NOTICE

The quality of this microform is heavily dependent upon the quality of the original thesis submitted for microfilming. Every effort has been made to ensure the highest quality of reproduction possible.

If pages are missing, contact the university which granted the degree.

Some pages may have indistinct print especially if the original pages were typed with a poor typewriter ribbon or if the university sent us an inferior photocopy.

Reproduction in full or in part of this microform is governed by the Canadian Copyright Act, R.S.C. 1970, c. C-30, and subsequent amendments.

AVIS

La qualité de cette microforme dépend grandement de la qualité de la thèse soumise au microfilming. Nous avons tout fait pour assurer une qualité supérieure de reproduction.

S'il manque des pages, veuillez communiquer avec l'université qui a conféré le grade.

La qualité d'impression de certaines pages peut laisser a désirer, surtout si les pages originales ont été dactylographiées à l'aide d'un ruban usé ou si l'université nous a fait parvenir une photocopie de qualité inférieure.

La reproduction, même partielle, de cette microforme est soumise à la Loi canadienne sur le droit d'auteur, SRC 1970, c. C-30, et ses amendements subséquents.
Optimization of Prestressed Concrete Bridge Ties for Open Deck Railway Bridges

Nigel William Peters

A Thesis in The Department of Civil Engineering

Presented in Partial Fulfilment of the Requirements for the Degree of Master of Applied Science at Concordia University Montreal, Quebec, Canada

March 1992

© 1992
The author has granted an irrevocable non-exclusive licence allowing the National Library of Canada to reproduce, loan, distribute or sell copies of his/her thesis by any means and in any form or format, making this thesis available to interested persons.

The author retains ownership of the copyright in his/her thesis. Neither the thesis nor substantial extracts from it may be printed or otherwise reproduced without his/her permission.

L'auteur a accordé une licence irrévocable et non exclusive permettant à la Bibliothèque nationale du Canada de reproduire, prêter, distribuer ou vendre des copies de sa thèse de quelque manière et sous quelque forme que ce soit pour mettre des exemplaires de cette thèse à la disposition des personnes intéressées.

L'auteur conserve la propriété du droit d'auteur qui protège sa thèse. Ni la thèse ni des extraits substantiels de celle-ci ne doivent être imprimés ou autrement reproduits sans son autorisation.

ABSTRACT

Optimization of Prestressed Concrete Bridge Ties for Open Deck Railway Bridges

Nigel W. Peters

This thesis presents a study in the optimization of prestressed concrete bridge tie design. A historical review of both the development of the bridge tie and the design of the existing tie is presented. The problem of the existing tie design cracking from rebound and the need for redesign is also outlined.

The data obtained from field investigations of a short span bridge was analyzed in order to determine the effect of various types of rail-tie and tie-girder pads on attenuating impacts from railway vehicles. Also, maximum bending strains from these impacts were determined.

A mathematical model is presented which predicts both the maximum static and dynamic strains experienced by the ties under impact. This model was then compared to the field results which led to the design of three types of prestressed concrete bridge ties.

Eighteen full scale samples (six of each type) were fabricated
for laboratory testing consisting of static load tests to
destruction. Measurements of strains and deflections were
obtained.

The purpose of the laboratory testing was to determine the
effect of increasing prestress levels in the compressive zone
of the tie on the cracking moment and ultimate moment capacity
of the tie.

As this is a joint Concordia University - Canadian National
Railway research undertaking, the conclusions and
recommendations presented will be of great interest to the
railway in assisting in the possible redesign of the existing
C.N. Rail prestressed concrete bridge tie.
ACKNOWLEDGEMENTS

The author wishes to express sincere gratitude and appreciation to the following individuals and organizations whose contributions helped in the successful completion of this research project:

Dr. Z. A. Zielinski, Professor of Civil Engineering, Concordia University for directing this research program, for his friendship, patient tutelage and constant encouragement throughout this study.

Mr. J. F. Scott, Senior Research Engineer, Canadian National Railway for graciously supplying field data and for his guidance and assistance.

Mr. A. Taras, formerly Project Engineer, Canadian National Railway, now with Hydro Quebec for his invaluable assistance in reviewing the mathematical model.

Mr. M. Rigotti and Mr. G. S. Pandher for their assistance during the experimental stages of this research program.

The Department of Civil Engineering, Concordia University for the use of their laboratory facilities.
Monarch Préco Limited of Laval, Quebec for fabricating and
donating the eighteen test ties specifically for this research
program.

Ms. E. Karosan of Monarch Préco Limited for her assistance in
supervising the fabrication of the ties and for supplying the
relevant control documents.

Mr. G. S. Weatherly and Mrs. G. J. Weatherly for typing the
final manuscript of this report.

Mr. C. O'Brian and Mr. D. Chow for their assistance with the
graphs and figures.
TABLE OF CONTENTS

LIST OF FIGURES xi
LIST OF TABLES xvi
LIST OF NOTATIONS xviii

CHAPTER 1 - BACKGROUND HISTORY 1
  1.1 The Open Deck Bridge 1
  1.2 Concrete Versus Timber Ties 1
  1.3 The Concrete Bridge Tie 4
  1.4 The Current Problem 8

CHAPTER 2 - THE EXISTING C.N. BRIDGE TIE 9
  2.1 General 9
  2.2 Design Parameters 9
  2.3 Material Properties 11
  2.4 Geometric Properties 11
  2.5 Design Loads 13
  2.6 Concrete Stresses 13
  2.7 Tie Cracking Strength 14
  2.8 Ultimate Moment Capacity 15
  2.9 Fatigue Cracking Moment 16

CHAPTER 3 - THE C.N. FIELD TESTS 18
  3.1 Evaluation 18
  3.2 The 1984 Bridge Configuration 18
  3.3 Rail-Tie and Tie-Girder Pads (1984) 18
  3.4 Strain Gauge Configuration (1984) 19
  3.5 The 1986 Bridge Configuration 22
  3.6 Rail-Tie and Tie-Girder Pads (1986) 24
# TABLE OF CONTENTS

CHAPTER 3 - THE C.N. FIELD TESTS (CONTINUED)

3.7 Strain Gauge Configuration (1986) 24
3.8 Calibration Measurements 25
3.9 Bending Strains of Concrete Bridge Ties 25
3.10 Strains of Prestressing Wires 28
3.11 Instrumentation 28
3.12 The 1984 and 1986 Test Procedure 36

CHAPTER 4 - ANALYSIS OF THE C.N. FIELD TEST DATA 39

4.1 General 39
4.2 1984 Results for Positive Bending 39
4.3 1984 Results for Negative Bending 45
4.4 1986 Results for Positive Bending 49
4.5 Comparison of the 1984 and 1986 Results for Ties 1 and 22 52
4.6 Evaluation of Impact Factors 57
4.7 Increased Strength of Concrete Under Impact 62
4.8 A.R.E.A. Specified Impact Factors 64
4.9 Frequency Response of the Bridge Ties 64

CHAPTER 5 - PROPOSED DESIGN MODEL 68

5.1 System Dynamics 68
5.2 Mathematical Model of the Concrete Bridge Tie Response in Positive Bending 70
5.3 Mathematical Model of the Concrete Bridge Tie Response in Negative Bending 89
5.4 Discussion on the Models 95
# TABLE OF CONTENTS

## CHAPTER 6 - DESIGN OF THE CONCORDIA TEST TIES

6.1 Conversion of Model Strains to Stresses ................................................. 96
6.2 Design of the Type 1 Tie ............................................................................. 98
6.3 Design of the Type 2 and 3 Ties ................................................................. 99
6.4 Ultimate Moment Capacity Check of the Type 1 Tie ................................. 100
6.5 Ultimate Moment Capacity Check of the Type 2 and 3 Ties ....................... 110
6.6 Cracking Moment of the Type 1 Tie .......................................................... 111
6.7 Cracking Moment of the Type 2 and 3 Ties .............................................. 113
6.8 Calculation of the Splitting Force .............................................................. 114
6.9 Calculation of an Equivalent Impact Factor ............................................. 118
6.10 Fatigue Cracking Moment ....................................................................... 119
6.11 Analysis of Disturbed Regions (End Block Analysis) .............................. 121

## CHAPTER 7 - TEST PROCEDURE FOR THE CONCORDIA TIES

7.1 Tie Specimens ............................................................................................ 122
7.2 Loading System .......................................................................................... 122
7.3 Instrumentation .......................................................................................... 123
7.4 Testing Procedure ...................................................................................... 125
7.5 Concrete Strength ...................................................................................... 127
7.6 Prestressing Steel ....................................................................................... 128

## CHAPTER 8 - COMPARISON OF THEORETICAL AND EXPERIMENTAL RESULTS FOR INDIVIDUAL TIES

8.1 General Test Observations ........................................................................ 133
8.2 Load-Deflection ......................................................................................... 137
TABLE OF CONTENTS

CHAPTER 8 - COMPARISON OF THEORETICAL AND EXPERIMENTAL RESULTS FOR INDIVIDUAL TIES (CONTINUED)

