Three-dimensional analysis of composite soil-steel structures.

Youssef Fouad Girges

University of Windsor

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THREE-DIMENSIONAL ANALYSIS
OF COMPOSITE SOIL-STEEL STRUCTURES

by

Youssef Fouad Girges

A Dissertation
submitted to the Faculty of Graduate Studies and Research through
the Department of Civil and Environmental Engineering
in Partial Fulfilment of the Requirements for the
degree of Doctor of Philosophy at the
University of Windsor

Windsor, Ontario, Canada
1993
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ABSTRACT

A three-dimensional finite element analysis is carried out in this research to study the behaviour of soil-steel structures. An experimental laboratory model was built and tested in order to verify the theoretical analysis. A good agreement has been found between the theoretical and experimental results. The behaviour of a single conduit is demonstrated using three-dimensional and two-dimensional finite element models. The live load dispersion in the soil above the conduit is examined and compared to the present codes (OHBDC, 1991; AASHTO, 1983). The thrust and bending moment around the conduit walls as well as the stability of a single conduit are presented. The behaviour of multi-span conduits is analyzed using both three-dimensional and two-dimensional finite element modelling. The analysis examines the effect of spacing between conduits on their behaviour as well as their load-carrying capacity. Both elastic and elastic-plastic material models are considered. A suggested procedure for the design of multi-span conduits is presented by introducing two factors: the first factor is introduced in order to increase the bending moment of multi-span conduits; and the second one is a reduction factor for
modifying the buckling stress in multi-span conduits. Also a three-dimensional analysis of soil-steel structures is presented, in which the depth of cover is varied from maximum at the middle part of the conduit to zero at the conduit edges. The circumferential thrust and bending moment variation along the conduit as well as the deflection and vertical pressure distribution in the soil are examined. Finally, the end effects on the stability of soil-steel structures are presented.
To My Lovely Family:

My Wife MAGDA

and

My Daughter MARY & My Son MINA
ACKNOWLEDGEMENTS

The author wishes to express his sincere gratitude to his advisor Dr. George Abdel-Sayed for his invaluable guidance, effort, inspiration, interest and continuous encouragement during the development of this research. His most helpful supervision is greatly appreciated.

The author would like to thank the faculty and staff of the Department of Civil and Environmental Engineering as well as the computer consultants of the Computing Services at the University of Windsor for their assistance during this research. Special thanks are also due to Mr. Frank Kiss, the laboratory technician, and the members of the Technical Support Centre for their assistance during the preparation and testing of the experimental work.

The author wishes to acknowledge the financial support provided by the Natural Sciences and Engineering Research Council of Canada as well as the Ontario Ministry of Transportation and Communications.

Special thanks are due to Mrs. Abdalla for her assistance in drafting the figures in this dissertation. The author is also grateful to his parents and family for their continuous encouragement.

Last but not least the author wishes to thank his wife Magda for her great support throughout the course of this study. Without her understanding, encouragement and patience the completion of this work would not have been possible.
TABLE OF CONTENTS

ABSTRACT ................................................................................................................................. vi

ACKNOWLEDGEMENTS ........................................................................................................... ix

List of Tables ........................................................................................................................... xiv

List of Figures ........................................................................................................................... xvi

Nomenclature .......................................................................................................................... xxiv

CHAPTER

I - INTRODUCTION .............................................................................................................. 1

1.1 General .......................................................................................................................... 1
1.2 The Need for Three-Dimensional Analysis ......................................................... 2
1.3 Objectives of the Research ...................................................................................... 3

II - LITERATURE SURVEY .................................................................................................... 5

2.1 General ......................................................................................................................... 5
2.2 Analysis and Design Procedure ................................................................................ 5
   2.2.1 Marston-Spangler Theory ................................................................................. 6
   2.2.2 Ring Compression Theory ............................................................................... 7
   2.2.3 Frame on Elastic Supports ............................................................................ 7
   2.2.4 Finite Element Method ................................................................................... 7
2.3 Live Load Dispersion in Soil ..................................................................................... 9
2.4 Multi-Span Conduits ................................................................................................. 11
2.5 End Effects .................................................................................................................. 12
2.6 Stability of Soil-Steel Structures .......................................................................... 12
   2.6.1 The Elastic Continuum Theory .................................................................... 13
   2.6.2 The Radial Spring Model ............................................................................. 13
2.7 Interface between Conduit Walls and Soil ............................................................... 14
III- THEORETICAL MODELLING

3.1 General ......................................................... 16
3.2 Finite Element Program ........................................ 17
3.3 Finite Element Procedure ...................................... 17
  3.3.1 Static Stress Analysis ........................................ 18
  3.3.2 Nonlinear Iterative-Incremental Analysis .................. 18
  3.3.3 Eigenvalue (Buckling) Prediction .......................... 20
  3.3.4 Material Modelling ......................................... 21
    3.3.4.1 Extended Drucker-Prager Model ...................... 22
3.4 Finite Element Model .......................................... 25
  3.4.1 Element Choice and Convergence Test .................... 25
    3.4.1.1 Two-Dimensional Mesh ................................ 25
    3.4.1.2 Three-Dimensional Mesh .............................. 26
  3.4.2 Element Properties ......................................... 28
    3.4.2.1 Corrugated Conduit Wall .............................. 28
    3.4.2.2 Soil Properties ........................................ 29
  3.4.3 Degrees of Freedom Constraints ........................... 30
    3.4.3.1 Boundary Conditions ................................ 30
    3.4.3.2 Multi-Point Constraints (MPC's) ...................... 31
  3.4.4 Load Representation ........................................ 32
    3.4.4.1 Dead Load ............................................... 32
    3.4.4.2 Live Load ............................................... 33
    3.4.4.3 Interface Between the Soil and the Conduit .......... 34
  3.4.5 Nonlinear Automatic Incrementation Control ............ 35
    3.4.5.1 Force and Moment Tolerances ......................... 35
    3.4.5.2 Increment and Iteration Limits ....................... 35
    3.4.5.3 Buckling Prediction Control .......................... 36

IV- MODEL VERIFICATION .................................................. 37

4.1 General ......................................................... 37
4.2 Laboratory Set-Up .............................................. 37
  4.2.1 Soil Container .............................................. 38
  4.2.2 Loading Set-Up .............................................. 38
  4.2.3 Conduits .................................................. 38
  4.2.4 Instrumentation ............................................ 39
  4.2.5 Soil ...................................................... 40
  4.2.6 Compaction Procedure .................................... 40
4.3 Experimental Procedure and Observations .................... 41
  4.3.1 Test No. 1 ................................................ 41
  4.3.2 Test No. 2 ................................................ 42
  4.3.3 Test No. 3 ................................................ 44
4.4 Comparison of Experimental and Theoretical Results .......... 45
**LIST OF TABLES**

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.1</td>
<td>Convergence Results of the Two-Dimensional Analysis Due to Concentrated Live Loads</td>
<td>75</td>
</tr>
<tr>
<td>3.2</td>
<td>Convergence Results of the Three-Dimensional Analysis Due to Uniformly Distributed Live Load</td>
<td>76</td>
</tr>
<tr>
<td>3.3</td>
<td>Maximum Thrust Due to Dead and Live Loads</td>
<td>77</td>
</tr>
<tr>
<td>4.1</td>
<td>Comparison between the Failure Load and the Load at First Yield for the Three Tests (Experimental Results)</td>
<td>78</td>
</tr>
<tr>
<td>4.2</td>
<td>Comparison between Deflection Results for the Three Experimental Tests at Load 8.5 kN</td>
<td>79</td>
</tr>
<tr>
<td>4.3</td>
<td>Comparison between Thrust Results for the Three Experimental Tests at Load 8.5 kN</td>
<td>80</td>
</tr>
<tr>
<td>4.4</td>
<td>Comparison between Bending Moment Results for the Three Experimental Tests at Load 8.5 kN</td>
<td>81</td>
</tr>
<tr>
<td>5.1</td>
<td>Comparison of the Vertical Pressure in the Soil for Three-Dimensional and Two-Dimensional Models</td>
<td>82</td>
</tr>
<tr>
<td>5.2</td>
<td>Comparison of Thrust and Bending Moment for Three-Dimensional and Two-Dimensional Models Due to Live Load</td>
<td>83</td>
</tr>
<tr>
<td>5.3</td>
<td>Comparison of the Elastic Buckling Stresses (Three-Dimensional and Two-Dimensional Analysis)</td>
<td>84</td>
</tr>
<tr>
<td>6.1</td>
<td>Comparison of Maximum Thrust and Bending Moment of Conduit No. 1 for Different S/D Ratios (Two-Dimensional Analysis &amp; Elastic Model)</td>
<td>85</td>
</tr>
<tr>
<td>6.2</td>
<td>Comparison of Maximum Thrust and Bending Moment of Conduit No. 2 for Different S/D Ratios (Two-Dimensional Analysis &amp; Elastic Model)</td>
<td>86</td>
</tr>
<tr>
<td>6.3</td>
<td>Comparison of Maximum Thrust and Bending Moment of Conduit No. 1 for Different S/D Ratios</td>
<td>xiv</td>
</tr>
</tbody>
</table>
6.4 Comparison of Maximum Thrust and Bending Moment of Conduit No. 2 for Different S/D Ratios
(Three-Dimensional Analysis & Elastic Model) 87

6.5 Comparison of Failure Load, Thrust, and Bending Moment of Conduit No. 1 for Different S/D Ratios
(Two-Dimensional Analysis & Elastic-Plastic Model) 88

6.6 Comparison of Maximum Thrust and Bending Moment of Conduit No. 2 for Different S/D Ratios
(Two-Dimensional Analysis & Elastic-Plastic Model) 89

6.7 Comparison of Failure Load, Thrust, and Bending Moment of Conduit No. 1 for Different S/D Ratios
(Three-Dimensional Analysis & Elastic-Plastic Model) 90

6.8 Comparison of Maximum Thrust and Bending Moment of Conduit No. 2 for Different S/D Ratios
(Three-Dimensional Analysis & Elastic-Plastic Model) 91

6.9 Comparison of the Elastic Buckling Stresses for Different S/D Ratios (Two-Dimensional Analysis) 92

6.10 Comparison of the Elastic Buckling Stresses for Different S/D Ratios under Truck Loading
(Three-Dimensional Analysis) 93

6.11 Comparison of the Buckling Stresses for Different S/D Ratios (Two-Dimensional Analysis & Elastic-Plastic Model) 94

6.12 Comparison of the Buckling Stresses between OHBDC and Two-Dimensional Analysis (Elastic-Plastic Model) 95

6.13 Comparison of the Buckling Stresses for Different S/D Ratios (Elastic Conduit Wall & Elastic-Plastic Soil Model)
(Three-Dimensional Analysis) 96

7.1 Comparison of the Elastic Buckling Stresses for the Cases of Uniform and Non-Uniform Dead Load
(Three-Dimensional Analysis) 97

7.2 Comparison of the Elastic Buckling Stresses 98

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# LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>The Most Common Shapes of Flexible Conduits</td>
<td>100</td>
</tr>
<tr>
<td>2.2</td>
<td>Illustration of Terminology Relating to the Cross-Section of the Structure</td>
<td>101</td>
</tr>
<tr>
<td>2.3</td>
<td>Variable Soil Cover along the Conduit</td>
<td>102</td>
</tr>
<tr>
<td>2.4</td>
<td>Pressure Distribution assumed in the Marston-Spangler Theory</td>
<td>103</td>
</tr>
<tr>
<td>2.5</td>
<td>Theoretical Interface Radial Pressures around Varies Conduits: (a) Circular Conduit; (b) Vertically-Elliptical Conduit; and (c) Pipe-Arch</td>
<td>104</td>
</tr>
<tr>
<td>2.6</td>
<td>Plane-Frame on Elastic Supports</td>
<td>105</td>
</tr>
<tr>
<td>2.7</td>
<td>Assumed Loading for the Stability Analysis: (a) Actual Live Load above the Conduit; and (b) Elastci Continuum Theory</td>
<td>106</td>
</tr>
<tr>
<td>3.1</td>
<td>Typical Yield Surfaces in the Deviatoric Plane for the Granular Model</td>
<td>107</td>
</tr>
<tr>
<td>3.2</td>
<td>Schematic Diagram of the p-t Plane for the Granular Model</td>
<td>108</td>
</tr>
<tr>
<td>3.3</td>
<td>Two-Dimensional Finite Elements: (a) Plane-Strain, Four-Node, Bilinear Element; and (b) Two-Node, Linear-Interpolation Beam Element</td>
<td>109</td>
</tr>
<tr>
<td>3.4a</td>
<td>Two-Dimensional Plane-Strain Model (Mesh No. 1)</td>
<td>110</td>
</tr>
<tr>
<td>3.4b</td>
<td>Two-Dimensional Plane-Strain Model (Mesh No. 2)</td>
<td>111</td>
</tr>
<tr>
<td>3.4c</td>
<td>Two-Dimensional Plane-Strain Model (Mesh No. 3)</td>
<td>112</td>
</tr>
<tr>
<td>3.4d</td>
<td>Two-Dimensional Plane-Strain Model (Mesh No. 4)</td>
<td>113</td>
</tr>
<tr>
<td>3.5a</td>
<td>Thrust Convergence Curve of the Plane-Strain Analysis</td>
<td>114</td>
</tr>
<tr>
<td>3.5b</td>
<td>Bending Moment Convergence Curve of the Plane-Strain Analysis</td>
<td>115</td>
</tr>
</tbody>
</table>
4.5 Locations of Dial and Strain Gauges

4.6 Layout of Construction Layers: (a) Case of One Conduit; and (b) Case of Two Conduits

4.7 Experimental Set-up of the Triaxial Test

4.8 Loading Area for the Three Experimental Tests

4.9 Test No. 1 during Construction (Dial Gauges Set-up)

4.10 Test No. 1 after Loading

4.11 Failure Shape of Test No. 1

4.12 Side View of the Failure Shape in Test No. 1

4.13 Deformation in Case of One Conduit

4.14 Thrust in the Walls of One Conduit

4.15 Bending Moment Distribution in Case of One Conduit

4.16 The Pre-shaped Sand Layer and the Two Conduits (Test No. 2)

4.17 Dial Gauges Set-up before Construction (Test No. 2)

4.18 Experimental Set-up before Loading (Test No. 2)

4.19 Failure Shape of Conduit No. 1 (Test No. 2)

4.20 Deformation in Case of Two Conduits (S/D = 0.5 Section I-I)

4.21 Deformation in Case of Two Conduits (S/D = 0.5 Conduit No. 1)

4.22 Thrust in the Walls of Two Adjacent Conduits (S/D = 0.5 Section I-I)

4.23 Thrust in the Walls of Two Adjacent Conduits (S/D = 0.5 Conduit No. 1)

4.24 Bending Moment Distribution in Case of Two Conduits (S/D = 0.5 Section I-I)
4.25 Bending Moment Distribution in Case of Two Conduits (S/D = 0.5 Conduit No. 1) 153
4.26 Experimental Set-up during Loading (Test No. 3) 154
4.27 Excessive Deformation Failure Mode of Test No. 3 155
4.28 Deformation in Case of Two Conduits (S/D = 0.1 Section I-I) 156
4.29 Deformation in Case of Two Conduits (S/D = 0.1 Conduit No. 1) 157
4.30 Thrust in the Walls of Two Adjacent Conduits (S/D = 0.1 Section I-I) 158
4.31 Thrust in the Walls of Two Adjacent Conduits (S/D = 0.1 Conduit No. 1) 159
4.32 Bending Moment Distribution in Case of Two Conduits (S/D = 0.1 Section I-I) 160
4.33 Bending Moment Distribution in Case of Two Conduits (S/D = 0.1 Conduit No. 1) 161
4.34 Load-Deflection Curves for the Laboratory Tests 162
4.35 Three-Dimensional Finite Element Model (Test No. 1) 163
4.36 Three-Dimensional Finite Element Model (Test No. 2) 164
4.37 Three-Dimensional Finite Element Model (Test No. 3) 165
4.38 Relationship between Load and Bending Moment (Test No. 1) 166
4.39 Deformation in Case of One Conduit at Load 8.90 kN (Section I-I) 167
4.40 Thrust Distribution in Case of One Conduit at Load 8.90 kN (Section I-I) 168
4.41 B.M. Distribution in Case of One Conduit at Load 8.90 kN (Section I-I) 169
4.42 Relationship between the Load and Bending Moment Test No. 2 (S/D = 0.50) 170

xix

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6.4 Two-Dimensional Plane-Strain Model (S/D = 0.10) 189
6.5 Deformed Model under Dead and Live Loads (S/D = 0.58) (Magnification Factor = 1.3 x 10^2) 190
6.6 Deformed Model under Dead and Live Loads (S/D = 0.10) (Magnification Factor = 1.2 x 10^2) 191
6.7 Thrust Distribution for Different S/D Ratios (Two-Dimensional Analysis) 192
6.8 Bending Moment Distribution for Different S/D Ratios (Two-Dimensional Analysis) 193
6.9 Three-Dimensional Finite Element Model (S/D = 0.58) 194
6.10 Three-Dimensional Finite Element Model (S/D = 0.42) 195
6.11 Three-Dimensional Finite Element Model (S/D = 0.26) 196
6.12 Three-Dimensional Finite Element Model (S/D = 0.10) 197
6.13 Thrust Distribution for Different S/D Ratios (Three-Dimensional Analysis) 198
6.14 Bending Moment Distribution for Different S/D Ratios (Three-Dimensional Analysis) 199
6.15 Relationship between Maximum Thrust and S/D Ratio (Elastic Model) 200
6.16 Relationship between Maximum Positive B.M. and S/D Ratio (Elastic Model) 201
6.17 Relationship between Maximum Negative B.M. and S/D Ratio (Elastic Model) 202
6.18 Deformed Model at Failure Load (Magnification Factor = 7.3) 203
6.19 Load-Deflection Curves for the Case of One Conduit (Two-Dimensional Analysis) 204
6.20 Deformed Model at Failure Load (S/D = 0.10)
6.21 Load-Deflection Curves at the Crown for Different S/D Ratio
(Two-Dimensional Analysis) 205

