evaluating the force required to build some of the largest ridges
observed in nature. This indirect approach has been accepted in the wake of any better model for far field force. Since the ridges are formed by the driving forces, they reflect on the nature of the environmental driving forces. Two methods have been proposed in the literature to evaluate the driving force or the ridge building force (Vivatrat and Krieder, 1981).

1. Energy Conservation Approach

In this approach the force required to develop an observed ridge is evaluated by equating the work done to the create the ridge. The energy dissipation during the formation of a ridge is due to potential energy, frictional force and the strain energy of the ice sheet.

2. Sheet Failure Force Estimate Approach

In this approach the available far field force is estimated by considering the failure mechanism at the ice sheet/ice feature interface. From observed ride-ups and rubble pile data, the far field force is evaluated using models developed by various researchers - Shapiro, 1976; Kry, 1977; Kry, 1980; and Kovacs et al, 1979. Vivatrat (1981) presented a modified model for ridge formation; this model includes forces from the following:

a. potential energy due to ice ride up
b. frictional force between ice pieces
c. energy to deform rubble pile
d. shear effects
e. force to strain and fracture the ice sheet

A summary of the ridge building force estimates by various researchers is given in Table 6.1. It can be seen that the range of the predicted ridge building force is from one to 55 kips/ft. (for a six foot thick parent ice sheet.)
In the actual ice force evaluation the limited driving force requires that the far field force be evaluated in conjunction with the wind and current drag force. Since the ridge building force is evaluated based on very limited observational data it is not proper for a designer to give this predicted force too much confidence. As the data base increases the confidence level will increase too, but until such time the models presented above should be used with caution.

6.3. Wind and Current Drag Force

Wind and current drag forces are evaluated from the following simple equation:

$$F_w = \rho_a C_a F V_a |V_a|$$

(6.2)

$$F_c = \rho_w C_c F (V_c - V_i) |V_c - V_i|$$

(6.3)

where

\[\begin{align*}
\rho_a &= \text{density of the air} \\
\rho_w &= \text{density of sea water} \\
V_a &= \text{velocity of wind} \\
V_i &= \text{velocity of ice feature} \\
V_c &= \text{velocity of the current} \\
C_c &= \text{current drag coefficient} \\
C_a &= \text{wind drag coefficient} \\
F &= \text{fetch length}
\end{align*}\]

The accuracy of the equation is dependent upon the parameter coefficient of drag, which is the most uncertain parameter in the analysis. The wind coefficient is a function of the surface profile of the ice feature. For sheets
with rubble pile, ridges, etc., the wind drag coefficient is considerably higher than that for a sheet or ice floe with few deformations.

Leavitt et al (1977), measured drag coefficient of $1.2 \times 10^{-3}$ for a smooth profile. Banke and Smith (1975), evaluated the drag coefficient considering the rough profile due to ridges, etc., as $2.7 \times 10^{-3}$. In winter when the currents are much slower and are not necessarily in the same direction as the ice movement, they may oppose the ice motion.

Croasdale and Marcellus (1981) used a slightly different form of the above equation:

\[ F_c = 0.5C_c \rho V_c^2 L^2 \]  \hspace{1cm} (6.4)

\[ F_w = C_{10} \rho_o V_w^2 L^2 \]  \hspace{1cm} (6.5)

where,

\[ C_c = 5.5 \times 10^{-3} \]  \hspace{1cm} [note $0.5C_c = 2.75 \times 10^{-3}$]

$L$ is the length of the ice feature (assuming the feature is square in shape) and

\[ C_{10} = 3 \times 10^{-3} \]
<table>
<thead>
<tr>
<th>Model</th>
<th>Kips/Ft.</th>
<th>KN/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prameter and Coon (1973)</td>
<td>1-3</td>
<td>14-43</td>
</tr>
<tr>
<td>Kovacs and Sodhi (1980)</td>
<td>2-45</td>
<td>29-643</td>
</tr>
<tr>
<td>Kry (1977,1980)</td>
<td>1-20</td>
<td>14-286</td>
</tr>
<tr>
<td>Vivatrat and Krieder (1981)</td>
<td>5-55</td>
<td>71-786</td>
</tr>
<tr>
<td>Assur (1971)/Kry (1980)</td>
<td>10-50</td>
<td>143-714</td>
</tr>
<tr>
<td>Mellor (1983)</td>
<td>0.7-1.4</td>
<td>10-20</td>
</tr>
<tr>
<td>Pritchard (1977)</td>
<td>5.0-7</td>
<td>70-100</td>
</tr>
<tr>
<td>AIDJEX (1977)</td>
<td>0.7-7</td>
<td>10-100</td>
</tr>
<tr>
<td>Nevel (1983)</td>
<td>2.8-10.5</td>
<td>40-150</td>
</tr>
<tr>
<td>Crossdale (1984)</td>
<td>1.4-24</td>
<td>20-350</td>
</tr>
</tbody>
</table>
LIMIT STRESS
\[ F = f_c \times A \]

LIMIT FORCE
\[ F = \text{[Minimum of]} \ F_w + F_c + F_t \ & \ f_c \times A \]

Figure 6.1 Concept of Limit Force and Limit Stress
Figure 6.2 Stages in Limit Force Interaction
(after Croasdale 1977)
7. Ice-Forces Due To Icefloe Impact

7.1. Introduction

One of the major concerns in design of Arctic structures is the possibility of having an impact with a large icefloe. In summer these ice floes are driven by wind and current and attain relatively high velocities (up to 1.5 m/sec). Due to their large masses their kinetic energy is extremely large even at relatively slow velocities. Floes of up to 10 million tons in mass have been sighted. Impact forces from these ice features can be very large.

The ice floe impact scenario is actually a dynamic problem, since during the impact the foundation of the structure responds dynamically to the high impact load and influences the force history. Croteau (1983) developed an extensive finite element analysis model for evaluating the response of foundation-structure system to ice floe impacts using non-linear behavior characteristics of the soils. A detailed discussion of this model is beyond the scope of this report. However a brief schematic representation of the model is presented in figure 7.1 and 7.2. More discussion of the foundation response is given in a later chapter.

The response of some of the structures under ice loading has been studied in the field. Cook Inlet structures were the first structures to be instrumented to collect ice force response. However more recently Tarsuit Island revealed some interesting data for summer ice floe impact forces. Although this data is proprietary, a general trend of the loads measured is shown in figures 7.3 and 7.4. These figures parametrically show how the load increases sharply due to a summer ice floe impact. Note the ratcheting high frequency component super-imposed on the low frequency peak load.

122
In the following sections a *Quasi Static* method of analysis of the impact problem is presented. The term *Quasi-Static* is used since the analysis ignores the dynamic response of the soil-structure system. The assumption is not valid for all types of foundation systems, yet it is considered that such an analysis provides a good upper bound on the force estimate. Moreover, in situations where the soil is relatively stiff, this analysis may provide a good estimate for the preliminary design force. Also, in most cases the structure/soil interaction cannot fully develop during the time interval of impact, therefore the quasi-static analysis is considered to be provide a relatively accurate estimate of the behavior.

7.2. Ice Floe Impact Scenario

The ice floe-structure impacts can be classified in two categories, 1. Concentric Impacts, and 2. Eccentric Impacts.

7.2.1. Concentric Impact

The concentric impact is an idealized situation in which it is assumed that the ice floe approaches the structure such that its direction of travel is exactly in line with the radial direction of the structure and the centroids of the structure and the ice floe are in line with the point of contact (figure 7.5 (a)). This type of collision also assumes that during the interaction the floe does not rotate, or change its direction of travel: in other words at the end of the impact the entire kinetic energy of the floe is absorbed and as a result the floe comes to a dead stop. Such a collision can be envisioned to follow the following steps:

i. The ice floe traveling at a steady speed approaches the structure (figure 7.6).
ii. At first contact the structure indents the ice and develops a contact force. As a result the ice fails in the vicinity of the contact zone. The contact force opposes the motion of the ice floe.

iii. The hydrodynamic effect of the added mass possibly alters the energy of the ice floe since the added mass of the floe changes due to change in its velocity. Just prior to impact, the water trapped between the floe and the structure starts to flow in the opposite direction, back under the floe, and also to escape around the sides, and as a result, drags the floe against its direction of travel causing it to slow down. This can be considered as a negative added mass effect.

iv. The floe penetrates further into the structure as a result of crushing of ice in contact. This results in larger area of the structure coming in contact with the ice. The net result is that the total reaction force increases and causes the floe to slow down further.

v. The floe continues to penetrate and slow. As the velocity slows, the strain rate decreases and the crushing strength decreases. The energy is continually dissipated due to crushing of ice. Eventually the entire kinetic energy is dissipated and the floe comes to a stop.

vi. After the floe has come to a dead stop the environmental driving forces continue to act on the ice floe and are transferred to the structure.