8.3 Comparison of Cracking Moments 145
8.4 Ultimate Moment Capacity 150
8.5 Strain Gauge Readings 156

CHAPTER 9 - CONCLUSIONS AND RECOMMENDATIONS 157

9.1 Conclusions 157
9.2 Recommendations 160

REFERENCES 162

APPENDIX A - TWO WIRE TRANSMITTER BOX OUTPUT VOLTAGE FORMULATION 165

APPENDIX B - OCCURRENCE VERSUS STRAIN AND CUMULATIVE PERCENT EXCEEDING GRAPHS FOR 1984 AND 1986 C.N. FIELD TESTS 176

APPENDIX C - ANCILLARY CALCULATIONS USED IN THE MODELS 213

APPENDIX D - DOCUMENTATION ON ELONGATION OF WIRE STRANDS 219

APPENDIX E - PHOTOGRAPHS OF FAILURE MODES OF LABORATORY TESTED TIES 241
LIST OF FIGURES

1.1 Comparison of Open Deck and Ballast Deck Railway Bridges 2
1.2 Diagram of Short Span Open Deck Railway Bridge 5
1.3 Test Bridge Before and After the Installation of Prestressed Concrete Bridge Ties 7
2.1 Cross Section of C.N. Rail Prestressed Concrete Bridge Tie 10
2.2 Diagram of Load Application 16
3.1 Regions of Influence for Rail-Tie and Tie-Girder Pads 20
3.2 Location of Strain Gauges on Field Test Ties 23
3.3 Calibration Curve for Concrete Ties 27
3.4 Calibration Curve for Wire 1 29
3.5 Calibration Curve for Wire 2 30
3.6 Calibration Curve for Wire 3 31
3.7 Calibration Curve for Wire 4 32
3.8 Relaxation of Wires 1 and 2 Over Time 33
3.9 Relaxation of Wires 3 and 4 Over Time 34
4.1 Summary Graph 3RB for 1984 40
4.2 Summary Graph 8RB for 1984 41
4.3 Summary Graph 15RB for 1984 42
4.4 Summary Graph 20RB for 1984 43
4.5 Summary Graph 8RT for 1984 46
4.6 Summary Graph 15RT for 1984 47
4.7 Summary Graph 20RT for 1984 48
4.8 Summary Graph 3RB for 1986 50
4.9 Summary Graph 15RB for 1984 51
4.10 Summary Graph 1RB for 1984 53

xi
LIST OF FIGURES

4.11 Summary Graph 1RB for 1986 54
4.12 Summary Graph 22RB for 1984 55
4.13 Summary Graph 22RB for 1986 56
4.14 Average Strain Versus Speed for Light Wheel Loads 58
4.15 Average Strain Versus Speed for Heavy Wheel Loads 59
4.16 Average Impact Factors Versus Speed for Light Wheel Loads 60
4.17 Average Impact Factors Versus Speed for Heavy Wheel Loads 61
4.18 Theoretical Resonant Modes and Frequency Response for the C.N. Rail Prestressed Concrete Bridge Tie 66
5.1 Schematic of Wheel Impacts 71
5.2 Dynamic Forces During Impact 72
5.3 Kinematic Mechanism of Wheel - Rail Impact 74
5.4 Distribution of Impact Force 75
5.5 Assumed Deflected Shape of Rail After Impact 77
5.6 Assumed Deflected Shape of Tie in First Mode 79
5.7 Assumed Displacement of Rail and Tie 82
5.8 Bending Moment Diagram for Tie 90
6.1 Strand Arrangement of the Type 1 Tie 101
6.2 Strand Arrangement of the Type 2 Tie 102
6.3 Strand Arrangement of the Type 3 Tie 103
6.4 Comparison of Various Tie Cross Sections 105
6.5 Comparison of Effective Prestress Levels in Various Ties 106
6.6 Stress - Strain Compatibility Diagram 107
6.7 Diagram of Splitting Forces 115

xii
### LIST OF FIGURES

7.1 Loading System Used in Laboratory Testing of Ties 124
7.2 Stress - Strain Curve for 3/8" (9.55 mm) 7 Wire Strand 130
8.1 Diagram of Typical Failure Mode of a Laboratory Tested Tie 134
8.2 Failure Mode of Tie T2B4 Inverted Position 135
8.3 Failure Mode of Tie T3B2 Right Side Up Position 136
8.4 Load-Deflection Curve for Tie T1B2 138
8.5 Load-Deflection Curve for Tie T1B3 139
8.6 Load-Deflection Curve for Tie T2B1 140
8.7 Load-Deflection Curve for Tie T2B2 141
8.8 Load-Deflection Curve for Tie T2B3 142
8.9 Load-Deflection Curve for Tie T3B1 143
8.10 Load-Deflection Curve for Tie T3B2 144
8.11 Relationship Between $M_{cr}/bh^2f_c'$ and $f_v/f_c'$ 149
8.12 Relationship Between $M_{cr}/bh^2f_c'$ and $f_v/f_c'$ 154
8.13 Relationship Between $M_{cr}/bh^2f_c'$ and $f_v/f_c'$ 155
A 1.0 Schematic of a Wheatstone Bridge 167
A 1.1 Schematic of Strain Gauge Rosettes 167
A 1.2 Schematic of a Wheatstone Bridge with Padding Resistors 171
A 1.3 Wheatstone Bridge with Calibration Switch 173
B 1.0 Summary Graph 3RB for 1984 177
B 1.1 Cumulative Exceeding Graph for 3RB, 1984 178
B 1.2 Summary Graph 8RB for 1984 179
B 1.3 Cumulative Exceeding Graph for 8RB, 1984 180
LIST OF FIGURES

B 1.4 Summary Graph 15RB for 1984 181
B 1.5 Cumulative Exceeding Graph for 15RB, 1984 182
B 1.6 Summary Graph 20RB for 1984 183
B 1.7 Cumulative Exceeding Graph for 20RB, 1984 184
B 1.8 Summary Graph 3, 8, 15, 20RB for 1984 185
B 1.9 Cumulative Exceeding Graph for 3, 8, 15, 20RB, 1984 186
B 2.0 Summary Graph 8RT for 1984 187
B 2.1 Cumulative Exceeding Graph for 8RT, 1984 188
B 2.2 Summary Graph 15RT for 1984 189
B 2.3 Cumulative Exceeding Graph for 15RT, 1984 190
B 2.4 Summary Graph 20RT for 1984 191
B 2.5 Cumulative Exceeding Graph for 20RT, 1984 192
B 2.6 Summary Graph 8, 15, 20RT for 1984 193
B 2.7 Cumulative Exceeding Graph for 8, 15, 20RT, 1984 194
B 3.0 Summary Graph 3RB for 1986 195
B 3.1 Cumulative Exceeding Graph for 3RB, 1986 196
B 3.2 Summary Graph 15RB for 1986 197
B 3.3 Cumulative Exceeding Graph for 15RB, 1986 198
B 3.4 Summary Graph 3, 15RB for 1986 199
B 3.5 Cumulative Exceeding Graph for 3, 15RB for 1986 200
B 4.0 Summary Graph 1RB for 1984 201
B 4.1 Cumulative Exceeding Graph for 1RB, 1984 202
B 4.2 Summary Graph 22RB for 1984 203
B 4.3 Cumulative Exceeding Graph for 22RB, 1984 204
LIST OF FIGURES

B 4.4 Summary Graph 1RB for 1986 205
B 4.5 Cumulative Exceeding Graph for 1RB, 1986 206
B 4.6 Summary Graph 22RB for 1986 207
B 4.7 Cumulative Exceeding Graph for 22RB, 1986 208
B 5.0 Average Strain vs Speed (Light Loads) 209
B 5.1 Average Strain vs Speed (Heavy Loads) 210
B 5.2 Average Impact Factors (Light Loads) 211
B 5.3 Average Impact Factors (Heavy Loads) 212
C 1.0 Period of Tie to Close Gap 214
C 1.1 Attenuation Properties of Railway Vehicle Springs 215
C 1.2 Duration of Motion of Wheel Vertically 217
C 1.3 Duration of Motion of Wheel Horizontally 218
D 1.0 Production Drawing of Type 1 Tie 220
D 1.1 Production Drawing of Type 2 Tie 227
D 1.2 Production Drawing of Type 3 Tie 234
E 1.0 Photograph of Failure Mode of Tie T1B4 242
E 1.1 Photograph of Failure Mode of Tie T2B1 243
E 1.2 Photograph of Failure Mode of Tie T2B2 244
E 1.3 Photograph of Failure Mode of Tie T2B3 245
LIST OF TABLES