6.22 Relationship between Load-Carrying Capacity and S/D Ratio
(Two-Dimensional Analysis & Elastic-Plastic Model) 206

6.23 Load-Deflection Curves for the Case of One Conduit
(Three-Dimensional Analysis) 207

6.24 Load-Deflection Curves at the Crown for Different S/D Ratio
(Three-Dimensional Analysis) 208

6.25 Relationship between Load-Carrying Capacity and S/D Ratio
(Three-Dimensional Analysis & Elastic-Plastic Model) 209

6.26 Relationship between Maximum Thrust and S/D Ratio
(Elastic-Plastic Model) 210

6.27 Relationship between Maximum Positive B.M. and S/D Ratio
(Elastic-Plastic Model) 211

6.28 Relationship between Maximum Negative B.M. and S/D Ratio
(Elastic-Plastic Model) 212

6.29 Buckling Shape of Multi-Span Conduits (S/D = 0.58 , First Mode)
(Magnification Factor = 3.3 x 10^3) 213

6.30 Buckling Shape of Multi-Span Conduits (S/D = 0.58 , Second Mode)
(Magnification Factor = 3.1 x 10^3) 214

6.31 Buckling Shape of Multi-Span Conduits (S/D = 0.42 , First Mode)
(Magnification Factor = 3.2 x 10^3) 215

6.32 Buckling Shape of Multi-Span Conduits (S/D = 0.42 , Second Mode)
(Magnification Factor = 3.1 x 10^3) 216

6.33 Buckling Shape of Multi-Span Conduits (S/D = 0.26 , First Mode)
(Magnification Factor = 3.1 x 10^3) 217

6.34 Buckling Shape of Multi-Span Conduits (S/D = 0.26 , Second Mode)
(Magnification Factor = 3.1 x 10^3) 218

xxii
<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.35</td>
<td>Buckling Shape of Multi-Span Conduits (S/D = 0.10, First Mode) (Magnification Factor = 3.1 x 10^3)</td>
<td>220</td>
</tr>
<tr>
<td>6.36</td>
<td>Buckling Shape of Multi-Span Conduits (S/D = 0.10, Second Mode) (Magnification Factor = 3.0 x 10^3)</td>
<td>221</td>
</tr>
<tr>
<td>6.37</td>
<td>Relationship between Elastic Buckling Stress and S/D Ratio</td>
<td>222</td>
</tr>
<tr>
<td>7.1</td>
<td>Three-Dimensional Finite Element Model (Case of Non-Uniform Dead Load)</td>
<td>223</td>
</tr>
<tr>
<td>7.2</td>
<td>Circumferential Thrust Variation along the Conduit Due to Live Load in Case of Non-Uniform Dead Load</td>
<td>224</td>
</tr>
<tr>
<td>7.3</td>
<td>Circumferential Thrust Variation along the Conduit Due to Dead Load in Case of Non-Uniform Dead Load</td>
<td>225</td>
</tr>
<tr>
<td>7.4</td>
<td>Circumferential Thrust Distribution due to Non-Uniform Dead Load</td>
<td>226</td>
</tr>
<tr>
<td>7.5</td>
<td>Circumferential B.M. Variation along the Conduit due to Live Load in Case of Non-Uniform Dead Load</td>
<td>227</td>
</tr>
<tr>
<td>7.6</td>
<td>Circumferential B.M. Variation along the Conduit due to Dead Load in Case of Non-Uniform Dead Load</td>
<td>228</td>
</tr>
<tr>
<td>7.7</td>
<td>Circumferential B.M. Distribution due to Non-Uniform Dead Load</td>
<td>229</td>
</tr>
<tr>
<td>7.8</td>
<td>Deflection Variation along the Conduit due to Live Load in Case of Non-Uniform Dead Load</td>
<td>230</td>
</tr>
<tr>
<td>7.9</td>
<td>Deflection Variation along the Conduit due to Dead Load in Case of Non-Uniform Dead Load</td>
<td>231</td>
</tr>
<tr>
<td>7.10</td>
<td>Vertical Pressure Distribution in the Soil in the Longitudinal Direction at the Invert Level due to Dead Load</td>
<td>232</td>
</tr>
</tbody>
</table>
NOMENCLATURE

\begin{itemize}
  \item \textbf{b} \quad \text{third invariant of deviatoric stress}
  \item \textbf{c} \quad \text{material cohesion}
  \item \textbf{D} \quad \text{conduit diameter}
  \item \textbf{D}_h, \textbf{D}_v \quad \text{dimensions relating to the conduit as defined in Fig. 2.1}
  \item \textbf{d} \quad \text{cohesion of the material in the p-t plane (Fig. 3.2)}
  \item \textbf{d}_e \quad \text{strain rate (Eq. 3.7)}
  \item \textbf{d}_e^{\text{el}} \quad \text{elastic strain rate}
  \item \textbf{d}_e^{\text{pl}} \quad \text{inelastic (plastic) strain rate}
  \item \textbf{E} \quad \text{modulus of elasticity}
  \item \textbf{F}_{\text{BM}} \quad \text{a design factor for modifying bending moment in multi-span conduits}
  \item \textbf{F}_m \quad \text{a reduction factor for modifying buckling stress in multi-span conduits}
  \item \textbf{f}_b \quad \text{buckling stress}
  \item \textbf{f}_e \quad \text{elastic buckling stress}
  \item \textbf{f}_y \quad \text{yield strength of the conduit wall}
  \item \textbf{h} \quad \text{soil cover above the crown level}
  \item \textbf{j} \quad \text{iteration number}
  \item \textbf{K} \quad \text{material parameter (Eq. 3.9)}
  \item \textbf{P}_c \quad \text{soil pressure at crown}
\end{itemize}
p equivalent pressure stress
q Mises equivalent stress
R radius of curvature of the conduit wall, at the mid-height of corrugations, at a transverse section
Re vector of external applied loads
Ri vector of internal resisting loads
Ru vector of unbalanced loads
r vector of nodal displacement
S spacing between multi-span conduits
Td maximum thrust in the conduit wall due to dead load
Tl maximum thrust in the conduit wall due to live load
Tmax maximum thrust in the conduit wall
t deviatoric stress measure (Eq. 3.8)
V buckling mode shape (eigenvector)
α dispersion angle in the longitudinal direction
β material angle of friction in the p-t plane (Fig. 3.2)
ΔK change in stiffness caused by live loads
θ dispersion angle in the span direction
λ eigenvalue
ν Poisson's ratio
σe° static yield stress in uniaxial compression
φ internal angle of friction

xxv
\( \psi \)  dilation angle (angle to the t-axis in the p-t plane, Fig. 3.2)

\([D]\) rigidity matrix of the general shell section

\([K]\) global stiffness matrix

\([K_e]\) global elastic stiffness matrix

\([K_\sigma]\) global geometric stiffness matrix

\([K_T]\) tangent stiffness matrix

\(\{P\}\) global load vector

\(\{U\}\) global displacement vector
CHAPTER I

INTRODUCTION

1.1 General

The last three decades have observed considerable progress in soil-steel structures, which increased in size and in numbers with the increase of information developed in this field. These structures are relatively easy to construct and are more economical than their conventional counterparts such as concrete box culverts and short-span concrete bridges. They are built using corrugated metal sheets and constructed so as to induce beneficial interaction between the conduit walls and the surrounding soil. The soil thus acts as an integral part of the structural system.

The theory of soil-steel structure assumes that after the completion of construction, the flexible steel conduit develops composite action with the surrounding soil. The conduit wall itself has relatively little inherent strength to sustain the loads, while its ability to support loads is due to the development of lateral soil pressure. As a result of the high degree of flexibility of the conduit wall, the loads imposed on the pipe are resisted mainly by membrane forces in the wall.

The methods of analysis and/or design of soil-steel structures vary in their degree of approximation and their practicality for everyday use. They range from the use of simple empirical formulas to rational semi-empirical approaches and elastic analysis to
numerical analysis such as finite element method in which both the material and geometric nonlinearity can be taken into account.

All the available methods are based on considering a slice of a unit width of the conduit wall and the surrounding soil. This approach has justification since the corrugated conduit walls have very low rigidities in the longitudinal direction when compared to those in the curved direction. It neglects the fact that the loading conditions may vary in the longitudinal direction, especially in the case of a live load acting over a shallow cover. The structure may also be subjected to a varying dead load due to the trapezoidal shape of the embankment and possible variation in the depth of cover.

1.2 The Need For Three-Dimensional Analysis

With the continuing trend of building larger structures with shallow covers, the need exists for a more accurate analysis for the behaviour of soil-steel structures. A comprehensive three-dimensional analysis can address problems such as:

i- The Load Dispersion in the Soil Cover

The live load effects are calculated by first considering load dispersion in the longitudinal direction of the conduit, and then analyzing a plane-strain slice of the structure (Abdel Sayed and Bakht, 1983; OHBDC, 1991). This approach oversimplifies the behaviour of the conduit, since the load dispersion occurs simultaneously in both directions;

ii- Load Transfer to the Ends of the Conduits

Some reports of failure in soil-steel structures suggest that failure may be
triggered at the ends of the conduit (Abdel-Sayed, 1989). At these locations, the soil cover is terminated over the conduit which suggests the possibility of load transfer between the middle section of the conduit and its ends which could explain this type of failure; and

iii- Multi-Span Conduits Interaction

It is recognized that the interaction between closely spaced conduits is affected by the spacing between them, their size and shape, as well as by the depth of cover. However, only empirical formulas or code requirements for the minimum spacing are available, with considerable differences between one and another (OHBDC, 1991; AASHTO, 1983). Proper three-dimensional analysis can determine the effect of spacing on the conduit performance especially for conduits subjected to unsymmetrical cases of loading.

1.3 Objectives of the Research

This dissertation presents a study of the behaviour of soil-steel structures using a three-dimensional finite element analysis. It is directed at the following:

1- To investigate the three-dimensional dispersion of live load through the soil cover;

2- To examine the end effects on the behaviour of the conduits (i.e., case of non-uniform dead load in the longitudinal direction); and

3- To study the effect of spacing between multi-span conduits on their load-carrying capacity. This study examines the effect of the conduits spacing on
their induced bending moment as well as on their stability.

In the course of this study, an experimental laboratory model was built and tested in order to verify the theoretical analysis.
CHAPTER II

LITERATURE SURVEY

2.1 General

The name "soil-steel structures" refers to structures built of metal flexible plates and embedded in the soil. These structures have been known also in the literature as: flexible conduits, buried pipes, structural plate culverts, composite soil-steel structures, or soil-steel bridges. The most common shapes of flexible conduits are shown in Fig. 2.1. These structures may be large diameter circular pipes, horizontal or vertical ellipses, pipe arches or pear-shaped pipes, re-entrant arches, semi-circular arches or part-arches. Figure 2.2 shows the terminology related to the cross-section of these structures.

2.2 Analysis and Design Procedures

An accurate procedure for the analysis of soil-steel structures should consider the soil and the steel shell as a composite three-dimensional, nonhomogeneous continuum. However, at the present time, all the methods developed to analyze and design the soil-steel structures are based on considering a slice of a unit width of the conduit and the surrounding soil. This assumption neglects the three-dimensional effect especially of the load variation in the longitudinal direction. It also neglects the end effects along the conduit where the soil cover changes from a maximum value at the middle section of the
conduit to zero at both ends as shown in Fig. 2.3. These two factors have a great effect on the behaviour of soil-steel structures and consequently on its load-carrying capacity.

The available methods of analysis and/or design vary in their degree of approximation. They range from the use of simple empirical formulas, semi-empirical approaches, and elastic analysis to numerical methods. A brief summary of these methods is presented in the following sections.

2.2.1 Marston-Spangler Theory

The Marston-Spangler approach was developed for small diameter circular conduits. It is comprised of Marston's estimation of the effective vertical load acting on the conduit (Marston, 1930), and Spangler's assumption for the load distribution around it (Spangler, 1941; 1960). It assumes uniform vertical soil pressures at the top and bottom of the pipe and parabolic horizontal pressures at the sides, with the maximum at the mid-height (Fig. 2.4). In the case of a uniform soil support provided by a well-compacted soil envelope, the maximum horizontal pressures on the sides are up to 35% greater than the vertical pressure on the top of the structure. In this Figure, $P_c$ is the soil pressure at the crown and $R$ is the radius of the conduit. Spangler concluded from his experimental investigation that if the vertical diameter of a circular flexible conduit decreases by about 20% from the initial diameter, the pipe is in a state of incipient collapse. Additional vertical load on the pipe causes failure due to reversal curvature or snap-through buckling. It became customary, then, to refer to failure conditions in flexible pipe as a 20% decrease in vertical diameter. Design is achieved by adjusting the
in-plane bending stiffness of the conduit to limit deflection to 5% or less of the diameter.

2.2.2 Ring Compression Theory

White and Layer (1960), suggested that the conduit wall of a soil-steel structure can be analyzed as a ring in uniform compression. A pure ring compression is developed in the conduit wall as the soil pressure adjusts itself with the deformation of the conduit. This theory implies the magnitude of the soil pressure to be in reverse proportion to the radius of curvature as illustrated in Fig. 2.5.

2.2.3 Frame on Elastic Supports

A simplified approach to analyze the soil-steel structures was proposed by Kloeppe and Glock (1970). This method models the structure as a plane-frame supported on springs which represent the backfill. The active soil pressure, which is assumed to act on the top portion of the conduit, is in the shape of a half sine wave as shown in Fig. 2.6. A linear analysis can therefore be conducted as well as a nonlinear analysis with formation of plastic hinges (Ghobrial and Abdel-Sayed, 1985).

2.2.4 Finite Element Method

The basic concept of the finite element method involves the subdivision of the total analytical model of the structure into subdomains of simple geometric form, i.e., into finite elements. The behaviour of each element is described by approximate functions, resulting first in algebraic relationships for the elements. Then, a large-order system of
algebraic equations is obtained upon connection of the elements to form the complete structure. Solutions of these algebraic equations lead to an approximate determination of the response of the structure to the applied loads. The main equation which represents the finite element theory is:

\[ [K_g] \{U\} + [K_d] \{U\} = \{P\} \tag{2.1} \]

where \( \{U\} \) is the global displacement vector; \( \{P\} \) is the global load vector; \([K_g]\) is the global elastic stiffness matrix; and \([K_d]\) is the global geometric stiffness matrix. Equation 2.1 can be simplified to the following form:

\[ [K] \{U\} = \{P\} \tag{2.2} \]

where \([K] = [K_g] + [K_d]\) is the global stiffness matrix.

Analysis has been conducted using the finite element method such as Katona et al. (1976), Hafez (1982), Desouki (1985), and Haggag (1989). More details about the finite element technique can be found in the literature such as Zienkiewicz (1977), Yang (1986), and Reddy (1984).

The finite element method has the ability to realistically model the full three-dimensional structures; however, such analysis requires a prohibitive amount of computer time. Thus, the three-dimensional analysis is usually reduced to a two-dimensional plane-strain analysis, i.e., the consideration of a unit width of the conduit wall and the surrounding soil. This analysis neglects the variation of the force effects along the conduit. The plane-strain analysis is not expected to yield accurate results especially for structures subjected to concentrated live loads.
The plane-strain analysis of a unit width of the structure neglects the variation of the height of fill above the conduit. However, the depth of cover is constant only above the middle section of the conduit, while the ends of the conduit are subjected to backfill of decreasing height. The three-dimensional analysis has been recommended in order to study the end effects on the behaviour of soil-steel structures (Abdel-Sayed, 1989; Mufti et al., 1989).

Moore (1988) studied the response of long thick elastic tubes to three-dimensional load systems. Exact solutions were given for the response of thin and thick tubes to bands and batches of radial pressure. However, his analysis has not been applied to the case of flexible soil-steel structures.

2.3 Live Load Dispersion in Soil

The live load dispersion in the soil above soil-steel structures has been examined before, using a plane-strain assumption. Abdel-Sayed and Bakht (1983) and Bakht (1981) examined the problem of live load dispersion in the soil. They concluded that the insertion of a flexible conduit in a half-space has the effect of changing the soil stiffness in the directions along or lateral to the conduit axis. Therefore, the load dispersion above the conduit may not neglect the presence of the conduit, i.e., it may not treat the embankment as being a half-space as allowed by the AASHTO (1983).

The live load dispersion along the flexible conduits has been examined by Abdel-Sayed and Bakht (1983) by analyzing a longitudinal unit-width slice of soil above the crown. The support provided by the metallic shell for the soil was simulated by
uniformly spaced springs. A dispersion ratio of two vertical to one horizontal has been recommended for the practical applications along the conduit.

On the other hand, the analytical results by Hafez (1982), who used a plane-strain finite element program, revealed that under concentrated loads at the embankment level, the vertical soil pressure at crown level is fairly widely distributed across the span. This observation had been confirmed by the experimental findings of Bakht (1981). A dispersion ratio of one vertical to one horizontal in the span direction has been recommended by Abdel-Sayed (1982).

These dispersion ratios have been adopted by the OHBDC (1991). On the other hand, the AASHTO (1983) specifies a dispersion ratio of one vertical to 0.875 horizontal in both directions without any consideration of the presence of the flexible conduit in the soil.

The above assumptions of live load dispersion ratios in the soil are examined in the present dissertation through a three-dimensional analysis which accounts for the simultaneous live load dispersion in both directions.

It is worth stating that contrary to what was concluded in the case of flexible soil-steel structures, Abdel-Karim et al. (1990) examined the live load dispersion through the soil in the case of rigid reinforced concrete box culverts. It was observed that the rate of load dispersion is virtually the same in both the longitudinal and transverse directions. This is mainly due to the fact that rigid culverts have comparable stiffness in both the transverse and longitudinal directions, which results in similar distribution characteristics in both directions.
2.4 Multi-Span Conduits

In the case of multi-span soil-steel structures, one of the common problems encountered by the designer is the minimum allowable spacing between the conduits. The OHBDC (1991) states that: "for a multi-conduit structures, the smallest of the clear spacings between adjacent conduits shall not be less than 1.0 m, nor less than one-tenth of the largest span". On the other hand, the AASHTO (1983) states that: "when multiple lines of pipes or pipe arches greater than 48 inches in diameter or span are used, they shall be spaced so that the sides of the pipe shall be no closer than one-half diameter or 3.0 feet, whichever is less". However, it may be noted that the OHBDC (1991) requires a reduction in the buckling strength of the conduit wall where the ratio of spacing to diameter falls between 0.1 and 0.5 (Abdel-Sayed et al., 1991).

A preliminary and simplified experimental study was conducted by Kung and Lau (1985), to examine the effect of spacing on the performance of the conduits. A total of three different spacings were used in the experiments with two different depths of cover. The deflections, change in diameter and the effect of spacing on the performance of the conduits were examined under two conditions of loading. In the first case, only one conduit was subjected to the applied load while in the second case both conduits were symmetrically loaded. Based on the deflection measurements, Kung and Lau concluded that the load carrying capacity increases with the increase in spacing between conduits.

The above experiments directed the attention for more studies to determine the effect of the spacing on the behaviour and load carrying capacity of multi-span conduits.
2.5 End Effects

Some reports of failure in soil-steel structures (Abdel-Sayed, 1989) have suggested that failure was triggered at the ends of the conduits. At that location, the section of the conduit is practically not covered by soil, i.e., it is not subjected to active dead load. Therefore, it is interesting to examine the possibility of load transfer between the middle section of the conduit and its ends which may explain this type of failure. Also, Mufti et al. (1989) concluded that the effect of uneven loading along the conduit length cannot be ignored.

The three-dimensional analysis has to be applied to study the end effects on the behaviour of these structures. Yet the author could not find any reference of such analysis in the present literature.

2.6 Stability of Soil-Steel Structures

The Ontario Highway Bridge Design Code (OHBDC, 1991) considers the buckling of the conduit wall as the only criterion used to determine the strength of the conduit wall and the ability of the conduit to sustain the imposed loading. Stresses induced by bending moment are not accounted for. In general, one of two approaches (Abdel-Sayed and Girges, 1992) have been followed to determine or to develop formulas to calculate the elastic buckling stress, $f_e$, namely: (i) plane-strain analysis of the conduit wall surrounded with a continuum of elastic medium (Forrestal and Herrmann, 1965; Moore, 1985), or (ii) plane analysis of the conduit wall supported by elastic springs which replace and simulate the effects of the surrounding soil (Luscher, 1966; Kloeppe and Glock, 1970; Ghobrial
2.6.1 The Elastic Continuum Theory

The elastic continuum model has been applied by Forrestal and Herrmann (1965) and by Moore (1985) to investigate the buckling of long cylindrical shells surrounded by elastic media. The loading exerted by the elastic medium on the shells in the buckling configuration was found by solving the boundary value problems of the linearized theory of elasticity in the presence of initial stress. It has been observed by Forrestal and Herrmann (1965) and Moore (1987) that the continuum model predicts pressures which are often too high. Moore accounted for this fact by applying a reduction factor equal to 0.55 in order to bring the theoretical results within a reasonable range of the reported test data. The reported high theoretical results can be attributed mainly to the fact that the above mentioned continuum analysis did not account for the actual manner in which loading is applied over the conduits. It is based on a linear formulation for the eigenvalue of uniform thrust over the conduit wall. However, the live load and the dead load due to soil cover constitute the active load which induces downwards displacement of the upper zone of the conduit (Fig. 2.7).

2.6.2 The Radial Spring Model

The radial spring model replaces the soil with equivalent spring supports for which the stiffness can be assumed either constant over the whole conduit (Kloeppe1 and Glock, 1970) or variable (Okeagu and Abdel-Sayed, 1984). The analysis can be nonlinear and
accounts for the formation of plastic hinges (Ghobrial and Abdel-Sayed, 1985) or it can be based on assumed hinges (Kloeppeel and Glock, 1970). Kloeppeel and Glock applied a radial spring model in which hinges are assumed to develop. This approach reduces the stability analysis of the conduit to that of an arch on elastic supports. Herein, an upper and a lower limit are obtained for the buckling stresses based on the assumed arrangement of hinges. The stability formulas of the OHBDC (1983) are based on the lower limit of the results of the Kloeppeel and Glock analysis. Ghobrial and Abdel-Sayed (1985) examined the formation of plastic hinges in conduits from which it was found that using the lower limit of the Kloppel and Glock analysis is too conservative. Therefore, the new OHBDC (1991) has been modified to relax the stability requirement within the range between the two limits.

The present approaches of stability study are all based on two-dimensional analysis. However, there is a need to examine the effect of live load dispersion as well as the soil pressure at the conduit ends on the buckling behaviour of soil-steel structures both in the case of a single conduit and of multi-span conduits.

2.7 Interface between Conduit Walls and Soil

The interaction between the soil and conduit wall at their interface has been examined in the literature. A realistic idealization is the one in which the interface shear between the conduit wall and the soil could develop through friction with provision for relative slip between the two materials.

Elling (1985) studied the influence of interface friction and tensile debonding on
stresses in buried cylinders. Structures with imperfect boundaries were analyzed by assuming stress functions for the soil field as well as for the buried cylinder. The normal stresses that exist at the soil cylinder interface were also represented through shape functions. The internal force components in the cylinder’s wall appear to be remarkably insensitive to conditions involving tensile debonding and interface friction, i.e., the tangential stress in the cylinder walls remains nearly the same whether an idealized, perfect interface or a nonidealized interface was assumed.