7.2.2. Eccentric Impact

In the case of an eccentric impact, a more probable scenario is envisioned, in which it is assumed that the center of gravity of the ice floe is not necessarily in line with the centroid of the structure and, the direction of travel of the ice floe is also not radial to the structure, figure 7.5 (b). During such an impact, the floe produces an impact force on the structure which is
not solely radial, but instead has a radial and a tangential component. Thus the foundation experiences both direct shear and torsion. The resultant force acting on the floe causes a moment about its centroid, which causes it to rotate during impact and change its direction of travel. Finally the ice feature clears of the structure and continues to travel with a residual velocity. In other words the structure does not absorb the entire energy of the ice floe, and the floe retains part of the energy.

Such an impact follows the following steps:

i. An ice feature traveling at a steady velocity approaches the structure. The velocity is not radial to the structure (figure 7.7).

ii. At first contact the ice feature penetrates the structure at the interface and the resulting force, limited by the crushing strength of the ice, is developed. Assuming there is no slipping at the interface the resultant force acts in a direction parallel to the direction of the velocity vector of the floe. The resultant force thus has two components, one radial and the other tangential. The radial component causes direct shear in the foundation, whereas the tangential component causes torsional force in the foundation. The reaction force also causes a moment on the ice floe, as a result of which it begins to rotate.

iii. The ice floe continues to penetrate the structure and simultaneously changes its direction of travel (with a lower velocity).

iv. Eventually the floe rotates enough to clear the structure and then continues to travel with a velocity lower than its initial velocity. Part of the kinetic energy is absorbed by the structure and the remaining retained by the ice floe.

It is worth noting here that the hydrodynamics of an eccentric impact are very important and possibly they will have a very significant affect on the
interaction.

7.3. Quasi-Static Analysis for Concentric Impact

For concentric Impact a finite difference procedure of analysis has been proposed by Rojansky (1983). The algorithm for this analysis technique is as follows:

STEP 1

Initialize the geometry of the structure, shape of the floe, ice properties and the energy of the ice floe.

STEP 2

Assume an incremental time step.

STEP 3

Calculate penetration of the ice floe based on average velocity of the floe.

STEP 4

Calculate the area of contact based on the penetration and the geometry of the floe and the structure.

STEP 5

Evaluate strain rate and read off the corresponding uniaxial strength of ice from the established ice strength curve.

STEP 6

Calculate the aspect ratio and compute the Indentation factor.

STEP 7

Calculate the contact factor based on the strain rate.

STEP 8

Evaluate the force using the above calculated parameters.

STEP 9
Calculate deceleration of the ice floe from the following equation:

\[ \frac{dv}{dt} = \frac{F}{M} \]

STEP 10

Calculate change in velocity by the following equation:

\[ \Delta V = \frac{dv}{dt} \cdot \Delta T \]

STEP 11

Calculate the new velocity at the end of the time step.

STEP 12

Calculate the average velocity

STEP 13

Repeat steps 1 through 12 and iterate until the velocity of the floe is zero.

A flow chart for the above mentioned algorithm is given in figure 7.8. A sample output is shown in figure 7.9 and 7.10.

The effects of ice strength and contact factor are very important in such an analysis. The effects are not very obvious and hence it is important to point out that while doing an impact analysis special care should be given to these factors. The reduction in effective strength, either due to lower ice strength or the use of a contact factor can sometimes lead to a higher global force on the structure. This is somewhat a paradox but has been proven to be true (see figure 7.10). The reason for this is that softer ice will result in lower forces initially, thus causing the floe to penetrate more, which will lead to a larger contact area. In some cases this situation will result in a higher maximum force.
7.4. Eccentric Impact Analysis

As noted in the preceding section, if the direction of travel of an ice feature is not in line with the radial direction of the structure, then the ice feature does not transfer its entire energy to the structure. Instead it rotates about the point of contact and finally clears the structure. Such an impact is not likely to produce forces as high as the concentric impact, yet this type of interaction produces torque in the foundation, which in turn could produce higher stresses in the foundation. Thus from global design standpoint it is very essential to consider eccentric impact.

The problem of eccentric impact is very complicated due to the continuous change of point of contact during interaction and also due to the inclusion of rotational inertia of the floe. Further complications arise due to the added mass effect with rotation of the floe. The solution of the problem is difficult yet not impossible. The method developed here is an approximate solution.

The method has been developed based on the design procedure for fender systems against impact from berthing ships (Lord Marine, 1976).

The main objective here is to evaluate the proportion of the total energy that is retained by the ice feature after impact, or the energy that will be transferred to the structure at the point of contact. In fender design, an eccentricity coefficient has been developed to define the proportion of the energy that is transferred into the fender due to impact from ships coming at oblique angles (figure 7.11). This coefficient is strictly a function of the impact geometry and ship shape, and is given by:

\[ C_E = K^2 + r^2 \cos^2 \phi \frac{\phi}{K^2 + r^2} \]  

(7.1)

where,

\[ K \] - radius of gyration of the ice feature or ship about its vertical axis
r - distance to the point of impact from c.g. of the ice feature

φ - angle between the direction of travel and vector r

The steps for computing the ice forces from an eccentric impact are as follows:

STEP 1
Based upon the geometry of the structure and the ice feature define a relationship for force and penetration. See figure 7.12 for an example.

STEP 2
From the Force vs. Penetration relationship determine the Energy vs. Penetration curve, this can be obtained by integration the Force vs. Penetration curve.

STEP 3
Determine the eccentricity coefficient using equation 7.1, based on the properties of the ice floe and the impact scenario.

STEP 4
Compute the energy of the ice floe under consideration,

\[ E_N = 0.5Mv_N^2 \]

The energy to be absorbed by the structure is,

\[ E = E_N \cdot C_E \]

where,

\( V_N \) is the velocity component of the floe normal to the interaction face.

\( E_N \) is the Kinetic Energy of the floe normal to the interaction surface.

STEP 4
For energy level E as calculated above, find the required penetration of the floe from curve developed in step 2.
STEP 5

From force penetration relationship find the Force corresponding to the penetration.
Figure 7.1 Schematic Model of the System including the Foundation, the Structure and the Ice Feature (after Croteau 1983)

- $M_d$ = mass of deck
- $M_s$ = mass of shaft
- $M_b$ = mass of base
- $M_{f1}$ = mass of enclosed fluids
- $M_{f2}$ = mass of entrained fluids
- $M_{I1}$ = mass of adhering ice rubble + its hydrodynamic added mass
- $M_{I2}$ = mass of ice field + its hydrodynamic added mass
- $k_I$ = nonlinear ice-structure interaction spring
- $k_x, k_z, k_e$ = soil springs (dashpots not shown)
Figure 7.2 An Example of Ice Force Time History (after Croteau 1983)
Figure 7.3 Summer Collision Ice Loading from Tarsuit Island (after Bea 1984)
Figure 7.4 Winter Global Ice Loading from Tarsuit (after Bea 1984)
Figure 7.5 Two Cases of Ice Floe Impact