3.1 Concrete Bridge Tie Strain Gauge Circuits for the 1984 Tests - Location and Designation 21
3.2 Concrete Bridge Tie Strain Gauge Circuits for the 1986 Tests - Location and Designation 26
3.3 Concrete Bridge Tie Tests - Strain Gauge Setup for 1984 and 1986 38
5.1 Summary of Ratios $\hat{R}_w/\hat{R}_3$ and $F(16,n)/F(48)$ 84
6.1 Characteristics of Tie Designs 104
6.2 Cracking and Ultimate Capacity of Ties 114
7.1 Summary of Concrete Compressive Strength $-f_c'$ 128
7.2 Summary of Effective Prestressing Forces - $P_{\text{eff}}$ 131
8.1 Elastic and Total Energy Absorption of Ties 145
8.2 Comparison of Negative Cracking Moment 146
8.3 Comparison of Positive Cracking Moments 147
8.4 Values for $M_{cr}/bh^2f'_c$ and $f_t/f'_c$ 148
8.5 Comparison of Theoretical and Experimental Ultimate Moment Capacities 151
8.6 Values for $M_u/bh^2f'_c$ and $f_t/f'_c$ 153
D 1.0 Elongation Measurements for Tie T1B1 221
D 1.1 Elongation Measurements for Tie T1B2 222
D 1.2 Elongation Measurements for Tie T1B3 223
D 1.3 Elongation Measurements for Tie T1B4 224
D 1.4 Elongation Measurements for Tie T1B5 225
D 1.5 Elongation Measurements for Tie T1B6 226
D 2.0 Elongation Measurements for Tie T2B1 228
D 2.1 Elongation Measurements for Tie T2B2 229
D 2.2 Elongation Measurements for Tie T2B3 230
## LIST OF TABLES

<table>
<thead>
<tr>
<th>Table</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>D 2.3</td>
<td>Elongation Measurements for Tie T2B4</td>
<td>231</td>
</tr>
<tr>
<td>D 2.4</td>
<td>Elongation Measurements for Tie T2B5</td>
<td>232</td>
</tr>
<tr>
<td>D 2.5</td>
<td>Elongation Measurements for Tie T2B6</td>
<td>233</td>
</tr>
<tr>
<td>D 3.0</td>
<td>Elongation Measurements for Tie T3B1</td>
<td>235</td>
</tr>
<tr>
<td>D 3.1</td>
<td>Elongation Measurements for Tie T3B2</td>
<td>236</td>
</tr>
<tr>
<td>D 3.2</td>
<td>Elongation Measurements for Tie T3B3</td>
<td>237</td>
</tr>
<tr>
<td>D 3.3</td>
<td>Elongation Measurements for Tie T3B4</td>
<td>238</td>
</tr>
<tr>
<td>D 3.4</td>
<td>Elongation Measurements for Tie T3B5</td>
<td>239</td>
</tr>
<tr>
<td>D 3.5</td>
<td>Elongation Measurements For Tie T3B6</td>
<td>240</td>
</tr>
</tbody>
</table>
LIST OF NOTATIONS

a - depth of equivalent rectangular stress block
a - shear span length
b - overall bottom width of ties
b₁ - overall top width of tie
b - shear span length
b - axle to axle distance of railway vehicle bogie
d - depth of tie
d - length of skid flat on steel wheel
d₁ - distance from extreme compression fibre to first level of tension zone reinforcement
d₂ - distance from extreme compression fibre to second level of tension zone reinforcement
d₃ - distance from extreme compression fibre to third level of tension zone reinforcement
d₄ - distance from extreme compression fibre to fourth level of tension zone reinforcement
e - eccentricity of design prestressing force parallel to the axis measured from the centroid of the tie
fₜ - stress in the bottom fibre of the tie
f₉ - compressive stress in concrete
f₉' - specified compressive strength of concrete
f₉ₙ - compressive strength of concrete at the time of prestress transfer
f₉ₜ - compressive strength of concrete under combined bi-directional compression and tension
f₉ₜₖ - compressive strength of concrete under combined bi-directional compression and tension due to prestress
fₚₚ - ultimate tensile strength of tendon
fₚ - modulus of rupture of concrete
LIST OF NOTATIONS

$f_{sc}$ - effective prestress in steel after losses
$f_t$ - stress in top fibre of tie
$f_{tc}$ - tensile strength of concrete under bi-directional tension and compression
$f_{tcs}$ - tensile strength of concrete under bi-directional tension and compression due to prestress
$f_{to}$ - pure tensile strength of concrete
$f_{tself}$ - stress in top fibre of the tie due to prestress and self weight

$g$ - acceleration due to force of gravity
$h$ - overall height of tie
$n$ - modular ratio
$n$ - tie number
$r$ - radius of steel railway vehicle wheel
$r_y$ - vertical displacement between axle and rail
$v_y$ - velocity of rail with respect to the wheel
$t$ - time

$A$ - cross-sectional area of concrete tie
$A_c$ - concrete cross-sectional area
$A_i$ - initial cross-sectional area of the tie
$A_n$ - mode coefficient of the tie

$A_{ps}$, $A_s$ - area of prestressed reinforcement
$A_{s1}$ - area of prestressed reinforcement at depth $d_1$
$A_{s2}$ - area of prestressed reinforcement at depth $d_2$
$A_{s3}$ - area of prestressed reinforcement at depth $d_3$
$A_{s4}$ - area of prestressed reinforcement at depth $d_4$

$A_t$ - transformed cross-sectional area of the tie

xix
LIST OF NOTATIONS

C - compressive force due to stress block

C\textsubscript{1} - force due to prestressed reinforcement in the compressive zone

E\textsubscript{c} - modulus of elasticity of concrete

E\textsubscript{s} - modulus of elasticity of prestressed reinforcement

F - frictional force at the wheel rail interface

\dot{F} - frictional force at the wheel rail interface due to impulse loading

I - impact factor

I - moment of inertia

I\textsubscript{0} - moment of inertia calculated on the apparent linear mass

I\textsubscript{t} - transformed moment of inertia

K - spring constant

L - overall length of tie

L - sprung load of the railway vehicle over one wheel

\dot{L} - sprung load, due to the impulse loading, of the railway vehicle, over one wheel

\ell - length of tie between supports

M\textsubscript{cr} - flexural cracking moment

M\textsubscript{d} - moment due to dead load

M\textsubscript{f} - fatigue moment

M\textsubscript{L+I} - moment due to live loading and impact

M\textsubscript{LL} - moment due to live loading

M\textsubscript{SD} - moment due to super-imposed dead loading

M\textsubscript{self} - moment due to self weight

M\textsubscript{T} - mass of tie per unit length
LIST OF NOTATIONS

$M_{tot}$ - moment due to all applied loads

$M_{r}$ - mass of rail per unit length

$M_{u}$ - ultimate moment capacity

$M_{w}$ - mass of wheel

$P$ - prestressing force

$P_{cr}$ - applied load at moment of first cracking

$P_{eff}$ - effective prestressing force after losses

$P_{f}$ - applied load which causes fatigue

$P_{i}$ - initial prestressing force applied to tendons

$P_{u}$ - applied load at ultimate failure

$R$ - reaction of rail beneath the wheel

$\dot{R}$ - reaction due to dynamic impulse loading of the rail beneath the wheel

$S$ - prestressing force

$S_{b}$ - section modulus - bottom of tie

$S_{t}$ - section modulus - top of tie

$T$ - tractive force on coupler

$T$ - period (inverse of frequency)

$\dot{T}$ - tractive force on coupler due to impulse loading

$T_{1}$ - tensile force due to prestressed reinforcement at depth $d_{1}$

$T_{2}$ - tensile force due to prestressed reinforcement at depth $d_{2}$

$T_{3}$ - tensile force due to prestressed reinforcement at depth $d_{3}$

$T_{4}$ - tensile force due to prestressed reinforcement at depth $d_{4}$

$U$ - stored energy
LIST OF NOTATIONS

$U_d$ - energy stored in the tie due to the downward deflection

$U_u$ - energy stored in the tie due to the upward deflection

$V$ - velocity

$V_{split}$ - shear force to cause splitting

$W$ - wheel loading

$X$ - depth of stress block

$X$ - length along tie

$X_{max}$ - maximum depth of stress block

$Y$ - vertical displacement

$Y_b$ - distance from the top fibre of the tie to the centroid

$Y_{max}$ - maximum vertical displacement

$Y_o$ - downward deflection of centre of the tie

$Y_{o'}$ - upward deflection of the centre of the tie

$Y_R$ - displacement of the rail

$Y_S$ - compression of the railway vehicle springs

$Y_t$ - distance from the bottom fibre of the tie to the centroid

$Y_T$ - displacement of the tie

$Y_{TR}$ - displacement of the tie at the rail seat

$Y_W$ - displacement of the wheel

$\dot{Y}_R$ - vertical velocity of the rail

$\dot{Y}_T$ - vertical velocity of the tie

$\dot{Y}_{TN}$ - vertical velocity of the $n^{th}$ tie

$\dot{Y}_{TR}$ - velocity of the tie at the rail seat

$\dot{Y}_W$ - vertical velocity of the wheel

$Z$ - gap between rail and wheel at skid flat
LIST OF NOTATIONS

$\theta_0$ - one half the angle of the skid flat as measured from the centre of the wheel

$\alpha$ - angular deceleration of the axle due to impact

$\epsilon_b$ - strain at bottom fibre of the tie

$\epsilon_{cu}$ - ultimate strain in concrete

$\epsilon_t$ - strain at top fibre of the tie

$\epsilon_{sc}$ - strain in prestressing steel after losses

$\epsilon_{su}$ - strain in prestressing steel when the tendon reaches its ultimate tensile strength