In order to avoid the complexity of interface elements, most of the studies on soil-steel structures have been conducted by assuming complete bond between the soil and the conduit wall, for example, Duncan (1975; 1976; 1979); Duncan et al. (1980). Burns and Richards (1964), Hoeg (1968), Elling et al. (1983) and Mufti et al. (1989).
CHAPTER III

THEORETICAL MODELLING

3.1 General

An accurate procedure for the analysis and design of soil-steel structures should consider the soil and the steel shell as a composite, three-dimensional and non-homogeneous continuum. The parameters governing the behaviour of the structure include the geometry of the system, load distribution, as well as the material properties of both the conduit wall and the soil.

The finite element method may be used for obtaining numerical solutions to boundary value problems in engineering. It has been developed for and has gained its widest utilization in application to problems in structural mechanics.

The single most important feature of the finite element analysis is its amenability to computer programming such that a single program can be written to deal with a wide variety of structural forms, materials, loading and boundary conditions. A general purpose finite element program ABAQUS (Hibbitt et al., 1989) has been used to study the behaviour of soil-steel structures especially in reference to the listed goals of this research (section 1.3). Different three-dimensional and two-dimensional elements, material models, loadings and boundary conditions have been applied. A general description of the finite element technique which has been used in this study is presented...
in the following sections.

3.2 Finite Element Program

The ABAQUS finite element package is a batch program, therefore, a "data deck" has to be assembled from model and history data. The model data defines the finite element model: elements, nodes, element properties, material definitions, etc. The history data defines the sequence of events or loadings for which the model's response is sought. This history is divided, by the user, into a sequence of steps. Each step is a period of response to a particular loading such as a static loading, a dynamic loading, etc. The definition of a step includes the procedure type (for example the static stress analysis), control parameters for the nonlinear solution procedures, the loading, and the output requests.

The state of the model at the end of a step becomes the initial condition for the start of the following step. For nonlinear analysis, the program increments and iterates as necessary depending on the degree of nonlinearity. The number of increments in one step, and the number of iterations in each increment are both generally limited in order to control the central processor unit time (CPU time).

3.3 Finite Element Procedure

Brief outlines are presented in the next sections for the static stress analysis, nonlinear iterative-incremental analysis, the eigenvalue (buckling) prediction, and material modelling.
3.3.1 Static Stress Analysis

A fundamental division of stress problems is into linear and nonlinear analysis. For a linear analysis, load cases are applied to a model whose response is obtained directly since the load and load effects are proportional. Nonlinear stress analysis may incorporate two sources of nonlinearity: namely material nonlinearity, and/or geometric nonlinearity.

3.3.2 Nonlinear Iterative-Incremental Analysis

Nonlinear finite element problems are usually solved by taking several linear steps, since the stiffness matrix itself is a function of displacements and the displacements are unknown, which makes it impossible to apply a one step solution for nonlinear structure. Different procedures have been proposed to solve nonlinear problems. The program uses the well-known Newton’s method as a numerical technique for solving the equilibrium equations.

The solution is obtained as a series of increments of loading, with iterations within each increment to obtain equilibrium. Each increment should be kept small in order to assure correct modelling of the structure behaviour. Newton’s method has a finite radius of convergence, which means that too large an increment can prevent any solution from being obtained. Thus, there is an algorithmic restriction on the increment size. The program provides both choices of automatic incrementation and direct user specification. The automatic incrementation in the program is based on extensive experience with a
wide range of problems, and therefore generally provides a reliable approach.

The program uses a scheme based on the maximum force residuals (force and moment tolerances), following each iteration. By comparing consecutive values of these quantities, the program determines whether convergence is likely in the number of iterations allowed by the analyst. One other ingredient in this algorithm is that a minimum increment size is specified. This prevents excessive computation in cases where buckling, limit load, or some modelling error causes the solution to stall.

For any displaced state of the structure, let $\mathbf{r}$ be the vector of nodal displacements; let $\mathbf{R}_i$ be the vector of internal resisting loads (i.e., the vector of loads in equilibrium with the internal forces of the structure); let $\mathbf{R}_e$ be the vector of external applied loads; and let $\mathbf{K}_T$ be the current tangent stiffness matrix of the structure. The vector of unbalanced loads, $\mathbf{R}_u$, is given by:

$$\mathbf{R}_u = \mathbf{R}_e - \mathbf{R}_i$$

(3.1)

which provides a measure of the solution error.

The iterative sequence for iteration is as follows:

$$\mathbf{R}_u^j = \mathbf{R}_e - \mathbf{R}_i^j$$

(3.2)

and

$$\Delta \mathbf{r}^j = (\mathbf{K}_T^j)^{-1} \mathbf{R}_u^j$$

(3.3)
\[ r^{(i+1)} = r^i + \Delta r^i \]  

(3.4)

\[ R_i^{(i+1)} = \text{function} \left( r^{(i+1)} \right) \]  

(3.5)

in which \( j \) = iteration number.

Using an estimated load as an upper limit the automatic increment scheme can be applied. This scheme is based on the convergence of the iteration process of each increment, until the specified load tolerance in \( R^i_u \) is achieved. If the number of iterations exceed the maximum allowed, the increment size is reduced by a factor of four. If this results in a smaller increment than specified as a minimum in the input, the run is terminated. More details about the applied nonlinear analysis are available in the literature (Powell and Simons, 1981; Bergan et al., 1978).

3.3.3 Eigenvalue (Buckling) Prediction

The program contains a capability for estimating the elastic buckling by eigenvalue extraction. The buckling load estimate is obtained as a multiplier of the "live" loads, which are added to a set of "dead" loads. The "dead" state of the structure represents the initial state to which the "live" loads are added. The response to the "live" loads (when added to the "dead" state) should be elastic up to the estimated buckling load for the eigenvalue estimates to be reasonable. At the initial analysis corresponding to the "dead" loading, the stiffness of the structure is stored. A small value of the "live" loading is then added in a step, and the differential stiffness of the structure is calculated.
The program estimates the magnification of the live load value that could cause collapse, assuming the change in stiffness to be proportional to the change in live load magnitude and an eigenvalue problem is presented in the following equation:

\[
(K + \lambda \Delta K) \{ V \} = \{ 0 \}
\]

(3.6)

where \(K\) is the stiffness under the "dead" loads, \(\Delta K\) is the change in stiffness caused by the "live" loads, \(\lambda\) is the live load magnification factor (the eigenvalue), and \(V\) is the buckling mode shape (the eigenvector). The buckling load is then estimated as the "dead" load, plus the live load multiplied by the eigenvalue, \(\lambda\), while the mode of collapse is the eigenvector, \(V\).

3.3.4 Material Modelling

The material library in ABAQUS is intended to provide comprehensive coverage of both linear and nonlinear, isotropic and anisotropic material models. It includes several models of elastic behaviour, the simplest being a linear elastic isotropic material where Young's modulus and Poisson's ratio are constant.

Most materials of engineering interest have an elastic initial response, i.e., the deformation is fully recoverable. If the load exceeds some limit -the "yield load"- the deformation is no longer fully recoverable, i.e., some part of the deformation remains as the load is removed. The plastic theories model the material's mechanical response as it undergoes such non-recoverable deformation. Most of these models are incremental, in which the mechanical strain rate is divided into an elastic part and a plastic (inelastic) part.
Incremental plasticity models are usually formulated in terms of:

i- A yield surface, which generalizes the concept of "yield load" into a test function which determines if the material response is purely elastic at a particular state of stress;

ii- A flow rule, which defines the inelastic deformation that occurs if the response of material is no longer purely elastic; and

iii- Some evolution laws that define the hardening, i.e., the yield and/or flow definitions change as inelastic deformation occurs.

The Extended Drucker-Prager plasticity model has been used for granular materials and is presented below.

3.3.4.1 Extended Drucker-Prager Model

The extended Drucker-Prager plasticity model is used in the present work to model granular materials such as sand, in which the tensile and compressive yield strengths are significantly different. It uses a smoothed Mohr-Coulomb yield surface, associated with inelastic flow in the deviatoric plane, and separate dilation (ψ) and friction (β) angles. This model is simple to handle numerically since it applies a smooth yield function which differs from the Mohr-Coulomb model. Perfect plasticity as well as isotropic hardening are offered within this model. Perfect plasticity means that the yield stress remains constant with plastic strain. Isotropic hardening means that the yield surface changes size uniformly in all directions, so that as plastic strain occurs the yield stress increases in all stress directions.
A linear strain rate decomposition is assumed, so that

\[ \dot{\varepsilon} = \dot{\varepsilon}^e + \dot{\varepsilon}^p \]  

(3.7)

where \( \dot{\varepsilon} \) is the total strain rate; \( \dot{\varepsilon}^e \) is the elastic strain rate; and \( \dot{\varepsilon}^p \) is the inelastic (plastic) strain rate.

The yield surface used with this model makes use of the three stress invariants, defined as the equivalent pressure stress, \( p \); the Mises equivalent stress, \( q \); and the third invariant of deviatoric stress, \( b \). A deviatoric stress measure, \( t \), is defined as:

\[ t = \frac{q}{2} \left[ 1 + \frac{1}{K} - \left( 1 - \frac{1}{K} \right) \left( \frac{b}{q} \right)^3 \right] \]  

(3.8)

where \( K \) is a material parameter that controls the dependence of the yield surface on the value of the intermediate principal stress, as shown in Figure 3.1. For the extended Druker-Prager model, the value of \( K \) is determined from the following equation:

\[ K = \frac{(3 - \sin \Phi)}{(3 + \sin \Phi)} \]  

(3.9)

in which \( K \) is defined as the ratio of the yield stress in triaxial tension to the yield stress in triaxial compression. \( K = 1 \) implies that the yield surface is the Von Mises circle in the deviatoric principal stress plane (the \( \pi \)-plane, refer to Fig. 3.1).

It is possible to use the model as an associated flow model (\( \psi = \beta \), where \( \psi \) is the angle to the \( t \)-axis in the \( p-t \) plane (Fig. 3.2), or as a non-associated flow in the \( p-t \) plane, in the sense that the flow is assumed to be normal to the yield surface in the \( \pi \)-plane. The classical Druker-Prager model is available by setting \( \psi = \beta \) and \( K = 1 \).
The yield surface is defined by:

\[ t - p \tan \beta - \left( 1 - \frac{1}{3} \tan \beta \right) \sigma^0_e = 0 \tag{3.10} \]

where \( \sigma^0_e \) is the static yield stress in uniaxial compression; \( \beta \) is the material angle of friction in the \( p-t \) plane as shown in Fig. 3.2. The intercept, \( d \), with the \( t \)-axis, represents the cohesion of the material where \( d = (1 - 1/3 \tan \beta) \sigma^0_e \). The yield surface equation (3.10) can be rewritten in the following form:

\[ t - p \tan \beta - d = 0 \tag{3.11} \]

The material's data are most commonly available from triaxial compression test. Also, the constants \( \beta \) and \( d \) used in this model can be obtained as functions of the internal angle of friction, \( \phi \), and cohesion \( c \) as follows:

\[ \beta = \tan^{-1} \left( \frac{6 \sin \phi}{3 - \sin \phi} \right) \tag{3.12} \]

and

\[ d = c \left( \frac{6 \cos \phi}{3 - \sin \phi} \right) \tag{3.13} \]

The input data in the finite element program includes \( \beta, d, K \) and \( \psi \). More details about the Extended Drucker-Prager model can be found in the literature (Drucker, 1959; Drucker et al., 1957; Drucker and Prager, 1952; Schofield and W orth, 1968; Britto and Gunn, 1987).
3.4 Finite Element Model

The following sections present a review of the finite element modelling applied in the present analysis. They outline the type of elements, their material properties, and the boundary conditions. The procedure of applying both the dead and live loads is presented. Finally, the nonlinear automatic incrementation control is discussed, including the force and moment tolerances together with the increment and iteration limits.

3.4.1 Element Choice and Convergence Test

The accuracy and convergence of the finite element solution depends on the type of elements, the assumed displacement functions, and the nodal degrees of freedom. The program has a large library of elements of which three-dimensional solid elements and shell elements were used in modelling the soil and the conduit wall, respectively, for the three-dimensional analysis. Plane-strain elements and beam elements were used for the two-dimensional analysis. Convergence tests were conducted for the selected elements starting with the two-dimensional mesh.

3.4.1.1 Two-Dimensional Mesh

Plane-strain four-node bilinear elements were used to model the soil elements for the two-dimensional analysis. The active degrees of freedom at each node are the displacements $u_x$ and $u_y$ as shown in Fig. 3.3a. A two-node, linear interpolation beam element is applied to model the conduit wall elements with three active degrees of freedom ($u_x$, $u_y$, and $\phi_z$) at each node, Fig. 3.3b.
A circular conduit of a diameter 7.6 m and a depth of cover of 2.0 m was modelled using the above elements, Fig. 3.4a. A refined mesh was studied by dividing every plane-strain element into four elements and the beam element into two elements as shown in Fig. 3.4b. Two more refined meshes were introduced with additional elements in the area of interest around the conduit as shown in Figs. 3.4c and 3.4d. The convergence results from the four meshes are shown in Table 3.1. These results include the maximum thrust, the bending moment at the crown, the deflection at the crown, and the eigenvalue. The results shown in Table 3.1 are due to live load only. The convergence curves showing the relationship between the degrees of freedom and the above values are shown in Figs. 3.5a to 3.5d. From the above results, mesh number 3 (Fig. 3.4c), was selected to model the two-dimensional behaviour of the composite soil-steel structures.

3.4.1.2 Three-Dimensional Mesh

The three-dimensional finite element modelling of the soil-steel structures was carried out by using an eight-node, linear displacement solid element (C3D8), with three active degrees of freedom \( (u_x, u_y, \text{ and } u_z) \) at each node (Fig. 3.6a). Also, a second solid element (C3D8R) with reduced integration was used in cases of stability analysis. The conduit wall elements were modelled by a four-node, reduced integration, doubly curved shell elements (S4R) with six degrees of freedom \( (u_x, u_y, u_z, \phi_x, \phi_y, \text{ and } \phi_z) \) at each node (Fig. 3.6b). A second shell element (S4R5) with five degrees of freedom \( (u_x, u_y, u_z \text{ and two in-surface rotations}) \) was also used.
The convergence of the three-dimensional analysis was examined by considering the models shown in Figs. 3.7a, 3.7b and 3.7c using the elements C3D8 and S4R5. The same conduit of diameter 7.6 m and depth of cover of 2.0 m was examined in this model. The convergence of the thrust, bending moment and deflection under concentrated live load was found to be acceptable. Thus the second mesh in Fig. 3.7b was chosen to conduct the required three-dimensional analysis.

A stability analysis for the soil-steel structure using the above elements (C3D8 + S4R5) showed no convergence for the eigenvalue $\lambda$. Therefore, another study was conducted by reducing the third dimension (Y-axis) to 18.0 m instead of 24.0 m to improve the aspect ratio of the elements.

Different combinations of elements were also considered in the following order:

- Element model No. 1: C3D8 + S4R5;
- Element model No. 2: C3D8R + S4R5;
- Element model No. 3: C3D8 + S4R; and
- Element model No. 4: C3D8R + S4R.

A uniformly distributed load was applied at the surface over the longitudinal direction in order to compare the results of the thrust, bending moment, deflection and eigenvalue with the results of the two-dimensional model as shown in Table 3.2.

It was observed that changing the shell element has practically no effect on the thrust, bending moment, and deflection, i.e., the convergence is controlled only by the type of the soil element. Accordingly, the element model No. 1 (C3D8 + S4R5), has been considered adequate and has been applied in the following analysis. Figures 3.8a,
3.8b and 3.8c show the convergence of the thrust, bending moment and deflection, respectively from the above study. For the eigenvalue, $\lambda$, the results listed in Table 3.2 show that the element model No. 4 (C3D8R + S4R), leads to reliable three-dimensional values when compared with the two-dimensional results as shown in Fig. 3.8d.

Therefore, for stability study, the element model No. 4 was used, while for all other kinds of analyses, the element model No. 1 was chosen. Also, it was found that for meshes No.3 and No. 4, an excessive CPU time is needed for the analysis especially when considering nonlinear material properties. For example, one trial using mesh No. 4 consumed approximately 37 hours (CPU time). Therefore, the second mesh (Fig. 3.7b), was recommended again for the present analyses.

3.4.2 Element Properties

The properties of the conduit wall and the soil elements are discussed below for the cases of elastic and elastic-plastic models.

3.4.2.1 Corrugated Conduit Wall

In the present study, the corrugation profile 152 x 51 mm was chosen to conduct the required analysis. The corrugation terminology and the profile properties are shown in Appendix A.

For the two-dimensional analysis, a rectangular cross-section was considered for the beam elements in such a way that the same area and moment of inertia of the original corrugated profile are maintained. The material properties for the steel were assumed to
be: Modulus of Elasticity, \( E = 200 \) GPa, Poisson’s ratio, \( \nu = 0.30 \), and Yield Strength, \( f_y = 228 \) MPa.

In the case of a three-dimensional analysis, the corrugated wall was modelled by a shell general element. This element can be used for the simplest case of a linear elastic shell or in modelling a highly orthotropic shell (for example, a corrugated steel panel). It combines both the section specifications and the material properties in the form of a rigidity matrix \([D]\). The axial, shear, bending and torsional rigidities were calculated according to Abdel-Sayed (1970), see Appendix B.

In order to determine the stresses, strains, and element forces for the shell elements, an orientation or a definition of a local system of axes is specified. This means that instead of getting the output components in the global X, Y, and Z directions, the output values are determined in a local system. When this orientation option was used with the shell elements, one of the local directions (\( X^1 \)) was identified and the projections of the other two local directions (\( Y^1 \) and \( Z^1 \)) onto the surface of the shell were taken as the local directions (1 and 2) on the surface. Figure 3.9a shows the global and local directions of the shell elements, while Fig. 3.9b shows the local 1 and 2 directions for the output components.

### 3.4.2.2 Soil Properties

The soil surrounding the conduit has been treated as an isotropic and homogeneous material. Two models were applied: the elastic model and the extended elastic-plastic Druker-Prager model. In the elastic model, the modulus of elasticity and Poisson’s ratio
have been defined. Referring to Winterkorn and Fang (1975) and Jumikis (1967), the modulus of elasticity was given in the range of 10-25 MPa (loose sand) to 50-80 MPa (dense sand), from which an average value of 35 MPa was assumed. Also, the range of Poisson's ratio for various soils as given in Winterkorn and Fang (1975), was 0.30-0.35 for sand, from which a value of 0.35 was assumed in the analysis.

The extended elastic-plastic Druker-Prager model is defined by the angle of internal friction, $\phi^0$; the cohesion, $c$; the ratio, $K$, of the flow stress in triaxial tension to the flow stress in triaxial compression; and the dilation angle, $\psi$, as defined in section 3.3.4.1 of this chapter. A value of $\phi = 35^\circ$ was assumed and low cohesion of 0.12 MPa was considered in the analysis. According to the assumed values of $\phi$ and $c$, and by using Equations 3.9, 3.12, and 3.13, the following values were introduced in the analysis: $\beta = 54.8^\circ$; $d = 0.25$ MPa; $K = 0.679$; and $\psi = 54.8^\circ$.

3.4.3 Degrees of Freedom Constraints

Different ways were used throughout this study to apply displacement and rotation constraints. The boundary conditions were used to specify certain displacements or rotations at the boundaries. Multi-point constraints was a second method used to impose certain constraints within the model itself.

3.4.3.1 Boundary Conditions

In the case of a two-dimensional model, Fig. 3.4a, the boundary conditions were assumed to be fixed at the bottom ($u_x = u_y = 0.0$), while no horizontal displacements were
allowed at the sides ($u_x = 0.0$). For the case of a three-dimensional model, Fig. 3.7a, the boundary conditions were assumed to be fixed at the bottom surface ($u_x = u_y = u_z = 0.0$): no horizontal displacement ($u_x = 0.0$) was allowed in the X-direction for the side surface parallel to the Y-Z plane; no horizontal displacement ($u_y = 0.0$) was allowed in the Y-direction for the front surface parallel to the X-Z plane; at the back surface parallel to the X-Z plane, symmetry was assumed ($u_y = \phi_x = \phi_z = 0.0$); and finally at the middle surface parallel to the Y-Z plane, a similar symmetry was assumed ($u_x = \phi_y = \phi_z = 0.0$). The rotation constraints were imposed only to the nodes of the conduit walls.