(a) Concentric Impact

(b) Eccentric Impact
Figure 7.6 Concentric Impact Scenario
Figure 7.7 Eccentric Impact Scenario
Figure 7.8 Flow Chart for Concentric Impact Analysis Algorithm
| TIME | FORCE | PENT | VEL | I | FC | UCS | SR ARE | N | M | T | 
|------|-------|------|-----|---|----|-----|-------|---|---|---|-------|
| 0.10 | 509.52 | 0.12 | 1.22 | 1.02 | 1.00 | 7.00 | 0.09 | 0.70 | 6.96 | 1.00 |
| 0.20 | 1432.75 | 0.24 | 1.22 | 1.03 | 1.00 | 7.00 | 0.06 | 1.97 | 9.70 | 1.00 |
| 0.30 | 2684.86 | 0.37 | 1.22 | 1.04 | 1.00 | 7.00 | 0.06 | 3.62 | 11.87 | 1.00 |
| 0.40 | 4152.10 | 0.49 | 1.22 | 1.04 | 1.00 | 7.00 | 0.06 | 5.97 | 13.70 | 1.00 |
| 0.50 | 5824.26 | 0.61 | 1.22 | 1.05 | 1.00 | 7.00 | 0.06 | 10.22 | 16.76 | 1.00 |
| 0.60 | 7681.19 | 0.73 | 1.22 | 1.05 | 1.00 | 7.00 | 0.06 | 12.86 | 18.08 | 1.00 |
| 0.70 | 9703.96 | 0.85 | 1.22 | 1.06 | 1.00 | 7.00 | 0.06 | 15.69 | 19.32 | 1.00 |
| 0.80 | 11882.93 | 0.98 | 1.21 | 1.06 | 1.00 | 7.00 | 0.06 | 20.72 | 20.87 | 1.00 |
| 0.90 | 14060.49 | 1.09 | 1.21 | 1.07 | 1.00 | 7.00 | 0.06 | 25.17 | 21.56 | 1.00 |
| 1.00 | 16665.95 | 1.22 | 1.21 | 1.07 | 1.00 | 7.00 | 0.06 | 31.85 | 22.06 | 1.00 |
| 1.10 | 19252.04 | 1.34 | 1.21 | 1.07 | 1.00 | 7.00 | 0.06 | 38.65 | 22.56 | 1.00 |
| 1.20 | 21950.66 | 1.46 | 1.20 | 1.07 | 1.00 | 7.00 | 0.06 | 46.25 | 23.06 | 1.00 |
| 1.30 | 24778.71 | 1.51 | 1.20 | 1.07 | 1.00 | 7.00 | 0.06 | 54.95 | 23.56 | 1.00 |
| 1.40 | 27705.89 | 1.62 | 1.19 | 1.08 | 1.00 | 7.00 | 0.06 | 65.15 | 24.06 | 1.00 |
| 1.50 | 30734.29 | 1.73 | 1.18 | 1.09 | 1.00 | 7.00 | 0.06 | 76.95 | 24.56 | 1.00 |
| 1.60 | 33883.02 | 1.84 | 1.18 | 1.09 | 1.00 | 7.00 | 0.06 | 89.45 | 25.06 | 1.00 |

**Figure 7.9** An Example Calculation for Concentric Impact
Figure 7.10 Plot Showing Time History of Impact Force
Figure 7.11 Schematic Diagram of Non-Sliding Contact of Ship with Fender in Berthing Operation (after Vasco Costa 1965)
Figure 7.12 Example for Force vs. Penetration and Energy vs. Penetration
8. LOCAL ICE PRESSURES

8.1. Introduction

The previous discussions have been concentrated on the evaluation of *Global* ice loads. The term global refers to the maximum total load exerted by an ice feature on a structure. A structure is designed to withstand the global loads exerted by an extreme event and the structural design ensures that these loads are transmitted to the foundation with minimum (acceptable) movement (damage) of the structure. Thus the internal framing system of the structure has to be designed to provide a load path to the foundation (the details of the framing/bracing system will be discussed in section 9). The members of the internal framing system and the peripheral ice resisting wall have to carry high "local" ice pressures. The term *local* ice pressure refers to the high pressures developed over relatively small areas at the ice-structure interface.

While determining the global ice forces, the emphasis was on evaluating maximum load developed during the sea ice/structure interaction; however, maximum global load does not necessarily produce maximum local load. Maximum global load is reached when the ice feature has penetrated the structure considerably. Typically the loaded width of the structure at this point would encompass 3-5 spans of the ice wall. The effective ice pressure at any point is given by \( \sigma \times I \), where \( \sigma \) is the unconfined compressive strength and \( I \) is the indentation factor. Since indentation factor is a function of of the aspect ratio, \( D/t \), and it decreases with increasing values of aspect ratio, the effective pressure drops with increase in loaded width. For this reason the effective pressure corresponding to the maximum global load is lower than for...
the initial contact. Thus the local ice pressure is a function of the area of loading. Figure 8.1 shows a local ice pressure curve as a function of loaded area.

For the selection of design ice pressures for various members one also has to consider the structural behavior of the member. For example, the ice wall can be considered to be a continuous beam and thus the maximum moment at the center of the beam will be produced when the span is partially loaded, since the intensity of the load drops with increase in loaded width. Similarly one can evaluate the extent of the load and its position that will produce the maximum load in the bracing wall.

The above given discussion can be summarized as follows:

1. The local ice pressure is dependent upon the area of loading.
2. Since for each of the members a different loading condition (in terms of loaded area) will produce the critical design condition, local pressures for the design will have to be independently evaluated for each member.

8.2. Method for Computing Local Ice Pressures

The following step-by-step procedure outlines the method for computing local ice forces for the design of an individual member.

1. Select the design ice feature.
2. Define the basic physical properties of the ice feature, mass, velocity of approach, draft, thickness, shape, etc. Also establish the ice properties in the form of a uniaxial compressive strength as a function of strain rate.
3. Select the indentation factor model to be used in the analysis from the various indentation models available. Also determine the contact factor to be used in the analysis.
4. Based on the properties established in the above-stated steps carry out
the quasi-static analysis (see section 7 for details) to find the complete force history of the interaction. Plot the results from the analysis in the form of Pressure vs. Loaded Width; this plot provides maximum local pressure corresponding to a given loaded area.

5. The above-given steps provide the local pressure for various areas of contact. For the design of a member, say the peripheral ice resisting wall, the local pressures are to be determined such that the design parameters (moments, shear and axial force) are maximum. Note that the local ice pressure decreases with an increase in the loaded area (this is a basic characteristic of the Indentation factor, which decreases with increase in aspect ratio, D/t). Hence, the selection of the design ice pressure has to be done for each of the members in conjunction with the structural analysis of the member. Different parts of the same member (for a continuous beam the maximum positive moment may be caused by different loading conditions than that which produces maximum negative moment) may require independent evaluation of the local ice pressures.

Hence, for each of the members determine the loading area and corresponding pressure which will provide the critical design condition.

6. Since the local ice pressure decreases with increase in area, check the stresses for any smaller area of loading with higher local pressure; it is possible that the less obvious loading width might be more critical.

8.3. Local Ice Pressures for Sloping Sided Structures

The global ice force for a conical structure is lower than for a vertical sided structure, for the same ice feature. This does not imply that the local ice pressures will be lower, too. The local ice pressures are determined by the crushing strength of ice and the global loads by the global failure mode of the
ice feature. Even though the global failure for a conical structure is by bending, locally the ice crushes at the interface. As a result the local pressures are governed by the crushing strength only. Thus, the above given methodology is also applicable to the sloping sided structures.

8.4. Generalized Curves for Local Pressure

Since we have shown that the local ice pressure will be governed by the crushing failure of ice at the contact face, the problem of computing ice forces is reduced to that of finding crushing pressures only. As an alternative to the above procedure for finding the local ice forces, a generalized solution to the problem is developed in the form of curves.

The crushing force is given by the following equation:

\[ F = I \sigma_f c DT \]  \hspace{1cm} (8.1)

where,

- \( I f(D/T) \) = Indentation Factor,
- \( f_c \) = Contact Factor,
- \( D \) = Width of contact,
- \( T \) = Thickness of ice in contact,
- \( \sigma \) = Uniaxial Compressive Strength.

Note that this equation has two parameters which are to be selected by the designer, \( I \) and \( f_c \). Indentation factor \( I \), is a function of aspect ration \( D/T \). A number of models are available for the indentation factor. In the present analysis, the following model after Croteau (1983) has been used:
Curves shown in Fig. 8.2 give normalized local ice pressures as a function of loaded span width and various thickness of contact. The normalized pressure is the ratio of the local ice pressure and the product of the unconfined pressure and contact factor. The contact factor can be selected based on temperature and strain rate of the ice. Section 3.1.3 discusses the contact factor in detail. The unconfined compressive strength is different for different strain rates, but for the penetrations of a few feet the ice feature will not slow down to the level where the compressive strength may vary. The unconfined compressive strength can be selected for a given ice feature based on its ice type, initial velocity, and temperature.