$\epsilon_1$ - strain in prestressing steel at depth $d_1$

$\epsilon_2$ - strain in prestressing steel at depth $d_2$

$\epsilon_3$ - strain in prestressing steel at depth $d_3$

$\epsilon_4$ - strain in prestressing steel at depth $d_4$

$\epsilon_{max}$ - maximum strain

$\epsilon_x$ - tensile strain

$\lambda$ - length along rail of ties under consideration

$\mu$ - poisson's ratio

$\rho_c$ - mass density of concrete

$\sigma$ - stress

$\sigma_b$ - stress at bottom outer fibre of the tie

$\sigma_t$ - stress at top outer fibre of the tie

$\omega$ - angular velocity

$F$ - frequency

$\Sigma$ - summation

$\phi$ - capacity reduction factor
CHAPTER 1
BACKGROUND HISTORY

1.1 THE OPEN DECK BRIDGE

The floor systems of railway bridges are divided into two classes. These are open deck, where the floor consists of timber bridge ties, and solid or ballast decks comprised of steel or reinforced concrete decks supporting a ballasted track structure. In open deck bridges the bridge ties in a deck girder or beam bridge rest directly on the top flanges of the girders or beams. In through span bridges the ties are supported by longitudinal stringers. A comparison of an open deck and ballast deck bridge is shown in Figure 1.1.

Open deck structures were popular because they were less costly to construct and they induced less dead loading to the substructure. In addition because of their reduced depth they were practically suited to locations where overhead clearances were restricted, such as underpasses. A diagram of a short span open deck bridge is shown in Figure 1.2.

1.2 CONCRETE VERSUS TIMBER TIES

Up until the last twenty years or so timber remained the dominant material for both track and bridge ties in North America. This was because of the relative abundance of timber in North America and the generally satisfactory performance of both hardwood and softwood ties. In addition the state of
Figure 1.1 Comparison of Open Deck and Ballast Deck Railway Bridge
prestressed concrete technology in regards to the performance of the ties was not fully understood.

There is now a greater tendency in North America towards the acceptance and use of prestressed concrete ties as an acceptable alternative to the use of timber ties. This is primarily due to the fact that in the last twenty to thirty years rail traffic frequency, axle loadings, train lengths and speeds have all risen dramatically. Under these large increases the service life and performance of timber ties, both hardwood and softwood, has decreased especially on curves and open deck bridges.

The second reason is the growing scarcity of timber of the size required for railway ties, especially in the hardwood species. With the reduction in the availability of the so-called old growth timber, new growth timber is being used. However, the performance of this new growth timber is not nearly as satisfactory as that of the old growth timber. For these reasons concrete ties are a viable alternative.

There are several disadvantages in using timber bridge ties on open deck bridges. The close spacing, usually 14" (356.0 mm), on centre of the ties on such decks and their exposure on all sides makes them vulnerable to fire. Such fires can spread rapidly over the length of a bridge causing considerable
damage to the deck and supporting structure, often in the millions of dollars. In addition these ties are susceptible to rot, checking, splitting, warping, crushing and mechanical abrasion.

Because of the above problems, the railways often found it difficult to maintain proper alignment, gauge or level across open deck bridges in heavy usage.

1.3 THE CONCRETE BRIDGE TIE

In the late 1970's the Canadian Prestressed Concrete Institute established a liaison committee to promote the use of prestressed concrete on Canadian railways. In consultation with Canada's two major railways it was concluded that, due to the problems associated with timber bridge ties, a prestressed concrete bridge tie for use on open deck bridges would be an ideal place to start.

The Engineering Departments of both major railways proceeded to prepare design drawings for prestressed bridge ties, while the liaison committee compiled the specifications for the rigorous laboratory testing.

The laboratory testing was subsequently conducted at Queen's (12) and McGill Universities (8, 9), who obtained grants from the National Sciences and Engineering Research Council (14).
In addition the Transportation Development Centre of Transport Canada was retained to provide a technical review. Within a period of three years a great deal of research was conducted.

Based upon this research, C.N. Rail decided to design its' ties based on a 60 percent impact factor. In September of 1984, C.N. Rail installed 22 of these ties on a short span bridge located at mile 122.4 (km 198.3) of its Drummondville Subdivision, some 50 miles (80.5 km) from Montreal. These ties were installed on 16" (406 mm) centres and are shown in Figure 1.3.

The test ties are 12' (3.66 m) long, 12" (305 mm) deep, 10" (254 mm) wide at the top and 12" (305 mm) wide at the bottom. They are prestressed with 32 - 0.196" (5 mm) diameter indented steel wires which provided a static moment capacity of 640" kips (72.3 kN.m) in positive downward bending. The design is based on a steel bridge girder support spacing of 8'-0" (2.44 m) and Cooper's E-80 loading. In addition, each prestressed concrete tie is assumed to carry 1/3 of the static axle loading plus a 60 percent impact factor.

A total of 10 prestressed bridge ties had 44 strain gauge circuits connected to them. The circuits had previously been calibrated so as to establish the relationship between bending moment and strain. In 1984 and again in 1986 C.N. Rail
Figure 1.3 Test Bridge Before and After the Installation of Prestressed Concrete Bridge Ties
recorded the strains caused by both passenger and freight train traffic over this bridge.

In 1988 during an inspection it was found that 17 of the 22 bridge ties had cracks in their centres. The cracks start at the top of the ties and progress towards the middle of the tie. It was obvious that the ties were cracking due to negative bending and must be subjected to rebound. There were no cracks from positive bending. In a follow up inspection conducted in May of 1990, it was determined that the number of cracked ties had increased to 18.

1.4 THE CURRENT PROBLEM
Although 18 of the 22 existing C.N. bridge ties have cracks at their top centres, tests performed (15, 16) indicate that these ties can carry axle loads for 20 million cycles. However, the problem is not one of impending structural failure, but one of durability and resistance to damaging freeze thaw cycles. This problem has previously been recognized (6, 11, 15, 16) and therefore the need to design ties which do not crack from rebound is a must.
CHAPTER 2

THE EXISTING C.N. BRIDGE TIE

2.1 GENERAL

The existing C.N. prestressed concrete bridge tie, designated as the Type A modified design, is trapezoidal in cross section. It has a top width of 10" (254 mm) and a bottom width of 11-23/32" (298 mm). It has an overall height of 12" (305 mm) and an overall length of 12'-0" (3,658 mm). It is prestressed by the use of 32 - 0.196" (5 mm) diameter indented steel wires. The existing C.N. design concrete bridge tie is illustrated in Figure 2.1.

2.2 DESIGN PARAMETERS

The Type A modified ties were designed assuming the following design parameters:

1) track gauge = 4'-8-1/2" (1,435 mm)
2) centre to centre of rails = 5'-0" (1,524 mm)
3) rail section is 132 lb. R.E.
4) centre to centre of girder supports = 8'-0" (2.44 m)
   actual spacing of girders = 7'-10" (2.39 m)
5) tie spacing to be 16" (406 mm) on centre
6) design loading based on Cooper's E-80
7) impact based on 60 percent
8) static wheel load distribution = 33.33 percent
Figure 2.1 Cross Section of C.N. Rail Prestressed Concrete Bridge Tie
2.3 MATERIAL PROPERTIES

The type A modified ties were constructed from materials having the following minimum specified material properties:

a) Concrete:

Compressive strength at 28 days = 6,000 psi (41.4 MPa)
Compressive strength at transfer = 4,500 psi (31.0 MPa)
Flexural stresses after transfer:
Compression: \(0.6f_{ci}' = 0.6 \times 4,500 = 2,700\) psi (18.6 MPa)
Tension: \(3(f_{ci}')^{1/2} = 3(4,500)^{1/2} = 201\) psi (1.4 MPa)
Flexural stresses at service loading after losses:
Compression: \(0.4f_{c}' = 0.4 \times 6,000 = 2,400\) psi (16.5 MPa)
Tension: \(3(6,000)^{1/2} = 232\) psi (1.6 MPa)
(although in actual design no tension was allowed)

b) Prestressing Steel:

0.193" (4.9 mm) diameter indented wire
Area of prestressing wire; \(A_{ps} = 0.029\) in\(^2\) (18.9 mm\(^2\))
Minimum tensile strength; \(f_{pu} = 237,000\) psi (1,634 MPa)
Jacking stress; \(0.80f_{pu} = 189,600\) psi (1,307 MPa)
Stress at transfer; \(0.7f_{pu} = 165,900\) psi (1,144 MPa)
Stress at service load; \(0.6f_{pu} = 142,200\) psi (980 MPa)
Modulus of elasticity; \(E_s = 29 \times 10^6\) psi (200 x 10\(^3\) MPa)

2.4 GEOMETRIC PROPERTIES

The ties also possess the following geometric properties:
The area of the cross section is:
\(A_c = d(b + b_t)/2 = 12(10 + 11.71)/2\)
\[ A_c = 130.28 \text{ in}^2 (84,064 \text{ mm}^2) \]