3.4.3.2 Multi-Point Constraints (MPC's)

This option allows constraint to be imposed between different degrees of freedom within the model. Different types of MPC's are used in this study. The first type, Fig. 3.10a, is a standard method for mesh refinement of first order elements. It keeps a node in a fixed position on a straight line between two nodes. The second type, Fig. 3.10b, is also a standard method for mesh refinement, but only for first order solid elements in three-dimensional modelling. It keeps a node in a fixed position on a bilinearly interpolated surface. The third MPC's type is used to join two nodes making the displacements equal but leaving the rotations. This type is used to join the soil elements to the conduit wall elements.

The constraint between different degrees of freedom was also imposed through a direct input of linear multi-point constraints (EQUATION), as follows:
\[ A_1 u_1 + A_2 u_2 + \ldots + A_n u_n = 0.0 \]

where \( u_{1,2,n} \) are the displacements and \( A_{1,2,n} \) are constants. In this equation, the variable \( u_1 \) is eliminated to impose the equation, and should therefore not be used to apply any boundary conditions, MPC, or equation.

3.4.4 Load Representation

The dead and live loads applied to the soil-steel structures are explained in the following sections. Also, the interaction between the soil and the conduit wall is discussed.

3.4.4.1 Dead Load

The construction of the soil-steel structures begins by bolting the sheets together to form a flexible steel conduit. At the same time, the bedding is pre-shaped to the invert radius of the conduit without compaction in order to allow structure relaxation into bedding. The backfill material is then placed and compacted in layers on each side of the conduit. Then the embankment materials are placed and compacted over the top of the structure.

Simulation of the construction process is available through the ABAQUS program by using the MODEL CHANGE option. This option is used to remove some elements during the analysis and to reactivate them later. The strain in reactivated elements corresponds to the total displacements of their nodes, not just the displacements since they were reactivated. This is incorrect for such processes since it does not allow adding a
new, strain free layer to a strained construction. To add unstrained elements to a strained model without requiring the same strain on both sides of the interface between the new and old elements, separate nodes must be used on the two sides of this surface. The EQUATION and BOUNDARY options need to be used to couple the displacements of these nodes correctly. This means that the number of nodes and degrees of freedom become approximately double. A two-dimensional mesh could be handled using this MODEL CHANGE option; however, for a three-dimensional model it was found that an extensive time is required to complete the analysis, especially for a nonlinear iterative-incremental analysis.

Since the main objective of this dissertation is to study the behaviour of the soil-steel structures under live loads, a simpler technique has been used to apply the dead load. The weight of the soil alone was applied to a section above the conduit in such a way that the maximum thrust in the conduit wall was found to be very close to the thrust predicted according to OHBDC (1991). The unit weight of the soil was assumed to be 18.85 kN/m$^3$ (120 lb/ft$^3$). The area loaded by the soil is shown in Fig. 3.11. The maximum thrust from the OHBDC (1991) is found to be 248.4 kN/m compared to 227.0 kN/m obtained from the finite element model. The predictions of the dead load thrust according to different methods are shown in Appendix C.

3.4.4.2 Live Load

The live load has been applied at the embankment of the three-dimensional model as four wheel loads, each of 70 kN (OHBDC 1983). In this case, the live load
dispersion in the soil has been a by-product of the analysis. However, in the two-dimensional model, the live load effect was calculated by considering the load dispersion first in the longitudinal direction and then by analyzing a plane-strain slice of the structure. Details of the live loads considered in the analysis are presented in chapter V.

3.4.4.3 Interface Between the Soil and the Conduit

The simulation of the interaction between the soil and the conduit wall was treated first in this study by using an interface element to join the soil and the conduit wall. Two different types of interface elements were tried. The first type was a simple two node interface element connecting one node on the soil side to another node on the conduit side. The second type was a two nodes per side interface element to be used with the plane-strain elements. The friction between the surfaces was simulated by introducing a friction coefficient and a shear stress limit. The analysis of the two-dimensional model using the above interface elements did not yield an acceptable results. However, the literature survey in chapter II (section 2.7), shows that other studies have been conducted by assuming a complete bond between the conduit wall and the soil. It may also be noted that the present work is mainly a comparative study for different cases either in three-dimensional or two-dimensional analyses. Therefore, an assumption of a perfect bond between the conduit and the soil is justified. The maximum thrust due to dead and live loads were compared to the results of other researchers as shown in Table 3.3. It has been found that the assumption of a complete bond leads to acceptable solutions.
3.4.5 Nonlinear Automatic Incrementation Control

The nonlinear automatic incrementation is controlled in the analysis by using different parameters. These parameters include the force and moment tolerances, the increment and iteration limits, and the buckling prediction parameters. A brief discussion of these parameters is presented in the next sections.

3.4.5.1 Force and Moment Tolerances

The value of the force tolerance (PTOL) is the basic tolerance measure for the solution of the equilibrium equations at each increment. All forces at all nodes (except those with prescribed displacements) must fall below this tolerance for the solution to be accepted; otherwise the program continues to iterate the increment as explained in section 3.3.2 of this chapter. Usually PTOL is set to a small fraction (10^-2-10^-4) of typical force value. In this study, it was assumed to be equal to 10^-3 of applied loads.

For the beam and shell elements, a moment tolerance (MTOL) is applied to serve the same function as PTOL. It is usually chosen as the PTOL value, multiplied by a typical element dimension.

3.4.5.2 Increment and Iteration Limits

The limits and parameters required to control the analysis are defined at the beginning of each step. INC is a parameter used to specify the maximum number of increments in a step. CYCLE is the maximum number of iterations in an increment. NLGEOM option is used to indicate that geometric nonlinearity should be accounted for
during the step. SUBMAX is a parameter used to suppress subdivision except when convergence is not achieved in the maximum number of iterations allowed. Initial increment size is defined to start the analysis. Also, a minimum increment size is introduced so that if a smaller increment is required, the analysis is terminated.

3.4.5.3 Buckling Prediction Control

A two stage process is required to obtain the eigenvalue for buckling load as explained in section 3.3.3 of this chapter. First, the "dead" load is applied, then the stiffness matrix is stored by inserting one step with a parameter BUCKLE DEAD. Following this step (and in the same run), the "live" load is applied as a small fraction of the expected critical load. This small live load is usually applied in a single step. Finally, the differential stiffness is calculated and the eigenvalue problem is calculated by using the parameter BUCKLE LIVE in a new step. The critical load is then calculated as the "dead" load, plus the eigenvalue multiplied by the change in load caused by the "live" load step between the BUCKLE DEAD step and the BUCKLE LIVE step.

Two complete input data files are presented for both two-dimensional and three-dimensional finite element models in a floppy disk attached to the back cover.
CHAPTER IV

MODEL VERIFICATION

4.1 General

The finite element models have been selected based on the study of convergence and the comparison between the two-dimensional and three-dimensional results described in chapter III. Also, experimental investigation has been conducted and the results of three laboratory tests are compared with the theoretical analysis for their verification. This verification is conducted prior to the application of the theoretical model in the analysis and parametric studies of the following chapters.

4.2 Laboratory Set-Up

Three tests were conducted in the structural laboratory at the University of Windsor. The first test was performed on a single circular conduit with a localized load applied at the grade in order to study the behaviour of the conduit, especially in reference to the load effects in the longitudinal direction. The second and third tests were conducted on two conduits with a ratio of spacing to diameter of 0.50 and 0.10, respectively, in order to experimentally examine the behaviour of multiple conduits. The load in these tests was applied locally on one conduit. The conduit walls were made of
corrugated aluminum in order to simulate the actual behaviour of the corrugated steel shells. The components of the laboratory tests are as follows:

4.2.1 Soil Container

A soil container, 2.7 m by 0.95 m by 1.25 m high, was built of 20. mm wood sheets supported by 75 x 50 x 5 mm steel angles. Only one side of the container was made of 12.7 mm plexiglass for visual observations.

4.2.2 Loading Set-Up

The load was applied and measured using a 222.4 kN capacity load cell in the first test and a 44.5 kN load cell in the second and third tests. It was applied over a 0.20 m x 0.40 m area at the soil surface at the middle length of the conduit as shown in Fig. 4.1. In the second and third tests, the load was applied above conduit No. 1 as shown in Figs. 4.2 and 4.3.

4.2.3 Conduits

Five circular conduits of corrugated aluminum sheets were manufactured by the Technical Support Centre at the University of Windsor. The 1.0 mm thick aluminum sheets were corrugated to the dimensions shown in Fig. 4.4. The conduit’s diameter was 0.60 m and the material used was utility aluminum (3003-H14) with a modulus of elasticity 70 GPa and a yield strength 137.9 MPa. The cross section area was 1.245 mm²/mm and the moment of inertia was 3.70 mm⁴/mm. The conduit ends were wrapped
with foam rubber to seal any gap between the conduit end and the sides of the container. In the same time, it was carefully observed that the sealing of the gap did not obstruct the free movement of the conduit wall at the ends.

4.2.4 Instrumentation

Two different cross sections were chosen to install the dial gauges. Six dial gauges were mounted at the middle of conduit No. 1 (section I-I), as shown in Fig. 4.5. At section II-II, i.e., at one quarter of the conduit, another set of six dial gauges were mounted as before, while only one dial gauge was mounted to measure the crown deflection at one quarter of the conduit length from the other side. An additional dial gauge was mounted to measure the deflection of the soil at the surface as shown in Fig. 4.8. In addition to the 14 dial gauges for conduit No. 1, a set of 7 dial gauges were mounted at the mid-length of conduit No. 2 in the second and third tests as shown in Fig. 4.5. One dial gauge was also used to measure the deflection of the soil above conduit No. 2, thus the total number of dial gauges was 22 in each test.

Electrical resistance strain gauges were used to monitor the strains at the same points of the dial gauges. At each location, one strain gauge was placed in the valley and the other in the ridge (see Fig. 4.4), making a total number of 26 strain gauges used for conduit No. 1 and 14 for conduit No. 2. All the strain indicators and dial gauges were zeroed before replacing any soil around the conduits. A multi-channel automatic digital strain indicator was connected to the strain gauges to record their readings during construction (after compaction of each layer), and under the live load.
4.2.5 Soil

The soil used in these tests was a mix of sand and gravel with the ratio of one to one by weight. The mixture was classified as SW (well graded gravelly sand), according to the unified soil classification system, and as A-1-b according to the AASHTO classification. A set of tests was conducted to determine the properties of the soil and included sieve analysis, shear box test, and an unconsolidated-undrained triaxial test. The properties obtained from these tests were used in the theoretical modelling of the laboratory experiments. These properties are as follows: \( E = 55 \text{ MPa} \); \( \nu = 0.36 \); \( \phi = 39^\circ \); and \( c = 0.06 \text{ MPa} \). Figure 4.7 shows the experimental set-up for the triaxial test. The unit weight of soil used in the triaxial tests was found to be equal to \( 19.6 \text{ kN/m}^3 \). The standard procedure of the triaxial test was followed as outlined in the literature (Bishop and Henkel, 1962; Bowles, 1978).

4.2.6 Compaction Procedure

A 260 mm thick soil layer was compacted on the bottom of the soil container. Then a 50 mm thick layer of a clean dry sand was pre-shaped to the invert radius of the conduit without compaction to allow structure relaxation into bedding. Then, the soil was placed in 150 mm thick layers as shown in Fig. 4.6. After reaching the crown of the conduit, two 80 mm thick layers were added on the top of the crown making the soil cover above the conduit equal to 160 mm. In the second and third tests, the same procedure was followed with the layout shown in Fig. 4.6. The average unit weight of the soil used in these tests was found to be equal to \( 18.7 \text{ kN/m}^3 \).
4.3 Experimental Procedure and Observations

The experimental procedure for each test is described in the next sections. Also, some observations during the tests and up to failure are discussed.

4.3.1 Test No. 1

The dimensions used in test No. 1 are shown in Fig. 4.1. After compacting the soil on the top of the conduit, the load was applied in small increments and the dial gauge readings and the strains were recorded at the end of each increment.

At the load of 11.12 kN, the strain readings showed that the stresses reached the yield strength at the shoulders. Increasing the load above that value created plastic hinges at the shoulders. The load was increased after that until it reached a maximum value of 13.34 kN. At that level, the load began to decrease while the deflection at the crown continued to increase.

Figure 4.8 shows the loaded area above the soil while Fig. 4.9 shows the dial gauges set-up. Figure 4.10 shows the first test after reaching the failure load. The deformed part of the conduit at the mid-length is shown in Fig. 4.11. A side view for the deflected conduit is shown in Fig. 4.12.

Figure 4.13 shows the deformed conduit under different increments of loads at sections I-I and II-II. The spring points started to move towards the soil while the crown and shoulders deflected away from the surrounding soil. The deflection at the crown of section I-I shows greater value than the one of section II-II.

The thrust in the conduit wall at sections I-I and II-II is shown in Fig. 4.14. At
section I-I, the magnitude of the thrust is found to be very small at the invert and increases in the upper half of the conduit wall. On the other hand, at section II-II, the thrust distribution in the conduit wall shows a tensile force at the crown which is due to the shell effects under the localized loading.

The bending moment distribution at sections I-I and II-II is shown in Fig. 4.15. It is clear that the bending moment at the shoulder is greater than its value at the crown. Also, the lower half of the conduit wall was subjected to low bending moment in comparison to the upper part.

4.3.2 Test No. 2

The multi-span conduits were examined in Test No. 2 as shown in Fig. 4.2. The spacing to diameter ratio S/D, was equal to 0.50. The soil bedding layer, the pre-shaped sand layers, and the two conduits are shown in Fig. 4.16. Figure 4.17 shows the dial and strain gauges in both conduits. The ten soil layers are constructed around and above the two conduits in the same sequence as shown in Fig. 4.6b. The final set up for the test is shown in Fig. 4.18 before the application of the live load.

By recording the dial and strain readings, it was clear that the maximum capacity of conduit No. 1 has been decreased to 11.21 kN. This capacity is less than the first test by about 16%. This is attributed to the effect of the adjacent conduit No. 2, since it provides less support to conduit No. 1 in comparison with that of the back fill. At load 11.21 kN, the load cell reading began to decrease in spite of the hydraulic supply while the deflection continued to increase. The strain readings showed that at load 9.92 kN, the
stresses at the left shoulder of conduit No. 1 reached the yield strength of the conduit wall. Increasing the load from 9.92 kN to 11.21 kN was accompanied by considerable increase in the stresses at the right shoulder of conduit No. 1 and at the crown of conduit No. 2. The failure in this test, is attributed to the formation of plastic hinges at the shoulders of conduit No. 1 and at the crown of conduit No. 2. Figure 4.19 shows the deformed conduit No. 1 after failure.

The deformed shape of both conduits at section I-I is shown in Fig. 4.20, while the deformed shape of conduit No. 1 at sections I-I and II-II is shown in Fig. 4.21. As expected the left spring point of conduit No. 2 has been deflected away from the soil due to the pressure coming from conduit No. 1. In the same time, the crown of conduit No. 2 has been deflected upward due to the lack of support from the soil above the crown level.

The thrust distribution in both conduits at section I-I is shown in Fig. 4.22. The maximum thrust in conduit No. 1 was found within the left shoulder, while for conduit No. 2, it existed at the crown with less value at the invert and some tension at the left spring point. Figure 4.23 shows the thrust distribution for conduit No. 1 at sections I-I and II-II where a tensile force has been created at the crown of section II-II similarly to the conditions in test No. 1.

The bending moment distribution for both conduits at section I-I is shown in Fig. 4.24. The maximum bending moment value was found at the left shoulder of conduit No. 1 where the plastic hinge started. Figure 4.25 shows the bending moment distribution for conduit No. 1 at sections I-I and II-II.
4.3.3 Test No. 3

The case of two conduits of narrow spacing of 0.10 of the diameter was examined in test No. 3 (Fig. 4.3). The same procedure was followed as outlined in test No. 2. The final set-up for the test is shown in Fig. 4.26.

The behaviour of the structure was identical as expected where the crown of conduit No. 1 was deflected, while the right spring line deflected very rapidly towards the soil pushing the left spring line of conduit No. 2. Also, the crown of conduit No. 2 was deformed upwards pushing the soil layer as recorded by the dial gauges. The load was applied in increments up to 9.42 kN, when the load cell reading started to decrease together with rapid increase in the deformations in both conduits. This failure load was found to be less than the failure load of test No. 1 by about 29% and less than the failure load of test No. 2 by about 16%.

The strain readings showed that the stresses at the left shoulder of conduit No. 1 reached the yield strength at one side of the corrugated sheet at load 8.66 kN. These stresses were not enough to create plastic hinges at the left shoulder or any other point. In other words, the failure here was due to excessive deformations when the load reached a value of 9.42 kN. Figure 4.27 shows the deformed shape of the two conduits at failure with a significant distortion in both conduits.

The deformed conduits at section I-I is shown in Fig. 4.28, which shows an excessive deformation at the crown and the spring lines of both conduits. The deformation of conduit No. 1 at sections I-I and II-II is shown in Fig. 4.29.

The thrust distribution at section I-I for both conduits is shown in Fig. 4.30. The

44

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maximum thrust for conduit No. 1 took place at the left shoulder while a tensile force was created at the crown of conduit No. 2. Figure 4.31 shows the thrust distribution of conduit No. 1 at sections I-I and II-II.

The bending moment distribution for both conduits at section I-I is shown in Fig. 4.32. A high value of bending moment at the left shoulder of conduit No. 1 exists, while a negative bending moment has been created at the crown of conduit No. 2. Figure 4.33 shows the bending moment distribution for conduit No. 1 at sections I-I and II-II.

The load-deflection curves for conduit No. 1 at the crown at section I-I are shown in Fig. 4.34 for the three tests. A comparison between the failure load and the load at first yield for the three different tests is shown in Table 4.1. Table 4.2 shows a comparison between the deflection results at load 8.5 kN for the three experimental tests. A similar comparison between the thrust and the bending moment results is shown in Tables 4.3 and 4.4.

4.4 Comparison between Experimental and Theoretical Results

The three laboratory tests have been analyzed following the theoretical modelling outlined in chapter III. Figure 4.35 shows the finite element model used for test No. 1. Only one quarter of the whole structure has been modelled. The load was applied as a uniform distributed load, as shown in Fig. 4.35. The case of two conduits was modelled, as shown in Figs. 4.36 and 4.37, for the S/D ratios of 0.50 and 0.10, respectively (i.e., tests No. 2 and No. 3, respectively). The same three-dimensional finite elements with the same aspect ratio as presented in chapter III were considered to conduct the required
comparison.

Since the failure in the experimental tests No. 1 and No. 2 was due to formation of plastic hinges, a comparison between the experimental and theoretical bending moment was conducted. Figure 4.38 shows the bending moment at the crown in the case of one conduit (test No. 1). The maximum difference between the experimental and the theoretical values is about 18%. Because of the fact that soil modelling is very difficult, taking into consideration the actual behaviour of soil as in the field, the 18% is considered a reasonable difference. Also, Figs. 4.39, 4.40, and 4.41 show a comparison between the experimental and the theoretical deflection, thrust, and bending moment in the conduit wall at section I-I at load 8.90 kN, where a reasonable agreement was achieved. The comparison between the experimental and theoretical bending moment at the crown of conduit No. 1 in the case of S/D of 0.50, is shown in Fig. 4.42. In this case the difference between the experimental and the theoretical results reaches up to about 20%.

Finally, in the case of two conduits with S/D ratio of 0.10 (test No. 3), the failure was due to excessive deformation which was accompanied by a great increase in the bending moment at the left shoulder. This test has been examined as shown in Fig. 4.43 by comparing the bending moment at the left shoulder theoretically and experimentally. The difference in this case between the theoretical and the experimental values is about 26%.

4.5 Observations

The experimental work presented in this chapter and the comparison of results
with the theoretical modelling leads to the following conclusions:

The laboratory tests show reasonable agreement with the results obtained analytically. This verifies the validity of using the proposed theoretical modelling in the following parametric studies. In addition, the laboratory tests demonstrated the effect of the spacing between multiple conduits on their performance. The load-carrying capacity of the conduits is thus reduced with the reduction of their spacing.
CHAPTER V

PARAMETRIC ANALYSIS OF SOIL-STEEL STRUCTURES

5.1 General

A theoretical parametric analysis is conducted in order to study the soil-steel structures in reference to the objectives listed in chapter I. The theoretical modelling, which was outlined in chapter III, has been verified theoretically and experimentally and has been used in all the parametric studies presented in this chapter and in the following two chapters. In the next sections of this chapter, the case of a single conduit is considered. A three-dimensional finite element model is built where the actual rigidities of the corrugated walls and the concentrated live loads are simulated. Through the results of this model, the dispersion of live load in soil above the conduit and the assumed code formulas for load dispersion (OHBDC, 1991; AASHTO, 1983) are evaluated.