8.5. Sequential Procedure for Determining Local Ice Pressure from Curves.

1. Determine ice feature interaction thickness from the geometry of the ice feature and the structure. Or, based on the critical loading conditions the thickness can be arbitrarily selected.

2. Determine the loading width that will produce the critical design condition. As a first trial select the entire span to be loaded.

3. Select the contact factor based on temperature and strain rate.

4. Select the unconfined uniaxial compressive strength of ice based on any field observation or standard published data. Then select the strength value corresponding to the strain rate and the temperature under consideration.

5. From the curve read normalized pressure for the width of loading and the thickness of interaction.
6. Find local pressure from the following equation:

\[ P_I = f_L f_c \sigma \]  

(8.3)

where

\( P_I \) \textit{is the Local Ice Pressure}

\( f_L \) \textit{is the Normalized Local Pressure from fig. 8.2}.

7. As a check, evaluate local pressures for different widths of loading and see if any of these are more critical than the one previously selected. If the local pressure changes vary rapidly with loading area then it is important to analyze the member for a number of different loaded areas and evaluate the critical design condition. For the design of bracing walls, two full loaded spans will usually govern, whereas for the external wall between bracing walls, a loading of 0.5 to 0.6 of the span length will usually govern.

8.6. Field Measurements of Local Ice Pressures

Daley (1984) reported the data recorded on one of the instrumented panels of the U.S. Coast Guard icebreaker Polar Sea. This data is partially reproduced in figure 9.2. The local ice pressures have been measured over areas extending from .15 to 5 square meters, and the pressures recorded range from .1 to 8 MPa.
Figure 8.1 Local Ice Pressure Curve (after Bruen et al 1982)
Figure 8.2 Local Ice Pressure Curve

- $f_L$ - LOCAL PRESSURE COEFFICIENT
- $D$ - CONTACT WIDTH *
- $t'$ - THICKNESS OF ICE IN CONTACT *
- * SEE SKETCH FOR DETAIL
Figure 8.3 Typical Ice Wall System
EXTERNAL WALLS & INTERNAL FRAMING SYSTEMS

9. External Walls & Internal Framing Systems

9.1. Introduction

As an ice feature interacts with the structure, a reaction force is generated. This ice interaction force must be carried by various members of the structure without sustaining any damage, and finally transmitted to the foundation soils. It is imperative, then, that the structure provides an efficient load path to the foundation. In a typical structure the load path includes: (a) peripheral ice wall, (b) bracing walls, (c) diaphragm and base slabs, and (d) shear walls. Figure 9.1 schematizes the load path for a typical structure.

The loads experienced by a production platform are due to sea ice, hydrostatic pressure, wave slamming, ship impacts, temperature differentials, etc. Ice loads are much higher than any other load and thus normally govern the design of the structure. In recent years some exploratory structures, namely Tarsuit, Molikpaq, and Super CIDS, have been installed in the Beaufort Sea. The ice walls in these structures have been heavily instrumented to monitor their performance under ice loads. However, this data is proprietary and has not yet been made available to the public. As a result the present design procedures remain largely unverified. The best data available to-date for the design of ice walls for local ice pressures is from the monitoring program of ice breakers. Daley (1984) has presented (figure 9.2) the data collected from the monitored panels of U.S. Coast Guard icebreaker Polar Sea. Other examples of design for extreme concentrated loading are from the North Sea structures, which have been designed for accidental loading from ship impacts, and against "hard spot loading" on foundation slabs (due to seating of structures on sand lenses on the sea floor).
In order to avoid any brittle catastrophic collapse under extreme loading, the basic premise to be adopted in design is to provide adequate strength to resist design loads and a high level of ductility to resist overloads. To provide the structural elements with an acceptable level of reliability it is essential to design them based on semi-probabilistic methods, such as Limit State Design.
9.2. Internal Framing System

9.2.1. Design Premise for Internal Framing System

The design premise for the internal framing system is to provide a strong and cost effective system with high level of redundancy and ductility. Since the internal framing system often constitutes about 75% of the weight of the platform, it is very important that it be designed efficiently. Efficiency of the system is defined in terms of its ability to carry load at a minimum possible cost. Technically, this requires the system to disperse the load evenly into the various members of the structures so as the effectively utilize them.

The design of the internal framing system should be carried out for the following two categories of load:

(i) Global Ice Loads

(ii) Local Ice Loads

The Global loads are used for optimization of the framing configuration. The design of individual members, such as the ice wall, bracing walls, etc., will be governed by local ice pressures. Global loads are of high magnitude (although low intensity) spread over a large area, whereas local ice loads can be of very high intensity over small areas of loading. Besides the ice loads, the members also have to be designed to withstand operational loads, such as hydrostatic pressures, oil storage loads, and must also support the deck above and the foundation slab below.
9.2.2. Behavior Under Global Loads

The first criteria for the design of a framing system is its ability to efficiently disperse loads into the various members. Consider the two configurations shown in figure 9.3. One has only radial wall, and the other has a trussed wall system. The first design transmits the load towards the structure's center, and thus the active participation zone (APZ) is relatively small, whereas, the trussed members spread the load over a large number of members. The extent of the APZ will determine the intensity of loading for various members, and therefore, the design of the members is governed the global behavior of the system.

If horizontal diaphragm slabs are provided in the structure, then due to the box effect, the structure is considerably more stiff. The extreme stiffness of the horizontal slab attracts the load towards it thereby relieving other members of stress. Only a three dimensional analysis can include the stiffness of the horizontal diaphragm slabs, and thus provide a realistic behavior of the structure.

Since a typical circular structure with a central hole to accommodate the moon pool behaves like a doughnut when loaded transversely, under the global ice load the structure will try to bulge out in the direction perpendicular to the direction of loading (figure 9.4). This ovalling is not a desirable phenomena since it causes high tension in some of the members. Ovalling can be restricted by designing the system such that the load is carried around the structure rather than through it, or by providing horizontal diaphragm slabs.

9.2.3. Global Analysis

As discussed above, in order to establish a credible design of the internal framing system certain key features have to be examined (figure 9.5):
(1) Load Path

(2) Effect of Shape (Ovalling, Box Effect)

(3) Thermal Strains

To examine these features it is necessary to perform a three dimensional global analysis. However, due to the excessive cost involved in running large three dimensional finite element analyses it is not possible to analyse the entire structure with a single model. Small portions, one fourth to one sixth, of the structures should be analyzed in three dimensions in conjunction with larger two dimensional models. To understand the behavior of the structure, a load path should be evaluated for various load configurations. The load path can be plotted by using principal stress plots from the global analysis. The direction and magnitude should be represented on the load path plots.

A two-dimensional global analysis is not sufficient to study the features mentioned above, since the inplane stiffness of the horizontal diaphragm slabs is grossly misrepresented in such models.

A 3-dimensional global analysis also provides the boundary conditions for the finer finite element analysis of the individual members. The finite element analysis of the ice wall and other members requires that a global analysis be carried out with appropriate loading and the boundary conditions from this analysis be used as input into the detailed FEM in the form of moments and shear forces at the cut off points.

Care should be taken in determining the boundary conditions by global analysis since both three dimensional and two dimensional analysis have constraints. The axial forces and bending moments calculated from 2-D and 3-D global analysis can be markedly different. The 2-D model can be more accurately detailed to include the geometric details of the joints, so that the flexural response can be more accurately calculated. On the other hand, the
3-D model provides a more accurate representation of the inplane stiffnesses of diaphragm slabs, allowing better estimates of axial forces. Special attention should be given to the process of selecting the boundary conditions for the local FEM analysis.
9.2.4. Ice Loads for Global Analysis

The global ice loads are associated with the interaction forces generated by design ice features impinging on the structure, such as ice floes with embedded ridges. The evaluation of global ice loads is to be carried out in accordance with methods discussed in previous sections. Typically, for production structures the maximum global loads are generated by either multi-year ice floe impacts in summer or by the ice sheet with embedded ridges fully surrounding the structure in winter. Thus, a quasi-static ice-structure interaction analysis will yield the design ice loads in the form of pressure vs. area curves. The variation in pressure with area is due to the strain rate dependence and aspect ratio effects. This curve can be used to select the loading for various loaded widths. Global ice loads can be associated with ice features of different return periods.
9.3. Design Premise for Peripheral Ice Resisting Walls

The peripheral ice resisting wall is a key element of the load resisting system of a structure. The wall is subjected to high ice pressures and severe thermal strains. Since this element of the structure is very critical it requires some special design considerations.