The moment of inertia of the tie is:

\[ I = d^3(b^2 + 4bb_t + b_t^2)/36(b + b_t) \]

\[ I = 12^3(11.71^2 + 4 \times 11.71 \times 10 + 10^2)/36(10 + 11.71) \]

\[ I = 1,560.16 \text{ in}^4 (649.4 \times 10^6 \text{ mm}^4) \]

The distance from the neutral axis to the top of concrete fibre is:

\[ y_t = d(2b + b_t)/3(b + b_t) \]

\[ y_t = 12(2 \times 11.71 + 10)/3(10 + 11.71) \]

\[ y_t = 6.16 \text{ in} (156.5 \text{ mm}) \]

The distance from the neutral axis to the bottom outer concrete fibre is:

\[ y_b = d - y_t = 12 - 6.26 = 5.84" (148.3 \text{ mm}) \]

The top and bottom section moduli are:

\[ S_t = I/y_t = 1,560.16/6.16 = 253.27 \text{ in}^3 (4,150 \times 10^3 \text{ mm}^3) \]

\[ S_b = I/y_b = 1,560.16/5.84 = 267.15 \text{ in}^3 (4,378 \times 10^3 \text{ mm}^3) \]

The distance from the centroid of the prestressing wire to the top of tie is:

\[ \Sigma n_i d_i/\Sigma n_i = 7.78" (198 \text{ mm}) \]

The distance from the centroid of the prestressing wire to the neutral axis (the eccentricity) is:

\[ e = 7.78 - 6.16 = 1.62" (41.1 \text{ mm}) \]

The static wheel load distribution factor = 33.33 percent

The prestressing loss = 27 percent

The initial prestressing force was:

\[ P_i = 32 \times 5,170 = 165,440 \text{ lbs. (735.9 kN.)} \]
The effective prestress force was:

\[ P_{\text{eff}} = P_i \times 0.73 = 165,440 \times 0.73 = 120,771 \text{ lbs. (537.2 kN)} \]

2.5 DESIGN LOADS

In determining the loads on the tie Cooper's E-80 loading, 60 percent impact factor and 33.33 percent static wheel load distribution were used.

The service live load moment is:

\[ M_{\text{t,l}} = 80 \times 1.60 \times 33.33 \times 18 \times 1/2 \times 1/100 \]

\[ M_{\text{t,l}} = 384 \text{ in.kips (43.4 kN.m)} \]

The moment due to self weight of the ties is:

\[ M_b = 6 \times 12 \times 138 \times 1/1,000 = 9.94 \text{ in.k (1.12 kN.m)} \]

The moment due to superimposed dead loads (rail & fasteners)

\[ M_{\text{sd}} = 4 \times 12 \times 62 \times 1/1,000 = 2.98 \text{ in.k (0.34 kN.m)} \]

The total moment \( M_{\text{tot}} = M_b + M_{\text{sd}} + M_{\text{t,l}} \) is:

\[ M_{\text{tot}} = 9.94 + 2.98 + 384.0 = 396.9 \text{ in.k (44.85 kN.m)} \]

2.6 CONCRETE STRESSES

The top fibre stress with no service live load is:

\[ f_t = \frac{P}{A} - \frac{P e Y_t}{I} + \frac{(M_D + M_{SD}) Y_t}{I} \]

\[ f_t = \frac{120.77}{130.28} - \frac{120.77 \times 1.62 \times 6.16}{1,560.16} + \frac{12.92 \times 6.16}{1,560.16} \]

\[ f_t = 0.21 \text{ ksi (1.5 MPa) compression} \]

The bottom fibre stress with no service live load is:

\[ f_b = \frac{P}{A} + \frac{P e Y_b}{I} - \frac{(M_D + M_{SD}) Y_b}{I} \]
\[ f_b = \frac{120.77}{130.28} + \frac{120.77 \times 1.62 \times 5.84}{1,560.16} - \frac{12.92 \times 5.84}{1,560.16} \]

\[ f_b = 1.61 \text{ ksi (11.1 MPa) compression} \]

The top fibre stress with service live loading is:

\[ f_t = \frac{P}{A} - \frac{P \cdot e \cdot Y_t}{I} + \frac{M_{\text{tot}} \cdot Y_t}{I} \]

\[ f_t = \frac{120.77}{130.28} - \frac{120.77 \times 1.62 \times 6.16}{1,560.16} + \frac{396.9 \times 6.16}{1,560.16} \]

\[ f_t = 1.72 \text{ ksi (11.9 MPa) compression} \]

The bottom fibre stress with service live loading is:

\[ f_b = \frac{P}{A} + \frac{P \cdot e \cdot Y_b}{I} - \frac{M_{\text{tot}} \cdot Y_b}{I} \]

\[ f_b = \frac{120.77}{130.28} + \frac{120.77 \times 1.62 \times 5.84}{1,560.16} - \frac{396.9 \times 5.84}{1,560.16} \]

\[ f_b = 0.21 \text{ ksi (1.5 MPa) compression} \]

### 2.7 TIE CRACKING STRENGTH

The concrete bridge ties were manufactured in June of 1984 by Conforce Costain of Edmonton, Alberta. Although there were no 28 day cylinder tests performed specifically on these ties, there were 28 day cylinder tests done on all batches of concrete poured that month. The average 28 day cylinder tests for concrete poured during the month of June, 1984 at the Costain plant was 8,620 psi (59.4 MPa). This was well above the minimum specified 28 day compressive strength of 6,000 psi (41.4 MPa).
Concrete is assumed to crack when the tensile stress reaches the modulus of rupture, $f_r$ or $0.1f'_c$.

The modulus of rupture is given by:

$$f_r = 7.5(f'_c)^{1/2}$$

where $f'_c = 6,000 \text{ psi (41.4 MPa)}$

$$f_r = 7.5(6,000)^{1/2} = 696 \text{ psi (4.8 MPa)}$$

Under positive bending the cracking moment $M_{cr}$ is:

$$M_{cr} = \frac{(f_r + f_b) I}{Y_b}$$

$$M_{cr} = \frac{(0.58 + 1.66) \times 1,560.16}{5.84} = 598.4 \text{ in.k (71.2 kN.m)}$$

Under negative bending the cracking moment $M_{cr}$ is:

$$M_{cr} = \frac{(f_r + f_c) I}{Y_t}$$

$$M_{cr} = \frac{(0.58 + 0.16) \times 1,560.16}{6.16} = 187.4 \text{ in.k (21.2 kN.m)}$$

$$P_{cr} = \frac{M_{cr}}{18} = \frac{187.4}{18} = 10.41 \text{ kips (46.3 kN)}$$

### 2.8 Ultimate Moment Capacity

The ultimate moment capacity of the CN concrete bridge tie was calculated by the stress - strain compatibility method outlined in Chapter 6.

The ultimate moment capacity of the tie in positive bending has been calculated to be:
\[ M_u = 1,176.75 \text{ in.k} \quad (133.0 \text{ kN.m}) \]
\[ P_u = 65.38 \text{ k} \quad (290.8 \text{ kN}) \]

The ultimate moment is therefore achieved by the application of two loads equal to 65.38 k (290.8 kN) on each rail, as outlined below.

\[ P_u = 65.38 \text{ k} \quad (290.8 \text{ kN}) \]

\[ P_u = 65.38 \text{ k} \quad (290.8 \text{ kN}) \]

Figure 2.2 Diagram of Load Application

2.9 FATIGUE CRACKING MOMENT

The fatigue strength of concrete is very much dependent on; the range of loading, the rate of loading, prior loading history, material properties and environmental conditions. Of these, the most important influence is the range of loading which in turn dictates the stress range within the concrete. This stress range is the difference between the minimum and maximum stresses experienced by the concrete under repetitive loading.
In the calculation of the fatigue cracking moment of the concrete tie (method used by the CN Technical Research Centre), it has been assumed that concrete has zero tensile strength in fatigue. This is a valid assumption as it indicates that the fatigue bending capacity of the tie is directly related to the level of precompression within the tie.

The positive fatigue moment is given by:

\[ M_f = (f_b \times I)/Y_b \]

\[ M_f = (1.66 \times 1,560.16)/5.84 = 443.5 \text{ in.k} \ (50.1 \text{ kN.m}) \]

\[ P_f = \frac{M_f}{18} = \frac{443.5}{18} = 24.6 \text{ kips} \ (109.4 \text{ kN}) \]

The negative fatigue moment is given by:

\[ M_f = (f_t \times I)/Y_t \]

\[ M_f = (0.16 \times 1,560.16)/6.16 = 40.5 \text{ in.k} \ (4.6 \text{ kN.m}) \]

\[ M_f = 40.5 \text{ in.k} \ (4.6 \text{ kN.m}) \]
CHAPTER 3
THE C.N. FIELD TESTS

3.1 EVALUATION
In order to evaluate the design of the 22 prestressed ties testing was carried out in September of 1984 and again in April of 1986. For each test a different rail-tie and tie-girder pad configuration was used. In addition, the type of approach ties to the bridge were varied.