Also, the internal thrust and bending moment around the conduit walls are presented together with a stability study for the buckling of a single conduit. All the mentioned analysis is repeated using a two-dimensional finite element model in order to compare its results with the three-dimensional modelling.

The multi-span conduits are studied in chapter VI using both three-dimensional and two-dimensional finite element models. The effect of spacing on the behaviour of soil-steel structures is demonstrated by studying the thrust and bending moments and the
buckling of these structures. In chapter VII, the end effects on the behaviour of soil-steel structures is examined through a three-dimensional finite element model. In this model, the dead load above the conduit is changed from a maximum at the middle section of the conduit to zero at the ends.

5.2 Analysis and Behaviour of a Single Conduit

A three-dimensional finite element model has been used to study the case of a single conduit, the overall dimensions are shown in Fig. 5.1. The conduit diameter was 7.6 m with a depth of cover equal to 2.0 m. The details about the theoretical modelling are presented in chapter III. The number of elements used in this model is 1104 and the number of degrees of freedom is 5148. The live load has been applied with two trucks side by side, i.e., two sets of four wheels of the OHBDC design truck each equal to 70 kN. This loading represents the maximum live load to be applied above the middle section of the conduit, Fig. 5.2.

To compare the three-dimensional with the two-dimensional results, a plane-strain finite element model was built, as shown in Fig. 5.3. The number of elements in this model was 272 with a total of 672 degrees of freedom. The same conduit diameter and depth of cover were considered. The details of the two-dimensional model are discussed in chapter III. The live load was applied as two concentrated loads at the embankment level after considering the dispersion in the longitudinal direction with a two vertical to one horizontal.
5.3 Live Load Dispersion in Soil

According to the OHBDC (1991), the equivalent distributed load at the crown level is obtained by assuming a live load dispersion at a one to one slope (a 45° slope angle) in the direction of the conduit span and at an angle of two vertical to one horizontal along the conduit length. On the other hand for the AASHTO (1983), the equivalent distributed load at crown level is obtained by assuming that the dispersed load is uniformly distributed over a square, the sides of which are equal to 1.75 h, where h is the soil cover above the crown level. This means that the live load disperses at an angle of one vertical to 0.875 horizontal along both the longitudinal and span directions.

It is interesting to investigate the above discrepancy using the results of the three-dimensional analysis, which account for the simultaneous dispersion in both directions.

5.4 Vertical Pressure Distribution in the Soil

The vertical pressure distribution in the soil due to the concentrated live loads has been investigated using both the three-dimensional and two-dimensional models. Figures 5.4 and 5.5 show the results of the three-dimensional analysis for the vertical pressure distribution respectively, at a depth of 1.0 m and at the crown. Figures 5.6 and 5.7 show the vertical pressure distribution in the soil from the three-dimensional model in the longitudinal direction and in the span direction, respectively, at the same two levels.

It is clear from the three-dimensional analysis, that the dispersion of the live load in the longitudinal direction is different from that in the span direction. Also, the concentrated load disperses over a greater length in the span direction than in the longitudinal direction. This result supports the conclusions of Bakht (1981) and Abdel-
Sayed and Bakht (1983). The vertical pressure distribution at the same two levels in the span direction from the plane-strain two-dimensional model is shown in Fig. 5.8. Comparing the curves in Figs. 5.7 and 5.8, it can be seen that the distribution of the soil pressure in the span direction from the three-dimensional analysis is generally different from the corresponding values in the two-dimensional analysis as shown in Table 5.1. The vertical pressure at crown level from the three-dimensional analysis is found to be equal to $1.77 \times 10^2$ MPa, while that from the two-dimensional analysis is $0.70 \times 10^2$ MPa. In the case of the three-dimensional analysis, there is a maximum vertical pressure under the traffic load with lower values all around, differing from that of the two-dimensional analysis, Fig. 5.9.

The analysis of vertical pressure has been carried out to evaluate the dispersion angles $\theta$ and $\alpha$, Fig. 5.10. The dispersion angles are defined for practical design as follows: a concentrated load might be assumed to act uniformly over an imaginary area, such that the load divided by that area leads to pressure equal to the maximum pressure at the crown level. The values of vertical pressure from the three-dimensional analysis at the crown and at 1.0 m depth above the conduit gave a dispersion angle $\theta$ in the span direction ranging from $43^0$ to $47^0$. An average value of $45^0$ (a slope of one to one) was chosen. This slope is the same one as applied by the OHBDC (1991). In the longitudinal direction, the slope $\alpha$ has been found to be equal to $80^0$, which is approximately a slope of five vertical to one horizontal. This value is steeper than the two vertical to one horizontal according to the OHBDC (1991), and further disagrees with the AASHTO (1983) load dispersion, which is too wide in the longitudinal direction.

It should be noted that the load dispersion is usually one step in the calculation...
of the induced force components. Therefore, it should be established together with the applied approach to calculate the induced forces: namely, the thrust and bending moment.

5.5 Thrust and Bending Moment around the Conduit Walls

The dispersion of live load in the soil is clarified here by comparing the thrust and bending moment around the middle section of the conduit. The three-dimensional results show the maximum thrust to be equal to 35.9 kN/m at the shoulder. The thrust at the crown was found to be 26.2 kN/m. The thrust within the lower half of the conduit was found to be negligible: it was approximately zero at the invert. Also, the bending moment at the crown was found to be equal to 223.5 x 10^3 kN.m/m. Within the shoulders, the maximum negative moment was found to be equal to 211.3 x 10^3 kN.m/m. Again, the bending moment due to live load within the lower half of the conduit was found to be very small.

A comparison between the two-dimensional results based on a two to one dispersion in the longitudinal direction, and those of the three-dimensional, is shown in Table 5.2. It is observed that the maximum thrust in the case of the three-dimensional model is lower than the two-dimensional maximum thrust by about 5%. Also, the thrust distribution is found to be more uniform in the three-dimensional model than the two-dimensional one as shown in Fig. 5.11. This may be attributed to the three-dimensional effect of dispersing the load simultaneously in both directions. The comparison of the bending moment, obtained from the three-dimensional and two-dimensional models, is shown in Fig. 5.12. The bending moment at the shoulder from the three-dimensional model is 20% lower than that of the two-dimensional model. Similarly, the bending
moment at the crown in the three-dimensional model is found to be less than half the value from the two-dimensional model due to the three-dimensional effect.

5.6 Stability of a Single Conduit

The three-dimensional finite element model, shown in Fig. 5.1 has been used to study the stability of a single soil-steel structure. Here, the effect of the concentrated live load dispersion in the longitudinal direction on the buckling load is accounted for. After applying both the dead and live loads, the eigenvalue \( \lambda \), was determined as explained in chapter III. The maximum thrust (\( T_{\text{max}} \)), at buckling can be obtained as the thrust due to dead load (\( T_D \)), plus the eigenvalue \( \lambda \), multiplied by the thrust due to live load (\( T_L \)). The elastic buckling stress (\( f_e \)), is determined as the maximum thrust divided by the cross-sectional area of the conduit wall. Also, the stability of a single conduit is examined by considering a slice of a unit width of the conduit and surrounding soil using the plane-strain finite element mesh, shown in Fig. 5.3. Here, the live load was first dispersed in the longitudinal direction.

A comparison between the two-dimensional and three-dimensional elastic buckling stresses shows that the three-dimensional modelling leads to about 10% higher stability load than the two-dimensional analysis. This difference is due to the fact that the two-dimensional analysis neglects any interaction with the unloaded sections of the structure and its resistance to the applied loads. On the other hand, the three-dimensional modelling takes into account the behaviour of the whole structure in both the span and longitudinal directions. Table 5.3 shows the results of the above study. Figures 5.13 and 5.14 show the first and second buckling modes of failure for the case of one conduit as
obtained from the two-dimensional analysis. The buckling configuration of Fig. 5.13 may be explained by Fig. 5.9 which shows that the pressure induced by the live load is maximum near the shoulders and not at the crown of the conduit. Also, one should notice that the displacements of the conduit is shown with a magnification factor of \(2.6 \times 10^3\).

5.7 Observations

From the study of a single conduit using the three-dimensional finite element model, the following observations can be drawn:

1- The three-dimensional analysis revealed a good agreement with the dispersion angle assumed by OHBDC (1991) in the span direction, while the assumed slope in the longitudinal direction is considered reasonable for practical applications;

2- The AASHTO assumption of the dispersion angle in the longitudinal direction is too far from the actual behaviour of these structures;

3- A high stress value takes place in the section under the loaded zone at the crown level in the soil contrary to the two-dimensional results;

4- The magnitude of the thrust is practically not affected when comparing the results of the three-dimensional and the two-dimensional models; however, significant reduction is found in the bending moment values, especially at the crown; and

5- The buckling load of a single conduit obtained from a three-dimensional analysis is found to be about 10% higher than the two-dimensional result.
6.1 General

The behaviour of multi-span conduits is analyzed in this chapter using both the three-dimensional and the two-dimensional finite element modelling. The analysis examines the effect of spacing between conduits on their behaviour, taking into consideration the two design criteria; namely, a) the induced bending moment, and b) the stability of the conduit walls. Also, both the elastic and elastic-plastic material models are considered.

6.2 The Elastic Model

The elastic material model has been used for both the conduit wall and the soil in this section. A conduit with a diameter of 7.6 m and a depth of 2.0 m cover was examined together with twin conduits with four different spacings. These spacings are 4.4, 3.2, 2.0, and 0.76 m, corresponding to a spacing to diameter ratio (S/D) of 0.58, 0.42, 0.26, and 0.10, respectively. The live load in each case was applied only over conduit No. 1 and the effect of conduit No. 2 on the behaviour of conduit No. 1 has been examined. The thrust and bending moment distributions for both conduits are presented.
6.2.1 Two-Dimensional Analysis

The two-dimensional analysis of the case of one conduit has been presented in chapter V using the plane-strain finite element mesh shown in Fig. 5.3. The OHBDC design truck was applied after having the concentrated load dispersed in the longitudinal direction and then applied to a slice of a unit width. This leads to two concentrated loads each equal to 31.8 kN and 1.2 m apart.

The plane-strain finite element models used for multi-span conduits are shown in Figs. 6.1, 6.2, 6.3, and 6.4 for the spacing to diameter (S/D) ratios of 0.58, 0.42, 0.26, and 0.10, respectively. The deformed models under dead and live loads for the cases of S/D equal to 0.58 and 0.10 are shown in Figs. 6.5 and 6.6, respectively. Table 6.1 shows a comparison of the maximum thrust, maximum positive bending moment at crown, and maximum negative bending moment at shoulder for the above cases of conduit No. 1. Also, Table 6.2 shows a similar comparison for conduit No. 2. The thrust and bending moment distribution for both conduits in the cases of S/D of 0.58 and 0.10 are shown in Figs. 6.7 and 6.8, respectively.

A comparison between the case of one conduit and the case of S/D of 0.10 for conduit No. 1 shows practically no change in the magnitude of the maximum thrust. However, the positive bending moment at the crown is increased by about 13%, while the negative bending moment at the shoulder is increased by about 29%.

6.2.2 Three-Dimensional Analysis

The details of the theoretical three-dimensional finite element modelling are
explained in chapter III. The case of one conduit has been examined using the mesh shown in Fig. 5.1 and the truck live loads shown in Fig. 5.2. The four multi-span cases are examined here using the finite element models shown in Figs. 6.9, 6.10, 6.11, and 6.12 for the cases of S/D ratio of 0.58, 0.42, 0.26, and 0.10, respectively. The live loads applied only to conduit No. 1. Tables 6.3 and 6.4 show the thrust and bending moment for both conduits at the middle section. The thrust and bending moment distribution for both conduits in the cases of S/D of 0.58 and 0.10 are shown in Figs. 6.13 and 6.14, respectively.

By inspection, the above results show that the conduit spacing has almost no effect on the magnitude of the thrust. However, with the narrow spacing of S/D ratio of 0.10, the positive bending moment is increased by about 43% while the negative bending moment is increased by about 17%. The thrust and bending moment of the unloaded conduit are considerably below the magnitudes of the loaded one, i.e., the design of the conduit walls is governed by the loading being applied above the conduit or on both conduits.

6.2.3 Observations

The relationship between the maximum thrust and S/D ratios is shown in Fig. 6.15 for both conduits and for both two-dimensional and three-dimensional models. Also, Figs. 6.16 and 6.17 show the relationships between the maximum positive and negative bending moments and the S/D ratios.

The study of the above results shows the following:
i- Narrow spacing between conduits has a considerable effect on the induced bending moment, since the adjacent and unloaded conduit provides supports with lower stiffness to the side of the loaded conduit;

ii- The three-dimensional elastic model shows the same behaviour of multi-span conduits as the two-dimensional analysis. However, the three-dimensional elastic model gives, in general, lower values for the thrust and the bending moments in comparison to the two-dimensional analysis; and

iii- The thrust does not change significantly in the walls of the conduit.

6.3 The Elastic-Plastic Model

The elastic-plastic material model has been considered for both the soil and the conduit wall in order to determine the load-carrying capacity of the conduits. The details of the elastic-plastic models are presented in chapter III. A nonlinear iterative-incremental analysis has been considered in examining the examples of section 6.2. The study started by considering the two-dimensional finite element plane-strain models and then the three-dimensional finite element models.

6.3.1 Two-Dimensional Analysis

The case of one conduit has been examined using the finite element mesh shown in Fig. 5.3. Figure 6.18 shows the deformed model at failure. Figure 6.19 shows the load-deflection curves for this case at both the crown and the top of the soil. The multi-span conduits have been considered using the finite element meshes shown in Figs. 6.1.
6.2, 6.3, and 6.4. Figure 6.20 shows the deformed model at the failure load for the case of S/D ratio of 0.10. The load-deflection curves at the crown are shown in Fig. 6.21 for a single conduit and multi-span conduits with S/D ratios of 0.58 and 0.10.

A comparison of the failure loads, maximum thrust, maximum positive bending moment at the crown, and maximum negative bending moment at the shoulder for conduit No. 1 is presented in Table 6.5. Also, a similar comparison for conduit No. 2 is shown in Table 6.6.

The above results show that the failure load of multi-span conduits decreases by about 30% in the case of a narrow spacing of S/D = 0.10, Fig. 6.22. This decrease in the load-carrying capacity is accompanied by a decrease of 24% in the maximum thrust and with an increase in the bending moment especially at the shoulder.

6.3.2 Three-Dimensional Analysis

The elastic-plastic three-dimensional analysis has been conducted using the finite element models mentioned in section 6.2.2. Tables 6.7 and 6.8 show the results of these models for both conduits. Figure 6.23 shows the load-deflection curves at the crown and at the top of soil for the one conduit case, while the load-deflection curves at the crown for the case of one conduit and the two cases of S/D ratios of 0.58 and 0.10 are shown in Fig. 6.24.

The results of the three-dimensional models show that the failure load of multi-span conduits is decreased with the decrease in spacing between conduits (Fig. 6.25), similar to the two-dimensional results. Also, the maximum thrust has been decreased by
about 24% while the bending moment has been considerably increased, especially at the shoulders.

6.3.3 Observations

The relationships between the maximum thrust and S/D ratios for both the two-dimensional and three-dimensional elastic-plastic models are shown in Fig. 6.26. Also, Figs. 6.27 and 6.28 show the relationships between the positive and negative bending moments and the S/D ratios.

The study of the elastic-plastic cases shows the following:

The design codes (OHBDC, 1991; AASHTO, 1983) specify a minimum spacing between adjacent conduits. These arbitrary requirements were developed since it is expected that narrow spacing between the conduits affects their load carrying capacity. This affects both the bending moment in the conduit walls as well as their stability. Section 6.4 of this chapter discuss the stability of multi-span soil-steel structures.

6.4 Stability of Soil-Steel Structures

The objective of this section is to examine the effect of the following parameters on the stability of the conduit walls:

i- Spacing between multiple conduits; and

ii- Elastic vs. elastic-plastic behaviour.
6.4.1 Spacing Between Multi-Span Conduits

The effect of spacing on the stability of the conduit walls has been examined by taking the simple case of an elastic model for both the soil and conduit's wall. The analysis has been conducted using two-dimensional and three-dimensional finite element models.

The study started by considering a slice of unit width of the conduit and surrounding soil using the plane-strain finite element method. The case of a single conduit has been studied using the finite element mesh shown in Fig. 5.3. The multi-span conduits have been examined considering a spacing to diameter ratio equal to 0.58, 0.42, 0.26, and 0.10. The finite element plane-strain meshes used in these cases are shown in Figs. 6.1, 6.5, 6.9 and 6.13, respectively.

After applying both the dead and live loads, the eigenvalue \( \lambda \), has been determined as explained in chapter III. The maximum thrust \( (T_{\text{max}})\), and the elastic buckling stresses \( (f_e) \), are compared for the above cases as shown in Table 6.9. The elastic buckling stress in the case of one conduit was found to be equal to 372 MPa, while in the case of a spacing to diameter ratio of 0.10, the elastic buckling stress decreased to 321 MPa, i.e., a reduction of about 14%. The first and second buckling shapes for the multi-span cases are shown in Fig. 6.29 through Fig. 6.36.

The above comparison shows that the presence of a second conduit has a considerable effect in reducing the elastic buckling stresses in the soil-steel structures. This reduction is due to the lack of support (i.e., less soil stiffness) in one side of the conduit in comparison to the other side. Therefore, the design of multi-span conduits
should consider this reduction especially when the buckling criterion is governing the
design (OHBDC, 1991).

In order to consider the effect of live load dispersion in the longitudinal direction
of the conduit, the three-dimensional models have been considered to study the effect of
spacing on the buckling load of soil-steel structures. The live load in these models was
applied as concentrated truck loads. The case of one conduit has been studied using the
three-dimensional finite element mesh shown in Fig. 5.1, while the different multi-span
conduits cases have been examined using the models shown in Figs. 6.9, 6.10, 6.11, and
6.12.

The results of the three-dimensional modelling are presented in Table 6.10,
showing that the presence of another conduit leads to a lower buckling load. This
reduction reaches a value of 9% when comparing the case of one conduit to the case of
an S/D ratio of 0.10. Figure 6.37 shows the relationship between the elastic buckling
stress ($f_e$), and the S/D ratio for both the two-dimensional and three-dimensional analysis.

6.4.2 Elastic vs. Elastic-Plastic Behaviour

The buckling of the soil-steel structures has been examined using an elastic-plastic
model for both the soil and the conduit wall. This study has been conducted using the
plane-strain finite element models mentioned in section 6.2.1 of this chapter. The results
of the maximum thrust and the buckling stresses ($f_b$) are shown in Table 6.11. It shows
that the buckling stress for the case of one conduit is found to be 221. MPa, while for the
case of an S/D ratio of 0.10, it is found to be equal to 169. MPa. This means that there
is a reduction of about 24% when comparing the above two cases. A comparison between the buckling stresses from the above study with the OHBDC buckling stress is shown in Table 6.12.

Table 6.13 shows another set of results in the case of the elastic conduit wall and elastic-plastic soil model using the two-dimensional finite element models. The buckling stress in the case of one conduit is found to be equal to 266 MPa while in the case of an S/D of 0.10, it is equal to 198 MPa with a total reduction of 25%.

6.5 Suggested Procedure for the Design of Multi-span Conduits

Since the finite element analysis using the elastic-plastic model shows a reduction in the capacity of soil-steel structures in case of multi-span conduits, the minimum spacing allowed by the codes needs to be reconsidered.

The analysis shows that the effect of narrow spacing between multi-span conduits is mainly in increasing the bending moment and in decreasing the buckling capacity of the conduit walls. Therefore, another technique to allow the present limitation of the spacing is suggested as follows:

6.5.1 Induced Bending Moment in the Conduit Walls

A suggested factor, $F_{BM}$, is introduced in order to increase the bending moment of multi-span conduits by using the following equation:
\[ F_{BM} = \left( 0.90 + \frac{0.05}{S/D} \right) \geq 1.0 \quad (6.1) \]

where \( F_{BM} \) is a factor to be applied in order to increase the bending moment of a single conduit so that:

- for \( S/D = 0.50 \), \( F_{BM} = 1.0 \), and
- for \( S/D = 0.10 \), \( F_{BM} = 1.4 \).

This means that for multi-span conduits with an \( S/D \) ratio equal to or greater than 0.5, there will be no increase in the bending moment, and for a \( S/D \) ratio equal to 0.10, there will be a 40% increase in the bending moment value.