Due to the inherent randomness and the imperfections of ice models in general, the ice loading that can be experienced by a structure in the Southern Beaufort Sea is highly random. Thus, the design of the structure should take into account this variability of ice load. Therefore, the use of a purely deterministic design procedure may not be acceptable. The adopted design procedure should be semi-probabilistic, such as the limit state design method, in order to account for the high variability in ice forces. See chapter 15 for a discussion of the development of limit state design method.

The design premise for the ice wall is to provide enough strength to resist the design loads and to inhibit progressive collapse under larger loads. This design logic can be met by the Limit State Design approach. In this approach the structure is designed to resist a design load and it is also acknowledged that this load may be exceeded during the life of the structure, hence, the structure is detailed so as to mitigate any sudden failure. In other words ductility is provided in the member to avoid catastrophic progressive collapse. The load for each limit state is selected based on statistical distribution of ice loads and acceptable probability of failure (see chapter 15 for details).

The limit states for which the design is checked are as follows:

(1) Serviceability Limit State

Under the service load the structural element is designed to perform only in the elastic range, i.e. fully recoverable straining. In reinforced and pres-
tressed concrete members, limited cracking is permitted. Table 9.1 lists the recommended allowable crack widths (Gerwick 1981). The SLS load is also referred to as the normal operating load.

(2) Ultimate Limit State

Under the design load, or the ultimate load, properly factored, the materials are allowed to be stressed into the inelastic range. For reinforced concrete structures the stress in steel is not allowed to exceed 90% of yield. In concrete, cracking but not crushing is permitted in the extreme fibers.

(3) Progressive Collapse Limit State

To account for the possibility of the load exceeding the design value, a third limit state is introduced i.e. the Progressive Collapse Limit State. The objective of this limit state is to control the behavior of the member in the post ultimate stage. The failure mode should be ductile and not brittle. Ductility in a member will ensure that the load carrying capacity does not diminish suddenly after the ultimate strength is reached. As a result the energy absorption capacity will be available within the member.

Ductility of a flexure member is either the deflection ductility or the strength ductility. In figure 9.6, the deflection ductility is defined as the ratio of the maximum deflection to the deflection at the elastic limit.

\[ D_f = \frac{\Delta_{MAX}}{\Delta_E} \]  

(9.1)

The strength ductility is the ratio of the maximum load to the elastic limit load, or:

\[ D_s = \frac{P_{MAX}}{P_E} \]  

(9.2)

Another important design consideration is that of providing local redundancy within the member, i.e., the individual members should have built-in
active or standby redundancies so that in the event one element of the member fails, the structure's load carrying capacity will not diminish. Local redundancy can be provided by properly detailing the structure. For example, a reinforced concrete member can have steel in a number of layers in the tension zone (figure 9.7) so that if the bars start to fail, failure of any one layer of steel will not lead to complete failure. In a conventional steel design the plate may deform plastically but the stiffeners or frame may buckle with a serious reduction in capacity. In the PLS, the effect of failure of one of the load carrying members on the load carrying capacity of the structure must be investigated. For example, as the deformation of the ice wall, whether steel or concrete, increases, does this impose excessive bending in the adjacent bracing walls leading to their failure in buckling?

(4) Fatigue Limit State

Arctic production structures may be subjected to high cycle fatigue due to the "ratcheting" effect of ice loads. Structures at Cook Inlet have experienced such a phenomenon. Besides the cyclic ice loads, during tow, the structures may be subject to the hog-sag moments induced by waves. Such cyclic loads are to be accounted for in the Fatigue Limit State.

For the steel structures, the conventional S-N curve method may be used to design the members for fatigue. In some cases, it is estimated that up to half the endurance life of the steel members may be used up in tow.

For the concrete structures during tow, the base slab and the top diaphragm slabs may experience heavy cyclic loads due to hog-sag moments generated by the waves. Walls and bulkheads will similarly have been subjected to cyclic shear.

Consideration must be given to the properties and behavior of materials under cyclic loads in submerged conditions and at low temperature. Structural
normal weight concrete has shown reduced endurance when submerged (by an order of magnitude or more) as compared to same concrete in air, whereas structural light weight concrete does not appear to degrade when submerged. This favorable performance of light weight concrete may be due to the absence of micro-cracks. Current research is being directed to the reduction of micro-cracking in normal weight concrete, by the addition of condensed micro silica fumes. By controlling the micro-cracking it is believed that the endurance life will be increased.

For steel structures, special steels with good ductility and weldability have been developed for use in cold corrosive environment. Special welding procedures need to be used to produce welds with endurance life to match the parent steel, especially in the heat-affected zone.

The geometry of the structure plays an important role in determining the fatigue life of the structure. Mono-towers may be particularly vulnerable to fatigue at the base-tower joint. Dynamic amplification values of up to 100% have been reported from steel monotower lighthouses in the Baltic, leading to failure in two cases. Detailed provisions for design of reinforced and prestressed concrete for fatigue are given in ACI-357 and FIP documents.

In brief, the design premise discussed above is to provide:

1) Adequate strength to resist the factored design loads.
2) Sufficient strength under normal service conditions so as not to exceed the elastic state.
3) Deflection ductility under over load. Redundancy and ductility to resist Progressive Collapse
4) Fatigue endurance to resist cyclic loads
9.3.1. Behavior of Ice Wall Under Concentrated Loads

Because ice walls are subjected to very high concentrated loads they must endure large bending moments and high shear forces. As stated earlier, the objective of the design is not only to provide adequate strength to resist the design loads, but also to ensure high ductility of the beam in the post ultimate stages. Therefore, it is important to examine the behavior of the flexure member under highly concentrated loads.

An ordinary flexure member resists loads by a combination of beam action and arch action providing the beam is deep enough. If an ordinary beam is subjected to very high concentrated loads it will fail in shear. To enhance the behavior of the beam in post-ultimate state this shear failure should be avoided and the beam should be able to undergo large deformations without losing its load carrying capacity. Under large deformations the beam action and arch action diminish and the beam carries load by catenary action. This type of behavior does not occur in all beams and is dependent on the detailed design of the member. For ice resisting walls it is desirable to have such behavior. The following discussion outlines design details for enhancing the performance of beams to meet the criteria of ductility.

9.3.1.1. Beam and Arch Action

Consider a flexure member in figure 9.8 which is subjected to moment and shear simultaneously. The moment capacity in this condition is expressed as follows:

\[ M = Tjd \]  \hspace{1cm} (9.3)

The shear resistance, \( V \), of the beam can be computed by the rate of change of moment along the beam:

\[ V = \frac{dM}{dX} = jd \frac{dT}{dX} + T \frac{d(jd)}{dX} \]  \hspace{1cm} (9.4)
If the tensile force in the beam acts at a constant lever arm, then the second term is zero and the beam is said to be under pure beam action. This type of action is the most common means of shear resistance in beams. However, if the tensile stress in steel does not change along the length of the beam, i.e., bond is zero, then the first term is zero and the beam is in arching action. For pure arch action to exist the steel must be completely debonded from the concrete. In arch action the shear is resisted by the inclined compression as shown in figure 9.9.

Normally both beam and arch action exist simultaneously. The mobilization of the arch action requires a horizontal reaction, which is provided by the flexure steel and the adjoining members. The arch action can be enhanced by providing through wall shear reinforcement to inhibit the splitting of concrete under high inclined compression, figure 9.15.

The high inclined compression in the internal arch will cause principal tension which will produce laminar cracks in the concrete. This type of failure is to be avoided if high ductility is to be expected of the member. A typical stress trajectory is shown in figure 9.9. The laminar splitting of concrete can be restrained by any of the following methods (after Gerwick et al 1981)

1) Steel Fiber Reinforcement
2) Through Wall Stirrups
3) Composite Steel-Concrete Sections with Shear Connectors
4) Through Wall Pre-stressing

The above mentioned shear connectors act as stitch bolts across the potential laminar cracks and increase the shear capacity of the section. The cracks will stay stable as long as the steel remains well below yield and the
end anchorages of the reinforcement do not slip. For this condition 1.5 - 2 % shear steel is required.