3.2 THE 1984 BRIDGE CONFIGURATION
The 22 prestressed concrete ties were installed at 16\" (406 mm) centres on two girders, at 8'-0" (2.44 m) centres, spanning 29' (8.84 m). The ties were numbered consecutively from 1 to 22 starting at the east end of the bridge. The approach ties to the bridge consisted of 11 timber ties on 20" (508 mm) centres.

3.3 RAIL-TIE AND TIE-GIRDER PADS (1984)
Two types of tie-girder pads were used which determined two regions of influence. These were:

a) the hard SA-47 tie-girder pads placed under ties numbered 1 to 11.

b) the soft AASHTO, 60 durometer tie-girder pads placed under ties numbered 12 to 22.

Two types of rail-tie pads were used which were placed over each type of tie-girder pad. These were as follows:

a) the hard EVA rail-tie pads placed on ties numbered 1 to
4 inclusive and ties numbered 12 to 18 inclusive.

b) the soft SBR, 60 durometer rail-tie pads placed on ties numbered 5 to 11 inclusive and ties numbered 19 to 22 inclusive.

Figure 3.1 indicates the areas of influence of the rail-tie and the tie-girder pads.

3.4 STRAIN GAUGE CONFIGURATION (1984)

The following structural members were instrumented with strain gauge rosettes:

i) the concrete ties at top and bottom of their vertical faces.

ii) the girders at the top and bottom flanges on either side of the web.

Due to the symmetry of the structure the instrumentation was applied on one side of the structure only. A list of the strain gauges monitored in 1984 is shown in Table 3.1.

The following abbreviations are used to designate the location of the strain gauges on the structure:

3GT and 3GB - strain gauges located on the vertical face of concrete tie number 3, over the girder support. "T" indicates top of vertical face, 2" (51 mm) from the top; "B" indicates the bottom of the vertical
Figure 3.1 Regions of Influence for Rail-Tie and Tie-Girder Pads
<table>
<thead>
<tr>
<th>TIE NO.</th>
<th>GIRDER TOP</th>
<th>GIRDER BOTTOM</th>
<th>RAIL SEAT TOP</th>
<th>RAIL SEAT BOTTOM</th>
<th>TIE CENTRE TOP</th>
<th>TIE CENTRE BOTTOM</th>
<th>WIRE/STEEL GIRDER</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1GT</td>
<td>1GB</td>
<td>1RT</td>
<td>1RB</td>
<td>1CT</td>
<td>1CB</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>3GT</td>
<td>3GB</td>
<td>3RT</td>
<td>3RB</td>
<td>3CT</td>
<td>3CB</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>7CT</td>
<td>7CB</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>8GT</td>
<td>8GB</td>
<td>8RT</td>
<td>8RB</td>
<td>8CT</td>
<td>8CB</td>
<td>W1 AND W2</td>
</tr>
<tr>
<td>9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>9CT</td>
<td>9CB</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>G1,G2,G3,G4</td>
</tr>
<tr>
<td>14</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>14CT</td>
<td>14CB</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>15GT</td>
<td>15GB</td>
<td>15RT</td>
<td>15RB</td>
<td>15CT</td>
<td>15CB</td>
<td>W3 AND W4</td>
</tr>
<tr>
<td>16</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>16CT</td>
<td>16CB</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>20GT</td>
<td>20GB</td>
<td>20RT</td>
<td>20RB</td>
<td>20CT</td>
<td>20CB</td>
<td></td>
</tr>
<tr>
<td>22</td>
<td>22GT</td>
<td>22GB</td>
<td>22RT</td>
<td>22RB</td>
<td>22CT</td>
<td>22CB</td>
<td></td>
</tr>
</tbody>
</table>

**TABLE 3.1** Concrete Bridge Tie Strain Gauge Circuits for the 1984 Tests Location and Designation
face 1-1/2" (38 mm) from the bottom.

8RT and 8RB - strain gauges located on the vertical face of the concrete tie number 8 under the rail seat at the top and bottom of the vertical face as indicated above.

15CT and 15CB - strain gauges located on vertical face of concrete tie number 15 at the centre of the tie, at the top and bottom of the vertical face as indicated above.

W1 and W2 - two strain gauges located on prestressing wires, 7" (177.8 mm) apart and 1-3/4" (44.5 mm) from the base of tie number 8.

W3 and W4 - two strain gauges located on prestressing wires, 7" (177.8 mm) apart and 1-3/4" (44.5 mm) from the base of tie number 15.

G1, G2, G3, & G4 - the four strain gauges located on the four flanges of one of the girders directly under tie number 4.

Figure 3.2 illustrates the locations of the strain gauges on the C.N. field test ties.

3.5 THE 1986 BRIDGE CONFIGURATION

Between 1984 and 1986 hairline positive bending cracks were detected in the two end ties of the bridge, namely ties 1 and 22. These cracks were initiated from the high impact loading associated with the transition from the timber approach ties
Figure 3.2 Location of Strain Gauges on Field Test Ties
on ballast to the concrete ties at the abutments of the bridge. This is commonly referred to as a variation in track modulus.

To reduce the impact at the abutments, the timber approach ties at each end of the bridge were replaced by 50 concrete track ties, spaced at 24" (610 mm) centre to centre. In addition the cracked bridge ties (numbers 1 and 22) at the abutments were replaced with spares.

3.6 RAIL AND TIE GIRDER PADS (1986)
The 1986 test configuration was identical to the 1984 configuration except that the 5 mm EVA rail-tie pads were removed and replaced by 6.5 mm Hytrel studded pads. The measurements and data analysis focused on the evaluation of the new Hytrel rail-tie pads, in order to determine if there was any improvement in the reduction of high strains. These strains would be reduced by the attenuation of impact by the pads and the smoother transition at the abutments provided by the 50 concrete track ties.

3.7 STRAIN GAUGE CONFIGURATION (1986)
From a preliminary analysis of the 1984 results, it was found that the maximum static strains occurred as expected between the rail seats at the bottom fibre of the tie. However, the maximum dynamic strains occurred at the rail seat bottom
locations. Therefore, the 1986 test concentrated primarily on the rail seat bottom (RB) locations as these were the locations of maximum strain. In the 1986 tests a total of 8 prestressed concrete bridge ties were instrumented with 10 strain gauge rosettes. The list of strain gauges monitored in the 1986 test is shown in Table 3.2.

3.8 CALIBRATION MEASUREMENTS
In the evaluation of the 1984 and 1986 data on concrete bridge ties two types of measurements were calibrated with respect to the applied moment, these were:
1) bending strain measurements of the concrete ties and
2) tensile strain measurements of the prestressing wire

3.9 BENDING STRAINS ON CONCRETE BRIDGE TIES
The result of the calibrations of the concrete ties is plotted in the graph shown in Figure 3.3. This curve was used for the calibration of the 1984 and 1986 measurements for all concrete bridge tie bending strain gauges.

It should be noted that the strain gauge rosette readings are expressed in terms of $\epsilon_x(1+\mu)$ where:
$\mu$ is Poisson's Ratio for concrete (determined to be $\mu = 0.18$) and $\epsilon_x$ is the bending strain.
<table>
<thead>
<tr>
<th>TIE NO.</th>
<th>RAIL SEAT TOP</th>
<th>RAIL SEAT BOTTOM</th>
<th>TIE CENTRE BOTTOM</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>1RB</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>3RB</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td></td>
<td>14RB</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>15RT</td>
<td>15RB</td>
<td>15CB</td>
</tr>
<tr>
<td>16</td>
<td></td>
<td>16RB</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td></td>
<td>20RB</td>
<td></td>
</tr>
<tr>
<td>22</td>
<td>22RT</td>
<td>22RB</td>
<td></td>
</tr>
</tbody>
</table>

**TABLE 3.2 Concrete Bridge Tie Strain Gauge Circuits for the 1986 Tests - Location and Designation**
Figure 3.3 Calibration Curve for Concrete Ties
3.10 STRAINS OF PRESTRESSING WIRES

Strain gauges were installed on the initially unstressed prestressing wires of tie number 8 and 15 at the manufacturing plant. The strain gauge rosette readings are again expressed in terms of $\epsilon_x(1+\mu)$ where:

$\mu$ is Poisson's Ratio for steel = 0.30

and $\epsilon_x$ is the tensile strain

Each strain gauge was calibrated on its respective wire and the results are shown in Figures 3.4, 3.5, 3.6 and 3.7. The strain gauges were monitored during the pouring of the concrete, through the transfer of the prestress and up until the ties were removed from the forms. Relaxation of the wires over time is shown in Figure 3.8 and 3.9.

3.11 INSTRUMENTATION

All data recorded was in the form of bending strains. The circuits were temperature compensated by placing rosette gauges in a wheatstone bridge, with the signals amplified and transmitted by means of cables to a tape recorder. These signals were also played out on a high speed chart recorder.