The above suggested simplified equation is concluded, based on the extensive two-dimensional and three-dimensional finite element models presented in this chapter and on the experimental results presented in chapter IV.

### 6.5.2 Buckling Capacity of Soil-Steel Structures

In general, it can be observed that the buckling load of the conduit wall is reduced with the reduction of spacing between conduits. This effect may be expressed by the following formula:

\[ F_m = \left( 0.85 + 0.3 \frac{S/D}{\text{ }} \right) \leq 1.0 \quad (6.2) \]

in which \( F_m \) is a reduction factor for modifying buckling stress in multi-span conduits, where
for $S/D = 0.50 \quad F_m = 1.0$, and
for $S/D = 0.10 \quad F_m = 0.88$.

This means that for multi-span conduits with an $S/D$ ratio equal to or greater than 0.5, there is no reduction in the buckling stress, while for an $S/D$ ratio equal to 0.10, there is a 12% reduction in the buckling stress.

The above simplified equation is concluded based on the two-dimensional and three-dimensional finite element study presented in this chapter, and has been adopted by OHBDC (1991).
CHAPTER VII

END EFFECTS ON THE BEHAVIOUR OF SOIL-STEEL STRUCTURES

7.1 General

Practically all the available analyses of soil-steel structures are based on two-dimensional modelling of the structures, i.e., assuming no load transfer in the longitudinal direction. Their analysis and stability are usually obtained assuming equilibrium between the acting dead and live loads on the conduit wall and the soil reaction. However, it has been reported that some failures of conduits have been triggered at their ends where no dead or live loads are acting at the top of the conduit. It can be appreciated that load transfer develops in the longitudinal direction in spite of the high flexibility of the conduit walls and due to its interaction with the soil. A three-dimensional analysis of soil-steel structures is presented in this chapter in which the depth of cover above the conduit is varied from maximum at the middle part of the conduit to zero at the conduit edges.

7.2 Three-Dimensional Model

The three-dimensional behaviour of soil-steel structures, of non-uniform dead load, has been examined using the finite element mesh shown in Fig. 7.1. The dimensions of
this model are the same as the one used in chapter V in the case of uniform dead load. The loads applied in this case are the dead load and the truck live loads. The analysis of this model examines the circumferential thrust, the circumferential bending moment, the deflection, the vertical pressure distribution in the soil at the invert level, and finally the end effects on the stability of soil-steel structures.

7.3 Circumferential Thrust Variation Along the Conduit

The variation of the circumferential thrust along the conduit due to live load is shown in Fig. 7.2. The circumferential thrust at the crown is changed from a compression force to a tensile force some distance from the loaded middle section. This has also been observed with the laboratory models as explained in chapter IV. The circumferential thrust has almost vanished at the crown and at the invert in the conduit wall near the edges.

Figure 7.3 shows the circumferential thrust variation along the conduit due to dead load. As expected, the circumferential thrust at the crown decreases to zero at the edges where no soil cover exists. However, the circumferential thrust variation at the invert shows that while no dead load is acting above the conduit near the edges, considerable thrust takes place at the invert (i.e. 63. kN/m), which may be compared to 105. kN/m at the middle section. Figure 7.4 shows the circumferential thrust distribution due to dead load at the middle section and at the edges of the conduit.
7.4 Circumferential Bending Moment Variation Along the Conduit

The circumferential bending moment variation along the conduit due to live load is shown in Fig. 7.5. It shows that the circumferential bending moment is decreased at the edges for both the crown and the invert with a small negative bending moment at the crown. Figure 7.6 shows the circumferential bending moment variation along the conduit due to dead load. The bending moment at the invert is decreased from $349 \times 10^3$ kN.m/m at the middle section to $128 \times 10^3$ kN.m/m at the edges. The circumferential bending moment at the crown is changed from $175 \times 10^3$ kN.m/m at the middle section to a considerable negative bending moment of $-685 \times 10^3$ kN.m/m at the edges. Figure 7.7 shows the circumferential bending moment distribution due to dead load at the middle section and at the edges of the conduit wall.

7.5 Deflection Variation Along the Conduit

The deflection variation of the conduit wall at the crown and at the invert due to live load is shown in Fig. 7.8. This shows that the deflection is decreased at the edges away from the live load which is applied at the middle section. Figure 7.9 shows the deflection variation due to dead load. It shows that the deflection at the invert is decreased from 5.4 mm at the middle section to 2.7 mm at the edges while, at the crown, it is decreased from 14.8 mm at the middle section to almost nil at the edges.

7.6 Vertical Pressure Distribution in the Soil

The vertical pressure distribution in the soil in the longitudinal direction at the
invert due to dead load is shown in Fig. 7.10. The vertical pressure is decreased from $3.30 \times 10^{-2}$ MPa at the middle section to $2.10 \times 10^{-2}$ MPa at the edges. This reduction is equal to 36% although the dead load is reduced to nil at the edges. Also, this Figure shows that the dead load above the conduit is partially transmitted along the conduit, which in turn results in the high thrust at the invert and bending moment at the crown near the ends of the conduit, as shown in the previous sections.

### 7.7 End Effects on the Stability of Flexible Conduits

The three-dimensional modelling of a conduit with non-uniform dead load has been conducted in this study to determine the elastic buckling stress. The results of this case are presented in Table 7.1 with the results of the uniform dead load case (chapter V). Table 7.1 shows that the decrease in the dead load towards the edges of the conduit results in a reduction of about 11% in the buckling load of the conduit wall.

For comparison, the elastic buckling stresses have been calculated according to the OHBDC (1991) and the continuum theory (Moore et al., 1988), Table 7.2. This table shows that the continuum theory gives high values in comparison to the finite element results. This is attributed to the way of applying live loads to the conduit wall in the continuum theory. On the other hand, the OHBDC values are more conservative when compared to the results of the finite element. The calculations of the elastic buckling stresses according to the OHBDC (1991) and the continuum theory are shown in Appendix D.
7.8 Observations

The study of the soil-steel structures in the case of non-uniform dead load shows the following:

i- The vertical soil pressure at the invert due to dead load does not follow the variation of the dead load which usually decrease completely to nil at the edges;

ii- The circumferential thrust due to dead load at the invert at the edges has a significant value;

iii- The circumferential negative bending moment due to dead load at the crown at the edges is considerable in comparison to the positive bending moment at the middle section; and

iv- The thrust and bending moment values near the edges have the effect of reducing the buckling load of the whole structure.
CHAPTER VIII

CONCLUSIONS AND RECOMMENDATIONS

8.1 General

All the methods which so far have been developed to analyze and design the soil-steel structures are based on the consideration of a slice of a unit width of the conduit and the surrounding soil. Here, the three-dimensional analysis is usually avoided because of its extensive requirements of data preparation and computer processing time. However, the need exists for a more accurate approach to study these structures. Three-dimensional finite element modelling has been used throughout this study in order to evaluate the validity of the two-dimensional analysis especially in reference to the following:

i- The behaviour and stability of a single conduit;

ii- The effect of spacing on the behaviour and stability of multi-span conduits; and

iii- The behaviour and stability of conduits as affected by its ends where the dead load is terminated.

The finite element modelling has been verified through three laboratory tests. The analysis led to the following conclusions:

8.1.1 The Behaviour of A Single Conduit

The study of a single conduit using a three-dimensional model has shown the
The three-dimensional analysis revealed a good agreement with the dispersion angle assumed by OHBDC (1991) in the span direction, while the assumed slope in the longitudinal direction is reasonable for practical applications;

2- The AASHTO assumption of the dispersion angle in the longitudinal direction is too far from the actual behaviour of these structures;

3- A high stress value takes place in the section under the loaded zone at the crown level in the soil; and

4- The magnitude of the thrust is practically not affected when comparing the results of the three-dimensional and the two-dimensional models; however, significant reduction is found in the bending moment values, especially at the crown.

8.1.2 Multi-Span Conduits

The theoretical and experimental study of multi-span conduits leads to the following:

1- Closely spaced conduits are considerably affected by one another due to the fact that the stiffness of their support to one another could be lower than the soil support of the outer sides;

2- The reduction in the load-carrying capacity is mainly due to the increase in the bending moment especially at the shoulders, as well as to the reduced buckling limit; and

3- Design factors have been suggested as a function of the spacing between conduits in order to account for the increase in the bending moment, and/or the decrease in the buckling limit. It should be noted that the suggested values of these factors are based on the limited analysis conducted through this present research.
8.1.3 The End Effects (Non-Uniform Dead Load)

The three-dimensional finite element modelling of the case of non-uniform dead load along the conduit has shown the following:

1- The vertical pressure in the soil at the invert due to dead load does not change too much along the conduit although the dead load is decreased completely at the edges;

2- The circumferential thrust due to dead load at the invert has significant values near the edges;

3- The circumferential negative bending moment due to dead load at the crown near the edges is considerable in comparison to the positive bending moment at the middle section; and

4- The thrust and bending moment near the edges have the effect of reducing the buckling load of soil-steel structures.

8.1.4 Stability of Soil-Steel Structures

The stability of soil-steel structures using the finite element method leads to the following conclusions:

1- The buckling load of a single conduit obtained from a three-dimensional analysis is about 10% higher than the two-dimensional result;

2- The buckling load in the case of one conduit with non-uniform dead load was found to be 11% less than the case of uniform dead load; and

3- The buckling load in the case of multi-span conduits has been decreased especially with the decrease in spacing between the conduits.
8.2 Recommendations for Future Work

The present investigation leads to the following recommendations for future work:

i- A new method for design of soil-steel structures in order to account for the bending moment or the plastic hinges formulation;

ii- More studies for the case of non-uniform dead load in order to improve the load-carrying capacity of the conduit walls;

iii- More studies for the multi-span conduits by considering different heights of soil above the conduits and different conduit shapes; and

iv- Study the behaviour of soil-steel structures under dynamic loading.
Table 3.1: Convergence Results of the Two-Dimensional Analysis

Due to Concentrated Live Loads

<table>
<thead>
<tr>
<th>Mesh No.</th>
<th>Mesh No. 1</th>
<th>Mesh No. 2</th>
<th>Mesh No. 3</th>
<th>Mesh No. 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Elements</td>
<td>44.</td>
<td>160.</td>
<td>272.</td>
<td>688.</td>
</tr>
<tr>
<td>Number of Nodes</td>
<td>60.</td>
<td>192.</td>
<td>320.</td>
<td>768.</td>
</tr>
<tr>
<td>Number of D.O.F.</td>
<td>128.</td>
<td>400.</td>
<td>672.</td>
<td>1600.</td>
</tr>
<tr>
<td>Thrust*</td>
<td>-22.2</td>
<td>-30.8</td>
<td>-33.1</td>
<td>-34.8</td>
</tr>
<tr>
<td>B.M.&quot; at Crown</td>
<td>89.2</td>
<td>241.5</td>
<td>416.7</td>
<td>552.3</td>
</tr>
<tr>
<td>Deflection***</td>
<td>2.23</td>
<td>2.47</td>
<td>2.65</td>
<td>2.71</td>
</tr>
<tr>
<td>Eigenvalue</td>
<td>39.1</td>
<td>87.7</td>
<td>112.5</td>
<td>127.6</td>
</tr>
</tbody>
</table>

Note: * Thrust in kN/m

** Bending moment in 10^{-3} kN.m/m

*** Deflection in mm
Table 3.2: Convergence Results of the Three-Dimensional Analysis Due to Uniformly Distributed Live Load

<table>
<thead>
<tr>
<th>Mesh No.</th>
<th>Mesh No. 1</th>
<th>Mesh No. 2</th>
<th>Mesh No. 3</th>
<th>Mesh No. 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Elements</td>
<td>88.</td>
<td>640.</td>
<td>2176.</td>
<td>4352.</td>
</tr>
<tr>
<td>Number of Nodes</td>
<td>170.</td>
<td>918.</td>
<td>2856.</td>
<td>5394.</td>
</tr>
<tr>
<td>Number of D.O.F.</td>
<td>585.</td>
<td>2997.</td>
<td>9435.</td>
<td>17820.</td>
</tr>
<tr>
<td>C3D8+S4R5</td>
<td>-305.1</td>
<td>-346.7</td>
<td>-376.7</td>
<td>-380.3</td>
</tr>
<tr>
<td>C3D8R+S4R5</td>
<td>-286.3</td>
<td>-369.</td>
<td>-398.2</td>
<td>-393.2</td>
</tr>
<tr>
<td>C3D8+S4R</td>
<td>-305.1</td>
<td>-346.7</td>
<td>-376.</td>
<td>-381.</td>
</tr>
<tr>
<td>C3D8+S4R5</td>
<td>801.9</td>
<td>936.9</td>
<td>1075.7</td>
<td>1110.9</td>
</tr>
<tr>
<td>C3D8R+S4R5</td>
<td>1645.2</td>
<td>1204.1</td>
<td>826.8</td>
<td>776.8</td>
</tr>
<tr>
<td>C3D8+S4R</td>
<td>801.9</td>
<td>936.9</td>
<td>1075.7</td>
<td>1110.9</td>
</tr>
<tr>
<td>C3D8R+S4R</td>
<td>1645.2</td>
<td>1203.1</td>
<td>825.3</td>
<td>775.3</td>
</tr>
<tr>
<td>C3D8+S4R5</td>
<td>36.05</td>
<td>37.44</td>
<td>38.13</td>
<td>38.33</td>
</tr>
<tr>
<td>C3D8R+S4R5</td>
<td>42.62</td>
<td>40.55</td>
<td>39.21</td>
<td>39.01</td>
</tr>
<tr>
<td>C3D8+S4R</td>
<td>36.05</td>
<td>37.44</td>
<td>38.13</td>
<td>38.34</td>
</tr>
<tr>
<td>C3D8R+S4R</td>
<td>42.62</td>
<td>40.55</td>
<td>39.21</td>
<td>39.02</td>
</tr>
<tr>
<td>C3D8+S4R5</td>
<td>4.55</td>
<td>5.4</td>
<td>36.27</td>
<td>37.34</td>
</tr>
<tr>
<td>C3D8R+S4R5</td>
<td>3.14</td>
<td>5.13</td>
<td>30.48</td>
<td>31.69</td>
</tr>
<tr>
<td>C3D8+S4R</td>
<td>65.74</td>
<td>50.65</td>
<td>39.34</td>
<td>37.34</td>
</tr>
<tr>
<td>C3D8R+S4R</td>
<td>9.43</td>
<td>18.12</td>
<td>29.68</td>
<td>31.68</td>
</tr>
</tbody>
</table>

Note: * 2-D thrust = \(-383.3\) kN/m  
* 2-D bending moment = \(1280.2 \times 10^3\) kN.m/m  
* 2-D deflection = 38.5 mm  
* 2-D eigenvalue = 33.44
Table 3.3: Maximum Thrust Due to Dead and Live Loads

<table>
<thead>
<tr>
<th></th>
<th>Maximum Thrust*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dead Load</td>
</tr>
<tr>
<td>OHBDC (1983)</td>
<td>-179.1</td>
</tr>
<tr>
<td>Hafez and Abdel-Sayed</td>
<td>-156.</td>
</tr>
<tr>
<td>(1983)</td>
<td></td>
</tr>
<tr>
<td>OHBDC (1991)</td>
<td>-248.4</td>
</tr>
<tr>
<td>Present Study</td>
<td>-227.</td>
</tr>
</tbody>
</table>

Note: * Thrust in kN/m
Table 4.1: Comparison Between the Failure Load and the Load at First Yield for the Three Tests (Experimental Results)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Failure Load</th>
<th>Load at First Yield</th>
<th>Load*</th>
<th>% Difference</th>
<th>Load*</th>
<th>% Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test No. 1</td>
<td>13.34</td>
<td>11.12</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Test No. 2</td>
<td>11.21</td>
<td>9.92</td>
<td>11.12</td>
<td>-16%</td>
<td>9.92</td>
<td>-10%</td>
</tr>
<tr>
<td>Test No. 3</td>
<td>9.42</td>
<td>8.66</td>
<td></td>
<td>-29%</td>
<td></td>
<td>-22%</td>
</tr>
</tbody>
</table>

Note: * Load in kN
Table 4.2: Comparison Between Deflection* Results for the Three Experimental Tests at Load 8.5 kN

<table>
<thead>
<tr>
<th>Point</th>
<th>Conduit No. 1</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Test No. 1</td>
<td>Test No. 2</td>
<td>Test No. 3</td>
</tr>
<tr>
<td>Crown</td>
<td>-5.86</td>
<td>-8.08</td>
<td>-11.21</td>
</tr>
<tr>
<td>Right Shoulder</td>
<td>-1.31</td>
<td>1.18</td>
<td>1.49</td>
</tr>
<tr>
<td>Left Shoulder</td>
<td>-0.21</td>
<td>0.35</td>
<td>0.49</td>
</tr>
<tr>
<td>Right Spring</td>
<td>2.21</td>
<td>3.40</td>
<td>8.43</td>
</tr>
<tr>
<td></td>
<td>Conduit No. 2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Crown</td>
<td>-</td>
<td>2.85</td>
<td>6.72</td>
</tr>
<tr>
<td>Left Shoulder</td>
<td>-</td>
<td>-0.36</td>
<td>-4.72</td>
</tr>
<tr>
<td>Left Spring</td>
<td>-</td>
<td>-1.97</td>
<td>-8.41</td>
</tr>
<tr>
<td>Left Haunch</td>
<td>-</td>
<td>0.01</td>
<td>0.26</td>
</tr>
</tbody>
</table>

Note: * Deflection in mm

79
Table 4.3: Comparison Between Thrust* Results for the Three Experimental Tests at Load 8.5 kN

<table>
<thead>
<tr>
<th>Point</th>
<th>Conduit No. 1</th>
<th>Conduit No. 2</th>
<th>Conduit No. 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Test No. 1</td>
<td>Test No. 2</td>
<td>Test No. 3</td>
</tr>
<tr>
<td>Crown</td>
<td>-12.7</td>
<td>-11.8</td>
<td>-7.5</td>
</tr>
<tr>
<td>Right Shoulder</td>
<td>-5.3</td>
<td>-16.4</td>
<td>-9.8</td>
</tr>
<tr>
<td>Left shoulder</td>
<td>-9.9</td>
<td>-21.0</td>
<td>-20.2</td>
</tr>
<tr>
<td>Right Spring</td>
<td>-5.3</td>
<td>-4.2</td>
<td>-5.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Test No. 2</td>
<td>Test No. 3</td>
</tr>
<tr>
<td>Crown</td>
<td>-</td>
<td>-5.5</td>
<td>8.9</td>
</tr>
<tr>
<td>Left Shoulder</td>
<td>-</td>
<td>-0.4</td>
<td>-0.8</td>
</tr>
<tr>
<td>Left Spring</td>
<td>-</td>
<td>1.1</td>
<td>0.7</td>
</tr>
<tr>
<td>Left Haunch</td>
<td>-</td>
<td>-2.3</td>
<td>-3.1</td>
</tr>
</tbody>
</table>

Note: * Thrust in kN/m
Table 4.4: Comparison Between Bending Moment* Results for the Three Experimental Tests at Load 8.5 kN

<table>
<thead>
<tr>
<th>Point</th>
<th>Conduit No. 1</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Test No. 1</td>
<td>Test No. 2</td>
<td>Test No. 3</td>
</tr>
<tr>
<td>Crown</td>
<td>29.6</td>
<td>24.3</td>
<td>19.9</td>
</tr>
<tr>
<td>Right Shoulder</td>
<td>-77.6</td>
<td>-56.8</td>
<td>-34.2</td>
</tr>
<tr>
<td>Left shoulder</td>
<td>-92.6</td>
<td>-103.8</td>
<td>-147.5</td>
</tr>
<tr>
<td>Right Spring</td>
<td>0.01</td>
<td>-3.5</td>
<td>-51.9</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Point</th>
<th>Conduit No. 2</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>-</td>
<td>Test No. 2</td>
<td>Test No. 3</td>
</tr>
<tr>
<td>Crown</td>
<td>-</td>
<td>-23.1</td>
<td>-44.8</td>
</tr>
<tr>
<td>Left Shoulder</td>
<td>-</td>
<td>3.8</td>
<td>19.3</td>
</tr>
<tr>
<td>Left Spring</td>
<td>-</td>
<td>15.8</td>
<td>57.8</td>
</tr>
<tr>
<td>Left Haunch</td>
<td>-</td>
<td>-11.5</td>
<td>-16.1</td>
</tr>
</tbody>
</table>

Note: * Bending moment in $10^3$ kN.m/m
Table 5.1: Comparison of the Vertical Pressure in the Soil for Three-Dimensional and Two-Dimensional Models