Members which have axial compression under loading, such as arched walls, have two extra advantages:

1) Due to the multi-axial stress state in concrete, the strength of concrete can be increased (figure 9.10)

2) The compression in the concrete increases the shear capacity of the section as explained below.

Punching Shear

The concentrated ice loads, acting on the exterior wall can produce "punching shear" acting along a zone around the concentrated load. ACI 318-83 indicates that the critical area for punching shear in slabs, which are supported on all four edges, follows around the periphery of the loaded area at a distance d/2 from it (figure 9.17). The code recommends that the contribution to the shear strength by the concrete in non-prestressed slabs, be assumed not greater that \( 4f'_c \cdot 0.5 b_o d \) where \( b_o \) is the perimeter of the critical area as defined in figure 9.17. Shear reinforcing steel is to be provided to resist the additional shear, with an upper limit of \( 10 f'_c \). This method assumes that the member has no in-plane restraint. However, the typical ice wall is a deep beam and does experience in-plane (axial) compression as well as transverse shearing and bending. As a result, the shear failure plane is flatter, as shown in figure 9.11. Due to the flattening of the shear crack plane, the shear capacity of member increases, but more importantly the participation of the shear reinforcement also increases (see discussion below).

In designing for extreme loads, conservative practice has been to ignore the shear capacity of the concrete, and to design the through wall shear reinforcement to carry the entire shear force.
Shear Capacity for Members With High Inplane Compression

Allowable shear force in concrete

\[ V_u \leq \phi V_n \]  \hspace{1cm} (9.5)

where,

- \( V_u \) is factored ultimate shear strength, and
- \( V_n \) is the nominal shear strength of section;
- \( \phi \) is the strength reduction factor.

\[ V_n = V_c + V_s \hspace{0.5cm} \text{, where} \hspace{0.5cm} \]  \hspace{1cm} (9.6)

- \( V_c \) is nominal shear strength of concrete = \( v_c b_d \), and
- \( V_s \) is shear strength produced by shear reinforcement = \( A_v f_y \), where

\[ A_v \] is the area of shear reinforcement crossing the shear crack, and

\( v_c \) the allowable shear strength is given by:

\[ v_c = 2\sqrt{f_c} \hspace{0.5cm} \text{psi} \hspace{1cm} \]  \hspace{1cm} (9.7)

\[ v_c = 0.17\sqrt{f_c} \hspace{0.5cm} \text{MPa} \hspace{1cm} \]  \hspace{1cm} (9.8)

If the inplane compression is acting then the following two favorable phenomenon occur:

1) ACI-318 83 allows increase in shear capacity of concrete based on the following equation:

\[ v_c = 2\left(1 + \frac{N_u}{2000A_g}\right)\sqrt{f_c} \hspace{0.5cm} \text{psi} \hspace{1cm} \]  \hspace{1cm} (9.9)

\[ v_c = 0.17\left(1 + 0.0073\left(\frac{N_u}{A_g}\right)\sqrt{f_c}\right) \hspace{0.5cm} \text{MPa} \hspace{1cm} \]  \hspace{1cm} (9.10)

however, \( v_c \) shall not exceed:

\[ 3.5\sqrt{1 + \frac{N_u}{500A_g\sqrt{f_c}}} \hspace{0.5cm} \text{psi} \hspace{1cm} \]  \hspace{1cm} or
0.29 \sqrt{1 + 0.29\left(\frac{N_u}{A_g}\right)} \sqrt{\sigma_c} \text{ MPa, where}

N_u \text{ is the factored axial load (+ve for compression), and}

A_g \text{ is the gross area of section}

2) The second beneficial effect of the presence of axial compression is its effect on the inclination of the shear crack. The cracks tend to be flatter; as a result more shear stirrups are engaged in the resisting mechanism. The inclination of the crack can be assessed from the standard Mohr circle. The general solution, referring to figure 9.11, is as follows:

\[ \tan 2\alpha = \frac{2v_u}{N_u/h} \text{ where} \] (9.11)

\[ v_u = \frac{V_u}{d} \text{ therefore} \] (9.12)

\[ \tan 2\alpha = \frac{2V_u h}{N_u d} \] (9.13)
Effect of Confinement

One of the most effective ways of improving ductility of compression members has been by providing confinement through the use of steel stirrups. Confinement of reinforced concrete by steel stirrups has been effectively provided in the seismic design of columns. One of the major reasons for providing confinement is to enhance the behavioral properties of high strength concrete. High strength concrete in unconfined conditions is very brittle (figure 9.12), while with the use of confining steel, its ductility can be increased considerably.

The concepts of aseismic design are largely concentrated on the design aspects of direct compression members and are thus not directly applicable to the design of ice walls, yet they are relevant to the design of internal framing walls, which are subjected to large in-plane loads.

The confinement in ice walls can be provided by means of through wall stirrups, anchored behind the main reinforcement. This type of reinforcement not only provides the confinement but also prevents the buckling of longitudinal steel. In the post-ultimate stage, after the concrete has crushed, the through wall stirrups in association with the longitudinal steel form a very effective mesh which holds the crushed concrete together and helps it carry shear, thereby contributing to the overall ductility of the ice wall.

The confined concrete exhibits large strain capacity, which helps in increasing the deflection ductility.

9.3.1.2. Catenary Action

Once the concrete in the compression zone is crushed, and if there is enough steel to confine the crushed concrete together, the tensile and the
compression steel can be deflected into a catenary, and considerable load carrying capability maintained. Figure 9.13 shows the various stages the

The catenary action is preceded by a transition stage in which the load may drop slightly because the compression steel is deflected to a neutral stage before it goes into tension to form a double catenary with the tensile steel. Once the top steel goes into tension it is possible that the load carrying capacity may even exceed the original "ultimate" load.

The beam is not to be designed as a catenary per se, however this type of behavior can be provided in the design as a means of meeting PLS conditions. The objective is to have a ductile mode of failure and enhanced energy absorption capacity. Some guidelines for providing such a behavior are as follows:

1) Provide enough through wall shear reinforcement - about 2% by volume. The shear reinforcement can be provided by means of through wall pre-stressing or passive shear stirrups. Tests conducted at Delft Laboratories by Walvaren (1979), have established the enhancement in shear resistance of members with through wall shear reinforcement. Figure 9.14 shows the experimental results for shear capacity with varying crack widths.

2) Provide steel in a number of layers in the tension and the compression zone; this will provide redundancy to the system and will allow larger deflections.

3) In hybrid (steel-concrete-steel) design, long shear connectors should be provided to stitch the concrete and steel system together (figure 9.15). Under large deformations the steel plates and concrete core have a tendency to slip. If this slippage is permitted to occur then there will be no shear transfer between the two components, and as a result the load
resisting capacity of the member will be greatly reduced. The design of shear connectors should be done in a manner similar to the stirrups in reinforced concrete.

In case through wall diaphragms are used, holes may be provided in the diaphragms to allow the concrete to flow through and thus ensure adequate shear transfer.
9.3.2. Current Design Practices

The current practices for the design of reinforced concrete members can be classified into the following types:

1. Working Stress Design
2. Ultimate Strength Design
3. Probabilistic Design
4. Limit State Design

All the above-mentioned have been proven to yield building designs which have performed well. However, for offshore structures, some of these design practices yield designs which are either technically, or economically unacceptable. For example, the design of members with the working stress method yields very thick sections for the high loads of the offshore environment. Similarly, the ultimate strength design of members may lead to excessive cracking under service loads. The limit state design method is the current state of the art in the design of offshore structures in European countries, and has recently been adopted by the ACI code.

The various design methods are discussed below, with emphasis to their applicability to the design of ice resisting reinforced concrete members.

9.3.2.1. Working Stress Method

In this method the members are designed to ensure that service loads do not produce an overstressing of the concrete and steel beyond the working stress level. The materials are allowed to be stressed only within the elastic range. The allowable stress in the reinforcement and concrete is to be in direct proportion to their yield or ultimate strength. For example, the concrete is to be stressed to 45% of the 27 day concrete cylinder strength when it is loaded in bending.
In effect, this method uses a load factor of unity in association with large strength reduction factors. No distinction is made between various types of load. Working stress design is not allowed by either the ACI 357 or the DNV Rules.

9.3.2.2. Ultimate Strength Design

In the Ultimate Strength Design Method, also called Strength Design Method, the section is designed by considering the inelastic behavior of the material. The design is carried out for ultimate load, which is the sum of service load, live load, and environmental load, etc., with appropriate load factors for each class of loading. The strains in the materials are permitted to go beyond the elastic range.