The strain gauges were 2" (51 mm) long and had a resistance of 120 ohms. At each measurement location two strain gauges were stacked perpendicular to each other and connected in a rosette configuration to compensate for any temperature variations. But the temperature compensating gauges, placed perpendic-
Figure 3.4  Calibration Curve for Wire 1
Figure 3.5 Calibration Curve for Wire 2
Figure 3.6 Calibration Curve for Wire 3
Figure 3.7 Calibration curve for wire 4

Strain (micronches/inch x 1000) vs. Applied Load (kips)
Figure 3.8 Relaxation of Wires 1 and 2 Over Time
STRAIN GAUGED REINFORCING WIRES
INSTALLATION IN BRIDGE CONCRETE TIES

Figure 3.9 Relaxation of Wires 3 and 4 Over Time
ularly to the direction of principle strain, measure the Poisson's effect of strain $\mu \epsilon_b$. Therefore, the total strain output of each rosette is $(1+\mu)\epsilon_b$, where $\epsilon_b$ is the bending strain.

The two strain gauges of the rosette were connected inside a two wire transmitter box (or T.W.T.), in series to the left branch of a wheatstone bridge. It was configured to form a half bridge with two 350 ohm dummy resistors placed in the opposite branch. The two wire transmitter box provides the wheatstone bridge with voltage excitation of the bridge, and an amplification of 500 of the bridge voltage output with a conversion of the voltage into a current signal. The mathematical expression of voltage change for this system (22) is presented in Appendix A.

This current signal is transmitted directly to a receiver box along data cables across a constant differential voltage but with floating reference voltage. In this manner the measurements do not suffer voltage drops or noise pick up along the length of the cables.

The receiver box converts the current signal back into a voltage signal. Also, it permits the balancing of the wheatstone bridge and the setting of a variable gain.
The two wire transmitter and receiver system provides an amplitude resolution of the micro volts. The frequency response was flat to 2,000 Hz and the circuits were calibrated through an internal calibration system.

The receiver output from each measurement location was recorded onto a tape recorder and simultaneously played on a chart recorder. There were 52 strain gauge circuits and data was taken in stages with 3 tape recorders being used in parallel. As a reference in time, the three tape recorders recorded in parallel with a common signal to each.

The three tape recorders were FM type recorders made by Kyowa. One was a 14 channel RT-600B, while the other two were 6 channel RTP-501A's. Their basic characteristics were a tape speed of 3.75 in/s (95.2 mm/s), a signal to noise ratio of 46 decibels and a frequency response which was flat up to 2,500 Hz.

The chart recorder was a Datagraph model HR2000 made by Bell and Howell. It is a light sensitive chart recorder with a flat frequency response up to 5,000 Hz.

3.12 THE 1984 AND 1986 TEST PROCEDURE

The measurements taken in 1984 were undertaken in three stages which are shown in Table 3.3. The first stage was set up to
evaluate the rail-tie and tie-girder pad combinations at each end of the bridge. These are as follows:
EVA and SA-47 for ties numbered 1 to 4
SBR-60A and AASHTO for ties numbered 19 to 22

The second set up was to evaluate the following combination of rail-tie and tie-girder pads:
SBR-60A and SA-47 for ties numbered 5 to 11
EVA and AASHTO for ties numbered 12 to 18

The third set up was to evaluate the following combination of rail-tie and tie-girder pads:
EVA and SA-47 for ties numbered 1 to 4
SBR-60A and SA-47 for ties numbered 5 to 11
EVA and AASHTO for ties numbered 12 to 18
SBR-60A and AASHTO for ties numbered 19 to 22

The data obtained in 1986 was undertaken in one set up and consisted mainly of measuring strains at the rail seat bottom of tie location as this is the location of maximum strains. In this set up the 5 mm EVA pads were replaced by 6.5 mm Hytrel pads and the ties over the abutments (ties 1 and 22) were replaced. In addition, 50 concrete track ties were installed on each approach of the bridge, replacing timber ties. The set up is also shown in Table 3.3.
### CONCRETE BRIDGE TIE TESTS - STRAIN GAUGE SETUP FOR 1984

<table>
<thead>
<tr>
<th>TIE #</th>
<th>1</th>
<th>3</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>14</th>
<th>15</th>
<th>16</th>
<th>20</th>
<th>22</th>
<th>TRAINS</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>SET UP NO. 1</strong></td>
<td>GB/VT</td>
<td>GB/VT</td>
<td>GB/VT</td>
<td>RB/RT</td>
<td>CB/CT</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>RB/RT</td>
<td>CB/CT</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>SET UP NO. 2</strong></td>
<td></td>
<td></td>
<td>CB/CT</td>
<td>GB/VT</td>
<td>RB/RT</td>
<td>CB/CT</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>7</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>CB/CT</td>
<td>CB/CT</td>
<td>CB/CT</td>
<td>CB/CT</td>
<td>CB/CT</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>SET UP NO. 3</strong></td>
<td>RB/RT</td>
<td>CB</td>
<td>CB</td>
<td>RB/RT</td>
<td>CB</td>
<td>CB</td>
<td>CB</td>
<td>CB</td>
<td>CB</td>
<td>CB</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td>CB</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### CONCRETE BRIDGE TIE TESTS - STRAIN GAUGE SETUP FOR 1986

<table>
<thead>
<tr>
<th>SET UP NO. 1</th>
<th>RB</th>
<th>RB</th>
<th></th>
<th>RB</th>
<th>RB/RT</th>
<th>RB</th>
<th>RB</th>
<th>RB/RT</th>
<th></th>
<th>12</th>
</tr>
</thead>
</table>

**TABLE 3.3** Concrete Bridge Tie Tests - Strain Gauge Setup for 1984 and 1986
CHAPTER 4

ANALYSIS OF THE C.N. FIELD TEST DATA

4.1 GENERAL
The strains measured at the rail seat top and rail seat bottom locations were obtained from the chartgraph printout of the results. These strains were statistically analyzed and the percent occurrence of strains and the percent exceeding of strains have been summarized and plotted with the resulting graphs included in Appendix B. Some of the graphs however, have been included in this chapter for completeness.

4.2 1984 RESULTS FOR POSITIVE BENDING
From a review of summary graphs 3RB, 8RB, 15RB, and 20RB for positive bending in 1984, Figure 4.1, 4.2, 4.3 and 4.4 respectively, it can be seen that in each graph two distinct regions are present. The left hand portion of each graph clearly indicates the strains due to quasi static loading with little or no impact. However, the right hand portion of each graph is clearly the combination of both. Each graph contains two distinct peaks at the left portion of the graph. These peaks are due to the static strains caused by empty cars and loaded cars with normal wheels. Thereafter, the graphs drop off to the right and indicate the small percentage of wheels which cause the high impacts.

It is evident from the summary graph of tie 20RB that the soft rail-tie pad and soft tie-girder pad combination gave the
Figure 4.1 Summary Graph 3RB for 1984
%OCCURRENCE vs STRAIN
SUMMARY 8RB - POSITIVE BENDING 1984

Figure 4.2 Summary Graph 8RB for 1984
%OCCURRENCE vs STRAIN
SUMMARY 15RB - POSITIVE BENDING 1984

Figure 4.3 Summary Graph 15RB for 1984
lowest total strains. This indicates that the softer rail-tie pad attenuates more impact than a hard pad.

The largest total strains occurred when a hard rail-tie pad and hard tie-girder pad combination were used. This is indicated in summary graph 3RB, Figure 4.1. It should be noted that there were 50 percent higher strain values recorded for this combination as compared to the soft rail-tie pad and soft tie-girder pad combination.

In comparing the summary graphs 3RB, Figure 4.1 with 15RB, Figure 4.3, hard rail-tie pad with hard tie-girder pad and hard rail-tie pad with soft tie-girder pad respectively, it can be seen that there is no difference in the largest strains recorded. This therefore indicates that the strain attenuation in the system is influenced the greatest by the rail-tie pad and very little by the tie-girder pad. This is consistent with recent laboratory findings (9, 10), which concluded that the tie-girder pad had an insignificant effect on the dynamic strains experienced by the tie.

By comparing summary graphs 15RB, Figure 4.3 and 20RB, Figure 4.4, the same conclusion as above can be reached as both these ties had the soft tie-girder pads.

From summary graphs 8RB, Figure 4.2 and 20RB, Figure 4.4 it
can be seen that these ties had the lowest recorded total strains, which is directly attributed to the use of the soft rail-tie pad.

It is worth noting that the use of the soft rail-tie pads causes a small redistribution of the static wheel load to occur. This can be seen in the shifting of the peaks to the left on summary graphs 8RB Figure 4.2 and 20RB, Figure 4.4. This redistribution is attributed to the combination of rail-tie and tie-girder pads used, but is greatest when a soft rail-tie and soft tie-girder pad combination is used.

The dynamically loaded area of the graphs is controlled mainly by the type of rail-tie pad used.

4.3 THE 1984 RESULTS FOR NEGATIVE BENDING

The strains from the 1984 negative bending results have been plotted in Figure 4.5, 4.6 and 4.7. The results indicate that the maximum negative bending strains vary between $103 \times 10^{-6}$ to $129.9 \times 10^{-6}$. These strains are caused by impact and are purely dynamic strains.