<table>
<thead>
<tr>
<th></th>
<th>Three-Dimensional Model</th>
<th>Two-Dimensional Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>At depth 1.0 m</td>
<td>2.80</td>
<td>1.80</td>
</tr>
<tr>
<td>At the crown</td>
<td>1.77</td>
<td>0.70</td>
</tr>
</tbody>
</table>

Note: * Pressure in $10^{-2}$ MPa.
Table 5.2: Comparison of Thrust and Bending Moment for Three-Dimensional and Two-Dimensional Models Due to Live Load

<table>
<thead>
<tr>
<th></th>
<th>Three-Dimensional Model</th>
<th>Two-Dimensional Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thrust*</td>
<td>-35.9</td>
<td>-37.5</td>
</tr>
<tr>
<td>B.M. at crown**</td>
<td>223.5</td>
<td>590.5</td>
</tr>
<tr>
<td>B.M. at shoulder**</td>
<td>-211.3</td>
<td>-252.7</td>
</tr>
</tbody>
</table>

Note: * Thrust in kN/m

** Bending moment in $10^{-3}$ kN.m/m
Table 5.3: Comparison of the Elastic Buckling Stresses
(Three-Dimensional and Two-Dimensional Analysis)

<table>
<thead>
<tr>
<th></th>
<th>Three-Dimensional Model</th>
<th>Two-Dimensional Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \lambda )</td>
<td>95.178</td>
<td>81.144</td>
</tr>
<tr>
<td>( T'_D )</td>
<td>168.4</td>
<td>193.4</td>
</tr>
<tr>
<td>( T'_L )</td>
<td>35.9</td>
<td>37.6</td>
</tr>
<tr>
<td>( T'_{\max} )</td>
<td>3585.</td>
<td>3245.</td>
</tr>
<tr>
<td>( f_e^{**} )</td>
<td>412.</td>
<td>372.</td>
</tr>
</tbody>
</table>

Note: * Thrust in kN/m
** Elastic Buckling Stress in MPa
Table 6.1: Comparison of Maximum Thrust and Bending Moment of Conduit No. 1 for Different S/D Ratios
(Two-Dimensional Analysis & Elastic Model)

<table>
<thead>
<tr>
<th>S/D</th>
<th>Thrust*</th>
<th>Positive B.M.**</th>
<th>Negative B.M.**</th>
</tr>
</thead>
<tbody>
<tr>
<td>One Conduit</td>
<td>-37.5</td>
<td>590.5</td>
<td>-252.7</td>
</tr>
<tr>
<td>S/D = 0.58</td>
<td>-37.3</td>
<td>603.4</td>
<td>-255.1</td>
</tr>
<tr>
<td>S/D = 0.42</td>
<td>-37.3</td>
<td>616.5</td>
<td>-256.2</td>
</tr>
<tr>
<td>S/D = 0.26</td>
<td>-37.5</td>
<td>637.1</td>
<td>-259.1</td>
</tr>
<tr>
<td>S/D = 0.10</td>
<td>-37.7</td>
<td>669.9</td>
<td>-324.9</td>
</tr>
</tbody>
</table>

Note: * Thrust in kN/m

** Bending moment in $10^{-3}$ kN.m/m
Table 6.2: Comparison of Maximum Thrust and Bending Moment of Conduit No. 2 for Different S/D Ratios
(Two-Dimensional Analysis & Elastic Model)

<table>
<thead>
<tr>
<th>S/D</th>
<th>Thrust*</th>
<th>Positive B.M.**</th>
<th>Negative B.M.**</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.58</td>
<td>-13.8</td>
<td>106.2</td>
<td>-64.9</td>
</tr>
<tr>
<td>0.42</td>
<td>-16.5</td>
<td>144.4</td>
<td>-83.4</td>
</tr>
<tr>
<td>0.26</td>
<td>-20.5</td>
<td>211.9</td>
<td>-111.9</td>
</tr>
<tr>
<td>0.10</td>
<td>-26.</td>
<td>330.1</td>
<td>-223.5</td>
</tr>
</tbody>
</table>

Note:  * Thrust in kN/m

** Bending moment in $10^3$ kN.m/m
Table 6.3: Comparison of Maximum Thrust and Bending Moment of Conduit No. 1 for Different S/D Ratios
(Three-Dimensional Analysis & Elastic Model)

<table>
<thead>
<tr>
<th>S/D</th>
<th>Thrust*</th>
<th>Positive B.M.**</th>
<th>Negative B.M.**</th>
</tr>
</thead>
<tbody>
<tr>
<td>One Conduit</td>
<td>-35.9</td>
<td>223.5</td>
<td>-211.3</td>
</tr>
<tr>
<td>S/D = 0.58</td>
<td>-35.3</td>
<td>228.7</td>
<td>-211.9</td>
</tr>
<tr>
<td>S/D = 0.42</td>
<td>-35.4</td>
<td>252.4</td>
<td>-212.3</td>
</tr>
<tr>
<td>S/D = 0.26</td>
<td>-35.9</td>
<td>286.2</td>
<td>-219.8</td>
</tr>
<tr>
<td>S/D = 0.10</td>
<td>-36.0</td>
<td>318.6</td>
<td>-246.4</td>
</tr>
</tbody>
</table>

Note: * Thrust in kN/m

** Bending moment in $10^3$ kN.m/m
Table 6.4: Comparison of Maximum Thrust and Bending Moment of Conduit No. 2 for Different S/D Ratios (Three-Dimensional Analysis & Elastic Model)

<table>
<thead>
<tr>
<th></th>
<th>Thrust*</th>
<th>Positive B.M.**</th>
<th>Negative B.M.**</th>
</tr>
</thead>
<tbody>
<tr>
<td>S/D = 0.58</td>
<td>-5.4</td>
<td>36.1</td>
<td>-23.4</td>
</tr>
<tr>
<td>S/D = 0.42</td>
<td>-7.1</td>
<td>51.4</td>
<td>-37.6</td>
</tr>
<tr>
<td>S/D = 0.26</td>
<td>-11.0</td>
<td>65.6</td>
<td>-66.1</td>
</tr>
<tr>
<td>S/D = 0.10</td>
<td>-13.2</td>
<td>177.2</td>
<td>-111.3</td>
</tr>
</tbody>
</table>

Note: * Thrust in kN/m  
** Bending moment in $10^3$ kN.m/m
Table 6.5: Comparison of Failure Load, Thrust, and Bending Moment of Conduit No. 1 for Different S/D Ratios
(Two-Dimensional Analysis & Elastic-Plastic Model)

<table>
<thead>
<tr>
<th>S/D</th>
<th>Load* (kN)</th>
<th>Thrust** (kN/m)</th>
<th>Positive B.M.*** (kN.m/m)</th>
<th>Negative B.M.*** (kN.m/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.58</td>
<td>3118.</td>
<td>-1923.</td>
<td>20.8</td>
<td>-4.6</td>
</tr>
<tr>
<td>0.42</td>
<td>2819.</td>
<td>-1773.</td>
<td>21.7</td>
<td>-8.3</td>
</tr>
<tr>
<td>0.26</td>
<td>2647.</td>
<td>-1710.</td>
<td>22.4</td>
<td>-9.9</td>
</tr>
<tr>
<td>0.10</td>
<td>2488.</td>
<td>-1613.</td>
<td>24.0</td>
<td>-12.8</td>
</tr>
<tr>
<td>0.10</td>
<td>2189.</td>
<td>-1470.</td>
<td>25.2</td>
<td>-14.6</td>
</tr>
</tbody>
</table>

Note:  
* Load in kN  
** Thrust in kN/m  
*** Bending moment in kN.m/m
Table 6.6: Comparison of Maximum Thrust and Bending Moment of Conduit No. 2 for Different S/D Ratios
(Two-Dimensional Analysis & Elastic-Plastic Model)

<table>
<thead>
<tr>
<th>S/D</th>
<th>Thrust*</th>
<th>Positive B.M.**</th>
<th>Negative B.M.**</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.58</td>
<td>-783.</td>
<td>5.7</td>
<td>-4.7</td>
</tr>
<tr>
<td>0.42</td>
<td>-872.</td>
<td>9.7</td>
<td>-8.1</td>
</tr>
<tr>
<td>0.26</td>
<td>-981.</td>
<td>14.7</td>
<td>-12.5</td>
</tr>
<tr>
<td>0.10</td>
<td>-1084.</td>
<td>20.9</td>
<td>-16.0</td>
</tr>
</tbody>
</table>

Note:  * Thrust in kN/m
** Bending moment in kN.m/m
Table 6.7: Comparison of Failure Load, Thrust, and Bending Moment of Conduit No. 1 for Different S/D Ratios
(Three-Dimensional Analysis & Elastic-Plastic Model)

<table>
<thead>
<tr>
<th></th>
<th>Load*</th>
<th>Thrust**</th>
<th>Positive B.M.***</th>
<th>Negative B.M.***</th>
</tr>
</thead>
<tbody>
<tr>
<td>One Cinduit</td>
<td>5815.</td>
<td>-1620.</td>
<td>14.8</td>
<td>-3.6</td>
</tr>
<tr>
<td>S/D = 0.58</td>
<td>4951.</td>
<td>-1562.</td>
<td>15.5</td>
<td>-5.1</td>
</tr>
<tr>
<td>S/D = 0.42</td>
<td>4660.</td>
<td>-1435.</td>
<td>16.1</td>
<td>-5.9</td>
</tr>
<tr>
<td>S/D = 0.26</td>
<td>4480.</td>
<td>-1278.</td>
<td>18.5</td>
<td>-7.1</td>
</tr>
<tr>
<td>S/D = 0.10</td>
<td>4303.</td>
<td>-1239.</td>
<td>20.2</td>
<td>-8.4</td>
</tr>
</tbody>
</table>

Note: * Load in kN
     ** Thrust in kN/m
     *** Bending moment in kN.m/m
Table 6.8: Comparison of Maximum Thrust and Bending Moment of Conduit No. 2 for Different S/D Ratios
(Three-Dimensional Analysis & Elastic-Plastic Model)

<table>
<thead>
<tr>
<th>S/D</th>
<th>Thrust*</th>
<th>Positive B.M.**</th>
<th>Negative B.M.**</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.58</td>
<td>-657.</td>
<td>2.7</td>
<td>-2.9</td>
</tr>
<tr>
<td>0.42</td>
<td>-761.</td>
<td>3.6</td>
<td>-4.2</td>
</tr>
<tr>
<td>0.26</td>
<td>-969.</td>
<td>4.5</td>
<td>-6.0</td>
</tr>
<tr>
<td>0.10</td>
<td>-1027.</td>
<td>12.2</td>
<td>-9.2</td>
</tr>
</tbody>
</table>

Note:  * Thrust in kN/m
** Bending moment in kN.m/m
Table 6.9: Comparison of the Elastic Buckling Stresses for Different S/D Ratios (Two-Dimensional Analysis)

<table>
<thead>
<tr>
<th></th>
<th>$\lambda$</th>
<th>$T_d'$</th>
<th>$T_L'$</th>
<th>$T_{\text{max}}$</th>
<th>$f_{c''}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>One Conduit</td>
<td>81.144</td>
<td>193.4</td>
<td>37.6</td>
<td>3245.</td>
<td>372.</td>
</tr>
<tr>
<td>S/D = 0.58</td>
<td>79.153</td>
<td>192.9</td>
<td>37.3</td>
<td>3145.</td>
<td>361.</td>
</tr>
<tr>
<td>S/D = 0.42</td>
<td>77.445</td>
<td>192.0</td>
<td>37.3</td>
<td>3081.</td>
<td>353.</td>
</tr>
<tr>
<td>S/D = 0.26</td>
<td>74.146</td>
<td>192.7</td>
<td>37.5</td>
<td>2973.</td>
<td>341.</td>
</tr>
<tr>
<td>S/D = 0.10</td>
<td>69.289</td>
<td>189.9</td>
<td>37.7</td>
<td>2802.</td>
<td>321.</td>
</tr>
</tbody>
</table>

Note: * Thrust in kN/m

** Buckling Stress in MPa
Table 6.10: Comparison of the Elastic Buckling Stresses for Different S/D Ratios under Truck Loading

(Three-Dimensional Analysis)

<table>
<thead>
<tr>
<th></th>
<th>λ</th>
<th>T_D^*</th>
<th>T_L^*</th>
<th>T_max</th>
<th>$f_c^{**}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>One Conduit</td>
<td>95.178</td>
<td>168.4</td>
<td>35.9</td>
<td>3585.</td>
<td>412.</td>
</tr>
<tr>
<td>S/D = 0.58</td>
<td>94.509</td>
<td>171.1</td>
<td>35.7</td>
<td>3543.</td>
<td>407.</td>
</tr>
<tr>
<td>S/D = 0.42</td>
<td>91.226</td>
<td>175.6</td>
<td>35.4</td>
<td>3405.</td>
<td>391.</td>
</tr>
<tr>
<td>S/D = 0.26</td>
<td>88.655</td>
<td>172.0</td>
<td>35.7</td>
<td>3337.</td>
<td>383.</td>
</tr>
<tr>
<td>S/D = 0.10</td>
<td>85.797</td>
<td>169.3</td>
<td>36.0</td>
<td>3258.</td>
<td>374.</td>
</tr>
</tbody>
</table>

Note: * Thrust in kN/m

** Buckling Stress in MPa
Table 6.11: Comparison of the Buckling Stresses for Different S/D Ratios (Two-Dimensional Analysis & Elastic-Plastic Model)

<table>
<thead>
<tr>
<th></th>
<th>Maximum Thrust*</th>
<th>$f_b^{**}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>One Conduit</td>
<td>1923.</td>
<td>221.</td>
</tr>
<tr>
<td>S/D = 0.58</td>
<td>1773.</td>
<td>204.</td>
</tr>
<tr>
<td>S/D = 0.42</td>
<td>1710.</td>
<td>196.</td>
</tr>
<tr>
<td>S/D = 0.26</td>
<td>1613.</td>
<td>185.</td>
</tr>
<tr>
<td>S/D = 0.10</td>
<td>1470.</td>
<td>169.</td>
</tr>
</tbody>
</table>

Note:  
* Thrust in kN/m  
** Buckling stress in MPa
Table 6.12: Comparison of the Buckling Stresses between OHBDC and Two-Dimensional Analysis (Elastic-Plastic Model)

<table>
<thead>
<tr>
<th></th>
<th>( f_b ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>2-D Analysis</strong></td>
<td>221.</td>
</tr>
<tr>
<td><strong>OHBDC</strong></td>
<td></td>
</tr>
<tr>
<td>( \rho = 0.725 )</td>
<td>157.</td>
</tr>
<tr>
<td>( \rho = 1.0 )</td>
<td>177.</td>
</tr>
</tbody>
</table>

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Table 6.13: Comparison of the Buckling Stresses for Different S/D Ratios (Elastic Conduit Wall & Elastic-Plastic Soil Model) *(Two-Dimensional Analysis)*

<table>
<thead>
<tr>
<th>S/D</th>
<th>Maximum Thrust*</th>
<th>$f_b^{**}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>One Conduit</td>
<td>2314.</td>
<td>266.</td>
</tr>
<tr>
<td>S/D = 0.58</td>
<td>2029.</td>
<td>233.</td>
</tr>
<tr>
<td>S/D = 0.42</td>
<td>2015.</td>
<td>231.</td>
</tr>
<tr>
<td>S/D = 0.26</td>
<td>1939.</td>
<td>223.</td>
</tr>
<tr>
<td>S/D = 0.10</td>
<td>1724.</td>
<td>198.</td>
</tr>
</tbody>
</table>

Note: * Thrust in kN/m

** Buckling stress in MPa
Table 7.1: Comparison of the Elastic Buckling Stresses for the Cases of Uniform and Non-Uniform Dead Load
(Three-Dimensional Analysis)

<table>
<thead>
<tr>
<th></th>
<th>Uniform D.L.</th>
<th>Non-Uniform D.L.</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\lambda$</td>
<td>95.178</td>
<td>84.914</td>
</tr>
<tr>
<td>$T_{D^*}$</td>
<td>168.4</td>
<td>174.7</td>
</tr>
<tr>
<td>$T_{L^*}$</td>
<td>35.9</td>
<td>35.6</td>
</tr>
<tr>
<td>$T_{\max}$</td>
<td>3585.</td>
<td>3198.</td>
</tr>
<tr>
<td>$f_e^{**}$</td>
<td>412.</td>
<td>367.</td>
</tr>
</tbody>
</table>

Note:  
* Thrust in kN/m  
** Buckling Stress in MPa
Table 7.2: Comparison of the Elastic Buckling Stresses

<table>
<thead>
<tr>
<th>Method</th>
<th>( f_b ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-D Analysis</td>
<td>372.</td>
</tr>
<tr>
<td>3-D Analysis</td>
<td></td>
</tr>
<tr>
<td>Uniform D.L.</td>
<td>412.</td>
</tr>
<tr>
<td>Non-Uniform D.L.</td>
<td>367.</td>
</tr>
<tr>
<td>OHBDC</td>
<td></td>
</tr>
<tr>
<td>( \rho = 0.725 )</td>
<td>184.</td>
</tr>
<tr>
<td>( \rho = 1.0 )</td>
<td>253.</td>
</tr>
<tr>
<td>Continuum Theory</td>
<td>581.</td>
</tr>
</tbody>
</table>
Fig. 2.1 The Most Common Shapes of Flexible Conduits
(after OHBDC 1991)
Fig. 2.2 Illustration of Terminology Relating to the Cross-Section of the Structure (after Abdel-Sayed et al., 1993)
Fig. 2.3 Variable Soil Cover along the Conduit
Fig. 2.4 Pressure Distribution assumed in the Marston-Spangler Theory

(after Abdel-Sayed et al., 1993)
Fig. 2.5 Theoretical Interface Radial Pressures around Varies Conduits: 
(a) Circular Conduit; (b) Vertically-Elliptical Conduit; 
and (c) Pipe-Arch (after Abdel-Sayed et al., 1993)
Fig. 2.6 Plane-Frame on Elastic Supports

(after Abdel-Sayed et al., 1993)
Fig. 2.7 Assumed Loading for the Stability Analysis:
(a) Actual Live Load above the Conduit; and (b) Elastic Continuum Theory

106
Fig. 3.1 Typical Yield Surfaces in the Deviatoric Plane for the Granular Model
(after Hibbitt et al., 1989)

\[ t = \frac{1}{2} q \left[ \left( 1 + \frac{1}{K} \right) - \left( 1 - \frac{1}{K} \right) \left( \frac{b}{q} \right)^3 \right] \]

<table>
<thead>
<tr>
<th>Curve</th>
<th>K</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>1.0</td>
</tr>
<tr>
<td>b</td>
<td>0.8</td>
</tr>
<tr>
<td>c</td>
<td>0.5</td>
</tr>
<tr>
<td>d</td>
<td>0.2</td>
</tr>
</tbody>
</table>
Fig. 3.2 Schematic Diagram of the p-t Plane for the Granular Model

(after Hibbitt et al., 1989)
Fig. 3.3 Two-Dimensional Finite Elements:
(a) Plane-Strain, Four-Node, Bilinear Element; and
(b) Two-Node, Linear-Interpolation Beam Element
Fig. 3.4b Two-Dimensional Plane-Strain Model (Mesh No. 2)
Fig. 3.4d Two-Dimensional Plane-Strain Model (Mesh No. 4)
Fig. 3.5a  Thrust Convergence Curve of the Plane–Strain Analysis
Fig. 3.5b  Bending Moment Convergence Curve of the Plane–Strain Analysis
Fig. 3.5c  Deflection Convergence Curve of the Plane—Strain Analysis

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Fig. 3.5d Eigenvalue Convergence Curve of the Plane-Strain Analysis
Element C3D8

active degrees of freedom at each node: $U_x, U_y, U_z$

(a)

Element S4R

active degrees of freedom at each node: $U_x, U_y, U_z, \phi_x, \phi_y, \phi_z$

(b)

Fig. 3.6 Three-Dimensional Finite Elements:
(a) Eight-Node, Linear-Displacement Solid Element; and
(b) Four-Node, Doubly-Curved Shell Element
Fig. 3.7a Three-Dimensional Finite Element Model (Mesh No. 1)
Fig. 3.7b Three-Dimensional Finite Element Model (Mesh No. 2)
Fig. 3.8a  Convergence Curve of the Maximum Thrust using Shell Element Type S4R5 and Solid Element Type C3D8

(Maximum Thrust = 383.3 kN/m in Two-Dimensional Analysis)
Fig. 3.8b Convergence Curve of the Maximum Positive Bending Moment using Shell Element Type S4R5 and Solid Element Type C3D8
Fig. 3.8c Convergence Curve of the Deflection at Crown using Shell Element Type S4R5 and Solid Element Type C3D8