Ultimate Strength Design Method is considered more reliable than the Working Stress Method since it utilizes material more rationally by accounting for its strength and deformations in the post elastic stages. This method allows more flexibility in design by permitting a rational selection of load factors. For example, if the precise expected load on a structure is known, then a lower load factor may be used, thus yielding a more economical design.

Ultimate strength design is usually the first stage of the Limit State design practice.

9.3.2.3. Probabilistic Design

Ideally, the design could be based on a probabilistic method, i.e., the statistical variability of both the loads, and the material strength must be taken into account. The design could incorporate the statistical variability and assure an acceptable level of failure probability of the member. This type of approach involves a great deal of mathematical complexity and is therefore
generally considered impractical to be used as a design tool. However, based on probabilistic methods and with the aid of strong statistical data base, load and material factors can be developed and used in the limit state design. Such an approach implicitly assures a certain maximum risk level. Chapter 15 has outlined a procedure for developing such factors.

9.3.2.4. Limit State Design

More recently, the features of both the Working Stress, and Ultimate Strength Designs have been incorporated into one design method to overcome their shortcomings. In Ultimate Strength Design there is danger the member might develop excessive cracks under the service loads, while still being strong enough to carry the ultimate loads. This is due to the fact that the steel is allowed to carry high stresses. Other problems, such as excessive deflection may also arise under service loads, since no criteria is used to check the performance of the structure under lower service loads.

For these reasons European codes have adopted the concept of Limit State Design (CEB-FIP Model Code). More recently ACI 357 code has also adopted the limit state design procedure. As per this concept various states of the structure are defined along with acceptable performance criteria. For offshore structures the following four limit states have been defined by the Det Norske Veritas.

1) Ultimate Limit State (ULS)

This limit state corresponds to the maximum load carrying capacity. Typically, for offshore structures, the loads associated with ULS are selected such that their return period is 100 years (if the structure's service life is 20 to 25 years). Under ULS, cracking is permitted. However the concrete should not be stressed beyond its ultimate strength (i.e. stress in concrete < 0.85 $f'_{c}$),
although local spalling may be permitted. The steel should also not be beyond yielding ($f_{\text{steel}} < 0.90 f_y$).

Ultimate limit state utilizes certain partial load and material factors. Different factors are used for different types of loads. The load factors from various codes are given in table 9.2. A summary of the material factors is given in table 9.3.

2) Fatigue Limit State (FLS)

This limit ensures acceptable performance, i.e. stress state and deflections of members, under repeated loads.

The Arctic production structures will be subjected to high cycle fatigue due to the "ratcheting" of ice loads. Such loads have been observed to occur on Cook Inlet structures during sheet ice-structure interaction. The structures being towed to the Arctic will also experience fatigue due to wave generated hog-sag moments in the structure. Such fatigue is especially critical to steel structures, for which it is estimated that up to half the endurance life of some critical members may be used up during the towing. Special care also needs to be given to the welding procedures and details to account for the fatigue. Performance of materials under submerged conditions at low temperature and cyclic loading is also an important factor.

3) Serviceability Limit State (SLS)

This limit state ensures that frequent loads, with occurrence rates of once a month to once a year, do not cause excessive cracking, deformations, or vibrations, that jeopardize the service performance of the structure. The SLS loads are much lower than the ULS loads. This limit state utilizes load and material factors equal to unity. The main objective here is to keep the stresses in both steel and concrete within the elastic range to limit crack width.
to acceptable values.

4) Progressive Collapse Limit State

No load criteria can be established with certainty. There always exists the possibility of having a load scenario beyond that envisioned during the design. This is relevant to arctic structures due to the scarcity of reliable data for ice loading and uncertain modeling techniques. For these reasons, special attention should be paid to the ductility of the structures. The main criteria for safety is that the structure should behave in a ductile manner in the event of overload. In other words, a brittle behavior should be avoided since brittle failures are sudden and do not give advance warning of failure. The ductility of the structure is established by designing the structure in such a way that it undergoes large deformations while maintaining its load carrying capacity under extreme loads. Thus the criterion for ductile design remains that the structure is allowed to suffer permanent damage provided that it does not lead to a progressive collapse, resulting in loss of life or environmental pollution or inability to repair the structure.

For arctic systems, the uncertainty in predicting local ice loads does exist, and thus the approach in design should be to design a structure for three levels of loading as given in table 9.1.
9.3.3. Local Ice Pressures

Structures in the Beaufort Sea experience loads inflicted primarily by sea ice. Local ice loads will normally govern the design of the members of the structure. The following scenarios are among the possible local ice pressure loading cases:

1) uniform local ice pressure applied over a limited portion of the ice wall;
2) uniform ice pressure with isolated, high intensity hard spots; and
3) ice loads due to a relatively small ice piece impinging against the structure at high velocity imparted by waves.

9.3.3.1. Uniform Local Ice Pressure

The uniform local ice pressure is the standard format adopted by most engineers. In this format a high intensity ice pressure (700 psi or more) is considered to be spread over a portion of the span. The extent of the local pressure in the vertical direction below the water line should be corresponding to the thickness of the design feature. Very deep features usually have softer ice towards the bottom. Thus for structures in deep water depths the ice pressures near the bottom could be selected to be lower than near the water line.

The high contact stress is due to the confinement of ice due to the presence of the structure and the adjoining ice. This model assumes that the cracking of ice does not prevent the triaxial stress state in the ice to be formed, and that the ice strength is uniform throughout. The ice loading is presented in the form of area of loading vs. pressure curves (figure 9.16). The procedure for evaluating local ice pressure by this method is discussed in detail in section 8;
9.3.3.2. Local Ice Pressures for Sloping Sided Structures

The global ice force for a conical structure is lower than for a vertical sided structure for the same ice feature. However, this does not imply that the local ice pressures will be lower. The local ice pressures critical to the design of a member, e.g., the peripheral ice resisting wall, typically correspond to a loading width of one span or less. For a loaded width of 20 ft, the penetration of a large ice floe is to the order of 1 feet. The mode of failure at this stage is strictly crushing at the ice/structure interface. It can be shown that in most cases the total force corresponding to 20-30 ft. of contact width is much smaller than the force required to cause a global failure of the ice feature in bending or shearing generally associated with the maximum force condition. Hence, the above given methodology is also applicable to the sloping sided structures.

9.3.3.3. Hard Spots with Uniform Pressure

Local ice pressure, computed with the first approach are very high and are considered to be on the conservative side since the model is based on an extrapolation of small scale laboratory tests to large scale prototypes. An alternative, less conservative approach is one in which it is assumed that the full confinement of the ice is limited to small zones within the ice and that the majority of ice is unconfined due to micro-cracking. Such an approach, although reasonable has not been validated by any tests. Two examples of such loading are shown in figure 9.17.

9.3.3.4. Wave Accelerated Small Ice Features

Some ice features might be small enough in size to be picked up by waves in summer, accelerated to large velocities (up to 5-8 m/sec), and hurled at the structure. These ice features are of concern to the designer because the ice
wall should be able to absorb the energy from such impacts. Thus, it is important to assess the energy absorption capacity of ice walls.

9.3.3.5. Other Practices and Codes for Ice Pressure

Some codes for design of ice breakers have made recommendations for design ice pressures. These are:

Polar Class Ice Breaker Design

The outer plating of the ice breaker, in the ice belt, is designed to withstand localized "high impact" loads of 1150 psi acting over the full span of the panel [Specifications for 400 foot ice breaker for the United States Coast Guard, 1971].

Canadian Arctic Shipping and Pollution Prevention Regulations (CASPER)

"1500 psi pressure should be used for design of vessels with unlimited ice operations." This code does not specify the variation in ice pressure with area of loading.

American Petroleum Institute (API)

No specific recommendation is made regarding local ice pressures.
9.4. Recommended Design Procedure for Ice Walls

The recommended design procedure is as follows:

1) Load Selection: Select the design loads for two limit states, serviceability, and ultimate.

2) Define the performance criteria for the two limit states, as shown in table 9.1.

3) Design the structural member to meet the design load (ultimate), using standard or any accredited designing procedure.

4) After preliminary designing of all members have been done, perform a linear three dimensional analysis with appropriate local loading. The loading for each member such as ice walls, bracing walls, diaphragm slabs, etc. will be different. Carry out the global analysis on about a quarter segment of the structure if the structure is symmetrical.

5) Determine the boundary conditions, i.e. moment, shear, axial forces to be used in a non-linear finite element analysis of a smaller section of the structure from global analysis. Detail the 2-D finite element analysis model of at least 3 bays of the ice wall with precise shape of the members. In the analysis use non-linear properties of the material. Use results from this analysis to verify that the member meets the SLS and ULS criteria. If necessary modify the structure at this stage.

6) Isolate the critical member(s) in the structure and assume it to be overloaded and highly deformed or ruptured. Determine the behavior of the structure under such condition to verify the structure's stability under Progressive Collapse Limit State.

7) In the PLS analysis use proper material properties in the analysis to account for any passive or active confinement of concrete in the member.
during loading. Active confinement is provided by prestressing, and passive, by special through wall stirrups. All material models should have accurate nonlinear portions of the stress-strain relationships. Also take into account the post-elastic behavior of the member and allow for the arch and catenary action of the wall under large deflections.
9.5. Thermal Considerations

The arctic environment offers the structure very severe thermal conditions. The air temperature can be as low as -50° F with a relatively high internal structure temperature of +70° F. Seawater temperature is typically 28° F during winter. Figure 9.19 shows a typical temperature distribution. As a result of the thermal gradients in the ice belt zone the structure can undergo severe thermal strains. The thermal effects are thus very important and are to be studied at global, local and micro levels.

Global Thermal Loading

On the global scale some very unique thermal straining conditions can be generated. For a circular cylindrical structure with internal diaphragm slabs or walls, the external shell will have a tendency to shrink more than the internal will permit. The walls will try to deform to a smaller circumference, but are prevented from doing this by the diaphragms: thus through-section tensile stresses will occur. The thermal strains in the wall will be accentuated over the supports, due to the negative moments generated (figure 9.20). Extra reinforcing steel can be used to reduce the crack widths to acceptable limits.

Local Thermal Loading

Above the water line the ice wall will be subjected to extreme thermal strain but little or no ice loading. The thermal strains will tend to put the outer colder face of the wall in tension (locally) and cause cracks. This behavior requires special considerations, such as providing extra steel, pre-stressing, etc. Fortunately, the highest intensity ice loads occur below the waterline, and not above the water line where the maximum thermal strains occur.

Typical values for thermal straining are as follows:
\[ \sigma = \pm \alpha \frac{E \Delta T}{2(1-\mu)} \]  

(9.14)

where,

\( \alpha \) is coefficient of thermal contraction ( \( = 4.9 \times 10^{-6}/\,^\circ\text{F} \) )

\( E \) is Elastic Modulus (\( = 3.5 \times 10^6 \) psi)

\( \Delta T \) is Differential in Temperature (\( ^\circ\text{F} \))

\( \nu \) is Poisson's Ratio (\( = 0.20 \))

For \( \Delta T = 100 \, ^\circ\text{F} \); \( \pm 1000 \) psi

Thermal loading cases should include a variety of cases of internal and external thermal conditions.

Thermal strains are self-limiting in that as cracks develop, the stiffness of the wall reduces.
<table>
<thead>
<tr>
<th>Loading Case</th>
<th>Load Frequency</th>
<th>Design Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Service Load</td>
<td>Annual</td>
<td>Crack Width - inside face &lt; 0.25 mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Crack Width - outside face &lt; 0.15 mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Compression in Concrete &lt; 0.45 f'c</td>
</tr>
<tr>
<td>Ultimate Load</td>
<td>100 - 200 years</td>
<td>Steel - below yield</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Concrete - compressive stress &lt; 0.85 f'c</td>
</tr>
<tr>
<td>Extreme Event</td>
<td>1000 years</td>
<td>Ductility &gt; 6-9</td>
</tr>
</tbody>
</table>

Table 9.1: Loading Cases for Ice Wall
<table>
<thead>
<tr>
<th></th>
<th>DEAD</th>
<th>LIVE</th>
<th>ICE</th>
<th>THERMAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>DnV</td>
<td>1.0</td>
<td>1.0</td>
<td>1.3</td>
<td>1.0</td>
</tr>
<tr>
<td>MMS</td>
<td>1.2/0.9</td>
<td>1.2/0.9</td>
<td>1.3</td>
<td>1.2/0.9</td>
</tr>
<tr>
<td>ACI-357</td>
<td>1.2/0.9</td>
<td>1.2/0.9</td>
<td>*</td>
<td>1.2/0.9</td>
</tr>
<tr>
<td>ABS</td>
<td>1.2/0.9</td>
<td>1.2/0.9</td>
<td>1.3</td>
<td>1.2/0.9</td>
</tr>
<tr>
<td>FIP</td>
<td>1.1/0.9</td>
<td>1.3/0.9</td>
<td>1.3</td>
<td>1.1</td>
</tr>
</tbody>
</table>

* The code recommends that load factors should be selected independently for each site.

Table 9.2: Summary of Load Factors
### Capacity Reduction Factors / Material Coefficients

#### ACI-318

<table>
<thead>
<tr>
<th>Feature</th>
<th>Coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexure, with or without axial tension</td>
<td>$\phi = 0.90$</td>
</tr>
<tr>
<td>Flexure with axial compression:</td>
<td></td>
</tr>
<tr>
<td>i. if spirally reinforced</td>
<td>$\phi = 0.75$</td>
</tr>
<tr>
<td>ii. otherwise</td>
<td>$\phi = 0.70$</td>
</tr>
<tr>
<td>Shear and Torsion</td>
<td>$\phi = 0.85$</td>
</tr>
</tbody>
</table>

#### Det Norske Veritas

<table>
<thead>
<tr>
<th>Feature</th>
<th>Coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>ULS</td>
<td></td>
</tr>
<tr>
<td>Concrete</td>
<td>$\gamma = 1.2$</td>
</tr>
<tr>
<td>Steel</td>
<td>$\gamma = 1.15$</td>
</tr>
<tr>
<td>PLS</td>
<td></td>
</tr>
<tr>
<td>Concrete</td>
<td>$\gamma = 1.1$</td>
</tr>
<tr>
<td>Steel</td>
<td>$\gamma = 1.0$</td>
</tr>
</tbody>
</table>

*Table 9.3: Capacity Reduction Factors & Material Coefficients*
Figure 9.1  A Typical Load Path
Figure 9.2  Pressure versus Area Curves of equal probability (after Daley et al, 1984)
Figure 9.3 Portion of structure carrying ice load
Figure 9.4 Ovalling of structure under global ice loads
Figure 9.5  Elements of Global Analysis
Figure 9.6 Definition of Deflection Ductility = \( \frac{\Delta_{\text{MAX}}}{\Delta_{E}} \)
Figure 9.7 Reinforced Concrete Member for Ice Wall
Figure 9.8 Arch action in an idealized beam
(after Park & Pauley)
Laminar cracks can develop normal to principal compression stress, if not stitched by shear reinforcement.

Figure 9.9 Inclined compression stress trajectories in a deep beam
Figure 9.10  Triaxial stress model for concrete
Figure 9.11 Idealized beam with inclined shear crack

\[ C = \frac{d}{\tan \alpha} \]
Figure 9.12 Constitutive model of concrete under various levels of confinement
Figure 9.13  Various stages of a beam as it deflects under load
Figure 9.14 Shear capacity for varying percentages of shear reinforcement (from Walraven, 1979)
Figure 9.15  Typical ice wall configuration for reinforced concrete and hybrid steel-concrete construction
Figure 9.16  Local Ice Pressure Curve (after Bruen et al, 1928)
Figure 9.17 Critical Area for Punching Shear
Figure 9.18 Some Concepts for Hard Spot Local Pressure Loading
Select Loads
• Service Load
• Design Load

Preliminary Design

3-D Global Analysis

Boundary Conditions

2-D Linear Analysis on 3-4 Bays

CHECK

ULA
PLS
SLS

O.K.

2-D Non-Linear F.E.M. Analysis (1-Span)

CHECK

SLS
ULS
PLS
FLS

Figure 9.19 Design Procedure for Ice Walls
Figure 9.20 Typical Winter Temperature Regime
Figure 9.21  Global Thermal straining of a typical structure