Deflection at Crown = 36.5 mm in Two-Dimensional Analysis

Number of Degrees of Freedom

Deflection in mm

42 41 40 39 38 37 36 35 34

0 4000 8000 12000 16000 20000
Fig. 3.8d Convergence Curve of the Eigenvalue using Shell Element Type S4R and Solid Element Type C3D8R

(Eigenvalue = 33.4 in Two-Dimensional Analysis)
Fig. 3.9 Definition of a Local Axis System for the Output Components of the Shell Elements
Fig. 3.10 MPC Used for Mesh Refinement in Case of:
(a) Plane-Strain Analysis; and (b) Three-Dimensional Analysis

(after Hibbitt et al., 1989)
Fig. 3.11 Dead Load Area above the Conduit
Fig. 4.1 Details of Laboratory Test No. 1
(Case of One Conduit)
Fig. 4.2 Details of Laboratory Test No. 2
Case of Two Conduits (S/D = 0.50)
Fig. 4.3 Details of Laboratory Test No. 3
Case of Two Conduits (S/D = 0.10)
Fig. 4.5 Locations of Dial and Strain Gauges
Fig. 4.6 Layout of Construction Layers; (a) Case of One Conduit; and (b) Case of Two Conduits
Fig. 4.7 Experimental Set-Up of the Triaxial Test
Fig. 4.9 Test No. 1 During Construction
(Dial Gauges Set-Up)
Fig. 4.11 Failure Shape of Test No. 1
Fig. 4.14 Thrust in the Walls of One Conduit

Scale 1 cm = 14.0 kN/m
Fig. 4.15 Bending Moment Distribution in Case of One Conduit

Scale 1 cm = 56.0 x 10^{-3} kN.m/m
Fig. 4.18 Experimental Set-Up Before Loading (Test No. 2)
Fig. 4.20 Deformation in Case of Two Conduits
(S/D = 0.5 Section 1-1)

Scale 1 cm = 6.20 mm
Fig. 4.21 Deformation in Case of Two Conduits (S/D = 0.5 Conduit No. 1)

Section I-I

Section II-II

Scale 1 cm = 6.20 mm

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Fig. 4.22 Thrust in the Walls of Two Adjacent Conduits

(S/D = 0.5  Section I-I)

Scale 1 cm = 12.8 kN/m
Fig. 4.23 Thrust in the Walls of Two Adjacent Conduits
(S/D = 0.5 Conduit No.1)

Scale 1 cm = 12.8 kN/m
Fig. 4.24 Bending Moment Distribution in Case of Two Conduits (S/D = 0.5 Section I-I)
Scale 1 cm = 52.0 x 10^3 kN·cm/m
Fig. 4.25 Bending Moment Distribution in Case of Two Conduits
(S/D = 0.5 Conduit No. 1)

Scale 1 cm = 52.0 x 10^3 kN.m/m
Fig. 4.26 Experimental Set-Up During Loading
(Test No. 3)
Fig. 4.28 Deformation in Case of Two Conduits
(S/D = 0.1 Section 1-1)

Spacing is not to Scale

Scale 1 cm = 6.20 mm
Fig. 4.29 Deformation in Case of Two Conduits
(S/D = 0.1 Conduit No. 1)

Scale 1 cm = 6.20 mm
Fig. 4.30 Thrust in the Walls of Two Adjacent Conduits
(S/D = 0.1 Section I-I)

Scale 1 cm = 12.8 kN/m
Spacing is not to Scale
Fig. 4.31 Thrust in the Walls of Two Adjacent Conduits (S/D = 0.1 Conduit No. 1)

Scale 1 cm = 12.8 kN/m
Fig. 4.32 Bending Moment Distribution in Case of Two Conduits
(S/D = 0.1 Section 1-1)

Scale: 1 cm = 52.0 x 10^-3 kN.m/m

Spacing is not to Scale
Fig. 4.33 Bending Moment Distribution in Case of Two Conduits
(S/D = 0.1 Conduit No. 1)

Scale 1 cm = $52.0 \times 10^3$ kN.m/m
Fig. 4.34 Load–Deflection Curves for the Laboratory Tests
Fig. 4.38 Relationship Between Load and Bending Moment (Test No. 1)
Fig. 4.39 Deformation in Case of One Conduit at Load 8.90 kN - (Section I-I)

Scale 1 cm = 4.2 mm
Fig. 4.40 Thrust Distribution in Case of One Conduit at Load 8.90 kN – (Section I–I)

Scale 1 cm = 10.4 kN/m
Fig. 4.41 B.M. Distribution in Case of One Conduit at Load 8.90 kN — (Section I–I)

Scale 1 cm = 42.0 x 10^{-3} kN.m/m
Fig. 4.43 Relationship Between the Load and Bending Moment Test No. 3 (S/D = 0.10)
Fig. 5.1 Three-Dimensional Finite Element Model
Fig. 5.2 Details of the OHBDC Design Truck
(a) the OHBDC Design Truck; (b) Contact Area of the Heaviest Wheel on Ground; and (c) Dimensions of Two—OHBDC Design Trucks Side by Side
Fig. 5.3 Two-Dimensional Plane-Strain Model

31.8 kN each

1.2 m

48.0 m

11.0 m

11.8 m

23.6 m

174
Fig. 5.4 Vertical Pressure Distribution in the Soil at Depth = 1.0 m
Vertical Pressure in the Soil
($10^{-2}$ MPa)

FIG. 5.5 Vertical Pressure Distribution in the Soil at the Crown Level

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Fig. 5.6 Vertical Pressure Distribution in the Soil in the Longitudinal Direction (Three-Dimensional Analysis)
Fig. 5.7 Vertical Pressure Distribution in the Soil in the Span Direction (Three-Dimensional Analysis)
Fig. 5.8 Vertical Pressure Distribution in the Soil in the Span Direction (Two-Dimensional Analysis)
Fig. 5.9 Vertical Pressure Distribution in the Soil at the Crown Level
Fig. 5.10 Dispersion Angles $\Theta$ and $\alpha$
Fig. 5.11 Thrust Distribution in Case of One Conduit

Scale 1 cm = 18.6 kN/m
Fig. 5.12 B.M. Distribution in Case of One Conduit

Scale $1 \text{ cm} = 120. \times 10^{-3} \text{ kN.m/m}$
Fig. 5.13 Buckling Shape of a Single Conduit (First Mode)
(Magnification Factor = \(2.6 \times 10^3\))
Fig. 5.14 Buckling Shape of A Single Conduit (Second Mode)
(Magnification Factor = $2.5 \times 10^3$)
Fig. 6.1 Two-Dimensional Plane-Strain Model (S/D=0.58)

31.8 kN each

1.2 m

18.0 m | 12.0 m | 18.0 m

60.0 m

11.8 m | 12.0 m | 23.8 m

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Fig. 6.2 Two-Dimensional Plane-Strain Model (S/D=0.42)
Fig. 6.3 Two-Dimensional Plane-Strain Model (S/D=0.26)
Fig. 6.4 Two-Dimensional Plane-Strain Model (S/D=0.10)

189

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Fig. 6.5 Deformed Model under Dead and Live Loads (S/D = 0.58)
(Magnification Factor = 1.3 x 10^3)
Fig. 6.6 Deformed Model under Dead and Live Loads ($S/D = 0.10$)
(Magnification Factor $= 1.2 \times 10^3$)
Fig. 6.7 Thrust Distribution for Different S/D Ratios (Two-Dimensional Analysis)

Scale 1 cm = 250 kN/m
Fig. 6.8  Bending Moment Distribution for Different S/D Ratios (Two-Dimensional Analysis)

Scale 1 cm = 250.0 x 10$^3$ kN.m/m
Fig. 6.9 Three-Dimensional Finite Element Model (S/D = 0.58)
Fig. 6.10 Three-Dimensional Finite Element Model (S/D = 0.42)
Fig. 6.12 Three-Dimensional Finite Element Model (S/D = 0.10)
Fig. 6.13  Thrust Distribution for Different S/D Ratios
(Three-Dimensional Analysis)

Scale 1 cm = 25.0 kN/m
Fig. 6.15 Relationship Between Maximum Thrust and S/D Ratio (Elastic Model)
Fig. 6.16 Relationship Between Maximum Positive B.M. and S/D Ratio

(Elastic Model)
Fig. 6.17 Relationship Between Maximum Negative B.M. and S/D Ratio (Elastic Model)
Fig. 6.18 Deformed Model at Failure Load
(Magnification Factor = 7.3)
Fig. 6.19 Load–Deflection Curves for the Case of One Conduit
(Two-Dimensional Analysis)
Fig. 6.21 Load–Deflection Curves at the Crown for Different S/D Ratio (Two-Dimensional Analysis)
Fig. 6.22 Relationship Between Load-Carrying Capacity and S/D Ratio
(Two-Dimensional Analysis & Elastic-Plastic Model)
Fig. 6.23 Load–Deflection Curves for the Case of One Conduit
(Three-Dimensional Analysis)
Fig. 6.24 Load-Deflection Curves at the Crown for Different S/D Ratio (Three-Dimensional Analysis)
Fig. 6.25 Relationship Between Load-Carrying Capacity and S/D Ratio
(Three-Dimensional Analysis & Elastic-Plastic Model)
Fig. 6.26 Relationship Between Maximum Thrust and S/D Ratio
(Elastic-Plastic Model)
Fig. 6.27 Relationship Between Maximum Positive B.M. and S/D Ratio (Elastic-Plastic Model)
Fig. 6.28 Relationship Between Maximum Negative B.M. and S/D Ratio (Elastic-Plastic Model)
Fig. 6.29 Buckling Shape of Multi-Span Conduits (S/D = 0.58, First Mode)
(Magnification Factor = 3.3 x 10^3)
Fig. 6.30 Buckling Shape of Multi-Span Conduits ($S/D = 0.58$, Second Mode) (Magnification Factor $= 3.1 \times 10^3$)
Fig. 6.31 Buckling Shape of Multi-Span Conduits (S/D = 0.42, First Mode)
(Magnification Factor = 3.2 x 10³)
Fig. 6.32 Buckling Shape of Multi-Span Conduits (S/D = 0.42, Second Mode)
(Magnification Factor = 3.1 \times 10^3)
Fig. 6.33 Buckling Shape of Multi-Span Conduits (S/D = 0.26, First Mode)
(Magnification Factor = 3.1 x 10^3)
Fig. 6.34 Buckling Shape of Multi-Span Conduits (S/D = 0.26, Second Mode)
(Magnification Factor = 3.1 x 10³)
Fig. 6.35 Buckling Shape of Multi-Span Conduits (S/D = 0.10, First Mode)
(Magnification Factor = 3.1 x 10^3)
Fig. 6.36  Buckling Shape of Multi-Span Conduits (S/D = 0.10 , Second Mode)  
(Magnification Factor = $3.0 \times 10^3$)
Fig. 6.37 Relationship Between Elastic Buckling Stress and S/D Ratio
Fig. 7.1 Three-Dimensional Finite Element Model
(Case of Non-Uniform Dead Load)
Fig. 7.4 Circumferential Thrust Distribution Due to Non-Uniform Dead Load

Scale 1 cm = 150.0 kN/m
Fig. 7.5 Circumferential B.M. Variation Along the Conduit Due to Live Load in Case of Non-Uniform Dead Load
Fig. 7.6 Circumferential B.M. Variation Along the Conduit Due to Dead Load in Case of Non-Uniform Dead Load
Fig. 7.7 Cylindrical B.M. Distribution Due to Non-Uniform Dead Load

Scale 1 cm = 500.0 x 10^3 kN/m/m
Fig. 7.8 Deflection Variation Along the Conduit Due to Live Load in Case of Non-Uniform Dead Load
Fig. 7.9 Deflection Variation Along the Conduit Due to Dead Load in Case of Non-Uniform Dead Load
Fig. 7.10  Vertical Soil Pressure Distribution in the Longitudinal Direction at the Invert Level Due to Dead Load
Appendix A

Properties of the Corrugation Profile 152 x 51 mm (AISI 1984)

1- Corrugation radius (CR) : 28.58 mm
2- Wall thickness (T) : 7.0 mm
3- Area (A) : 8.712 mm$^2$/mm
4- Tangent length (TL) : 43.237 mm
5- Tangent angle (Δ) : 46.083 degrees
6- Moment of inertia (I) : 2675.11 mm$^4$/mm
7- Section modulus (S) : 92.56 mm$^3$/mm
8- Radius of gyration (r) : 17.523 mm

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Appendix B

The Rigidity Matrix of the General Shell Section

The rigidity matrix of the general shell section has been calculated according to Abdel-Sayed (1970) as follows:

1- Axial rigidity in S-direction \( (D_s) = \frac{L}{c} t E \)

2- Axial rigidity in X-direction \( (D_x) = \frac{E}{6(1-v^2)} \left( \frac{L^2}{f} \right) t \)

3- Shear rigidity in XS-plane \( (D_{xy}) = \frac{R}{2(1+v)} \left( \frac{Et}{L} \right) c \)

4- Bending rigidity in SZ-plane \( (B_s) = 0.522 \frac{E f^2}{t} \)

5- Bending rigidity in XZ-plane \( (B_x) = \left( \frac{c}{L} \right) \frac{E t^3}{12(1-v^2)} \)
6- Torsional rigidity \( (B_{\text{zs}}) = \frac{L}{c} \frac{Et^3}{12(1+v)} \)

where:

\( B_x, B_z \) = bending rigidity in XZ- and SZ-planes, respectively;

\( B_{\text{zs}} \) = torsional rigidity;

\( c \) = corrugation pitch \( (c = 76.0 \text{ mm}) \);

\( D_x, D_z \) = axial rigidity in X- and S-directions, respectively;

\( D_{\text{xs}} \) = shear rigidity in XS-plane;

\( E \) = modulus of elasticity of material \( (E = 200 \text{ GPa}) \);

\( f \) = half depth of corrugation \( (f = 25.5 \text{ mm}) \);

\( L \) = developed length of corrugation per pitch \( (L = 94.84 \text{ mm}) \);

\( R \) = reduction factor of shear rigidity \( (R = 1.0) \);

\( t \) = average thickness of corrugated sheet \( (t = 7.0 \text{ mm}) \); and

\( v \) = poisson's ratio \( (v = 0.30) \)
Appendix C

Maximum Dead Load Thrust

The maximum dead load thrust has been calculated according to OHBDC (1983), Hafez and Abdel-Sayed (1983), OHBDC (1991) and finally from the present study using the finite element method as follows:

1- OHBDC (1983)

The maximum thrust due to dead load ($T_D$), was calculated from the following equation:

$$ T_D = \frac{\mu_1 \sigma_{vo} R_i}{1000} $$

where

$R_i$ = radius of conduit at crown ($R_i = 3800$ mm);

$\mu_1$ = a conduit shape factor ($\mu_1 = 1.25$); and

$\sigma_{vo}$ = free field overburden pressure ($\sigma_{vo} = 37.7$ KPa).

This method resulted in a maximum dead load thrust equal to 179.1 kN/m.

2- Hafez and Abdel-Sayed (1983)

The maximum dead load thrust obtained from a two-dimensional finite element analysis conducted by Hafez and Abdel-Sayed (1983) was found to be equal to 156.0
kN/m.

3- OHBDC (1991)

The research presented by Haggag (1989), showed that the OHBDC (1983), underestimates the maximum dead load thrust especially for structures with larger spans. Therefore, the OHBDC (1991) requires that the maximum dead load thrust needs to be calculated as follows:

\[ T_D = 0.5 \left( 1.0 - 0.1 C_s \right) A_f W \]  \hfill (C.2)

where

- \( W \) = the weight per unit length of the column of soil directly above the conduit \((W = 403.4\) kN/m);
- \( A_f \) = a coefficient depends upon the shape of the conduit and the ratio of the depth of cover to the vertical dimension \((A_f = 1.25)\); and
- \( C_s \) = the axial stiffness parameter \((C_s = 0.153)\).

The maximum dead load thrust according to Equation C.2 was found to be equal to 248.4 kN.m.

4- The Present Finite Element Study

The finite element two-dimensional analysis has been used to determine the maximum dead load thrust in the conduit wall, as explained in chapter III (section 3.4.4.1). It was found that the maximum dead load thrust was equal to 227.0 kN/m. All the above values are presented in Table 3.3.
Appendix D

Comparison of Elastic Buckling Stresses with

OHBDC (1991) and Continuum Theory

The calculation of the elastic buckling stresses according to the OHBDC (1991) and the continuum theory (Moore et al., 1988) are presented as follows:

1- OHBDC (1991)

The elastic buckling stress, $f_e$, is calculated using the following equation:

$$f_e = \frac{3 \phi \rho F_m E}{(\frac{KR}{r})^2}$$

where

- $\phi$ = a resistance factor ($\phi = 1.0$),
- $F_m$ = a reduction factor for modifying buckling stress in multi-conduit structures ($F_m = 1.0$),
- $E$ = modulus of elasticity of steel ($E = 200000$. MPa),
- $R$ = radius of curvature of the conduit wall, at the mid-height of corrugation, at a transverse section ($R = 3800$. mm),
- $r$ = radius of gyration of corrugation profile ($r = 17.523$ mm),
- $\rho$ = a reduction factor for buckling stress due to shallow cover, and calculated
using the following equation:

\[ \rho = \left[ \frac{1000 \ H}{R_c} \right]^{0.5} \leq 1.0 \]  \hspace{2cm} (D.2)

in which

- \( H \) = depth of cover (\( H = 2.0 \) m),
- \( R_c = R \) at crown (\( R_c = 3800. \) mm), and
- \( K \) = a factor representing the relative stiffness of the conduit wall with respect to the adjacent soil, and calculated using the following equation:

\[ K = \lambda \left[ \frac{E \ I_s}{E_m \ R_c^3} \right]^{0.25} \]  \hspace{2cm} (D.3)

where

- \( I_s \) = second moment of cross-sectional area about its longitudinal axis (\( I_s = 2675.11 \) mm\(^4\)/mm).
- \( \lambda \) = a factor used in calculating \( K \) from the following equation:

\[ \lambda = 1.22 \left[ 1.0 + 1.6 \left( \frac{E \ I_s}{E_m \ R_c^3} \right)^{0.25} \right] \]  \hspace{2cm} (D.4)

in which

- \( E_m \) = a modified modulus of soil stiffness (MPa), calculated using the following equation:

\[ E_m = E^* \left[ 1 - \left( \frac{R_c}{R_c + 1000 \ H} \right)^2 \right] \]  \hspace{2cm} (D.5)

where
\( E' = \) modulus of soil stiffness \((E' = 20, \text{ MPa})\).

Since the Young’s modulus of soil of 35. MPa has been used in the finite element analysis, \( E_m \) has been assumed to be equal to \( E' \) in the calculation of \( \lambda \) and \( K \). Substituting the above values in Equations D.4, D.3, D.2, and D.1 give the following results for the elastic buckling stresses:

- for \( \rho = 0.725 \) \( f_e = 184. \text{ MPa}, \) and
- for \( \rho = 1.0 \) \( f_e = 253. \text{ MPa}. \)

The OHBDC buckling stress \( f_b \), shown in Table 8.6 are calculated according to the following equation:

\[
f_b = F_y - \left[ \frac{F_y^2}{12E} \left( \frac{KR}{r} \right)^2 \frac{1}{\rho} \right]
\]  

(D.6)

where \( F_y \) is the yield strength of the conduit wall \((F_y = 228. \text{ MPa}).\)

**2- Continuum Theory**

The elastic buckling stress, \( f_e \), is calculated using the continuum theory (Moore et al., 1988) as follow:

\[
f_e = \phi f_c
\]  

(D.7)

where

\( \phi = \) performance factor, and

\[
f_c = 1.3 \psi \frac{(E E_s)^{1/3} (E_s')^{2/3}}{A} R_x R_{h}
\]  

(D.8)

which is valid only when
$\frac{E I_s}{E_s R^3} < 10^{-2}$  \hspace{1cm} \text{(D.9)}$

where

$\psi$ = a reduction factor equal to 0.55,

$R_s$ = a correction factor for culvert shape ($R_s = 1.0$).

$R_h$ = a correction factor for shallow cover ($R_h = 0.746$).

$E_s$ = Young’s modulus of soil ($E_s = 35. \text{ MPa}$),

$E_s^*$ = soil stiffness calculated using the following equation:

$$E_s^* = \frac{E_s}{1 - v^2} \hspace{1cm} \text{(D.10)}$$

The calculation of the elastic buckling stress using the continuum theory gives a value of 581. MPa.
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246
VITA AUCTORIS

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