In comparing Figures 4.5, 4.6 and 4.7 it can been seen that summary graph 15RT, Figure 4.6, which had the hard rail-tie pad and soft tie-girder pad experienced the lowest negative bending strains and that summary graphs 8RT, Figure 4.5 and
Figure 4.5 Summary Graph 8RT for 1984
Figure 4.6 Summary Graph 15RT for 1984
Figure 4.7 Summary Graph 20RT for 1984
20RT, Figure 4.7, which had soft rail tie-pads experienced the largest strains.

The observation here is that soft rail-tie pads apparently provide for greater negative bending strains. However, the statistics may not be meaningful in this regard, as the graph that could have clarified this could not be drawn because of a malfunctioning gauge which was subsequently disconnected. Therefore, the data for the hard rail-tie pad and hard tie-girder pad is not available.

What can be said with certainty is that the maximum strain experienced in negative bending was $129.9 \times 10^{-6}$. The C.N. Rail tie was designed for 210 psi (1.5 MPa) compressive tie stress, which is equivalent to a strain of $47.5 \times 10^{-6}$. Therefore, the C.N. Rail tie is under designed by $129.9 \times 10^{-6} - 47.5 \times 10^{-6} = 82.4 \times 10^{-6}$.

### 4.4 1986 RESULTS FOR POSITIVE BENDING

The results of the 1986 positive bending strains have been plotted in Figure 4.8 and 4.9. Again, as in the 1984 positive bending results, there are two distinct regions to the graphs. The left hand portion of each graph indicates the strains due to quasi static loading with little or no impact present. The right hand portion of the graphs indicate a combination of both quasi static and dynamic strain. In addition each graph
Figure 4.8 Summary Graph 3RB for 1986
Figure 4.9 Summary Graph 15RB for 1986
contains two distinct peaks. These peaks are due to the static strains caused by the empty rail vehicles (cars) and loaded railway vehicles with normal wheels. The graphs then drop off to the right and indicate the small percentage of defective wheels that cause the high impacts.

A comparison of the 1984 graphs for positive bending indicates that the attenuation capability of the Hytrel rail-tie pads is greater than that of the SBR rail-tie pads. This indicates that the 6.5 mm studded Hytrel pad performs approximately 20 percent better in attenuating the high impacts than the SBR pads as the highest strain recorded is 225 x 10^-6 in./in.

4.5 COMPARISON OF 1984 AND 1986 RESULTS FOR TIES 1 AND 22
The data for 1984 and 1986 for ties 1 and 22 (end ties) has been plotted and is shown in Figures 4.10, 4.11, 4.12 and 4.13.

A comparison of the graphs for 22RB, Figures 4.12 and 4.13 in positive bending indicates an 18 percent reduction in the strains from 1984 to 1986. This reduction of strains is due to the combination of the installation of 50 concrete approach ties and the installation of the Hytrel pads.

It should be noted that there was no appreciable reduction in the strains in tie number 1. The reason for this is unknown.
%OCCURRENCE vs STRAIN
SUMMARY 1RB – POSITIVE BENDING 1984

Figure 4.10 Summary Graph 1RB for 1984
Figure 4.11 Summary Graph 1RB for 1986
%OCCURRENCE vs STRAIN

SUMMARY 22RB - POSITIVE BENDING 1984

Figure 4.12 Summary Graph 22RB for 1984
4.6 EVALUATION OF IMPACT FACTORS

In the evaluation of the data, it is apparent that the impacts being experienced were generated by both lightly loaded as well as heavily loaded cars. Figures 4.14 and 4.15 indicate the relationship between speed and average strains, comprised of static and dynamic strain. From these two graphs come some important observations.

For lightly loaded vehicles:
1) The lightly loaded vehicles exhibit large dynamic strains in relation to the static strains. This in turn means that they display large impact factors.
2) The total strains increase with increasing speed, and then decrease after about 52 mph (85.3 km/hr).
3) The dynamic strains start at approximately 26 mph (42.6 km/hr) and increase up to approximately 52 mph (85.3 km/hr), where after they decrease.

For heavily loaded vehicles:
1) The heavily loaded vehicles exhibit larger overall strains than the lightly loaded vehicles, but smaller dynamic strains in relation to the static strains. This means that the impact factor under heavily loaded cars is smaller than lightly loaded cars.
2) The total strains generally increase up to approximately 52 mph (85.3 km/hr) whereafter they decrease.
3) The dynamic strains start at approximately 32 mph (52.5 km/hr) reaching a maximum at 52 mph (85.3 km/hr)
Figure 4.14  Average Strain Versus Speed for Light Wheel Loads
AVERAGE STRAINS vs SPEED
HEAVY WHEEL LOADS

Figure 4.15 Average Strain Versus Speed for Heavy Wheel Loads
Figure 4.16 Average Impact Factors Versus Speed for Light Wheel Loads
Figure 4.17 Average Impact Factors Versus Speed for Heavy Wheel Loads
whereafter they decrease.

Figures 4.16 and 4.17 indicate the average impact factors for light wheel loads and heavy wheel loads. What is immediately evident on comparison of these graphs is that the lighter wheel loads exhibit greater impact factors than heavier wheel loads. This would indicate that larger impacts are more a function of both the unsprung mass of the railway vehicles and of the vehicle speed, rather than that of the total vehicle mass.

A comparison of the graphs in Figures 4.14 and 4.15 indicate the dynamic strains for light wheel loads and heavy wheel loads are comparable in magnitude. This is because the unsprung mass of the heavily loaded vehicles is the same as that of the lightly loaded vehicle and would, therefore, generate the same dynamic strains for similar wheel defects or rail irregularities at comparable velocities.

4.7 INCREASED STRENGTH OF CONCRETE UNDER IMPACT

It can be seen from the impact factors measured during the 1984 field test, that the majority are greater than the design factor of 60 percent. However, the concrete ties have not cracked in positive bending due to the large impacts. There are two main reasons for this phenomena which are outlined below:
1) The rate at which the ties are loaded is so fast, approximately 3 milliseconds, and because of the ties' mass and inertia, the tie does not have a chance to respond completely as the force which caused the impact has disappeared.

The energy of the impact is, therefore, dissipated in a number of ways as follows:

a) through the rail (longitudinally)
b) by deflection of the pad
c) by deflection of the tie
d) by heat, and
e) by noise

Impact load application rates of 7 to 20 milliseconds are considered the most damaging as the load duration interval increases the ties response to the load.

2) Concrete is considered to be a stress-rate sensitive material. Recent work (2) concluded that, in general, concrete is stronger and more energy absorbing under impact than under static loading. Studies (18, 20) have shown that strain values observed at or near failure in dynamic tests for high rates of loading were significantly larger than corresponding values in the static tests. In addition, the resistance to impact (measured by the ability to absorb strain energy)
increased significantly with the rate of loading. It has also been noted that weaker strength concretes will show greater strength improvement than higher strength concretes.

4.8 A.R.E.A. SPECIFIED IMPACT FACTORS

The American Railway Engineering Association (A.R.E.A.) specifies the use of an assumed 200 percent impact factor (I) to account for the dynamic effect of wheel and rail irregularities. Due to premature structural failures of concrete ties (6), the bending strength specifications of the A.R.E.A. have steadily increased in response to these failures. The manual does not specifically address bridge ties and the assumed impact factor of 200 percent does not assist in the design of ties subjected to rebound effects.

4.9 FREQUENCY RESPONSE OF THE BRIDGE TIES

The flexural resonant frequency response of a prestressed concrete bridge tie is dependent on four physical parameters. These are:

1) $I$, the moment of inertia of the tie
2) $L$, the length of the tie
3) $W$, the weight per unit length of the tie
4) $A_n$, the mode coefficient representing the mode shape and axle loading (or the mode coefficient for the $n^{th}$ mode.)
The flexural resonant frequency expression commonly used (19) is given by:

\[ F = \frac{A_n}{2\pi} \sqrt{\frac{g EI}{L^4 W}} \sqrt{A_n^2 + \frac{L^2 P}{EI}} \]

However, for all practical purposes, in the frequency response of the bridge ties:

\[ \frac{L^2 P}{EI} \ll A_n^2 \]

such that the frequency response of the tie can be approximated by:

\[ F = \frac{A_n}{2\pi} \sqrt{\frac{g EI}{L^4 W}} \]

Table 4.18 illustrates the theoretical modes and the corresponding frequency response for the bridge ties.

From the 1984 test data, power spectra frequency analysis was performed (19) which indicated two predominant crests, the first at 88 Hz and the second at 468 Hz.

Frequency response of the bridge ties which have very close agreement with the first and third resonant modes. This indicates that these modes are present at these frequencies.

The power spectra analysis also indicated a sharp valley at 223 Hz. This point is termed a point of anti-resonance and indicates a nodal point. A nodal point is a point of no
<table>
<thead>
<tr>
<th>Resonant Modes</th>
<th>Frequency</th>
<th>Mode Shapes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st Symmetric</td>
<td>86 Hz</td>
<td>![Image]</td>
</tr>
<tr>
<td>2nd Symmetric</td>
<td>240 Hz</td>
<td>![Image]</td>
</tr>
<tr>
<td>3rd Symmetric</td>
<td>472 Hz</td>
<td>![Image]</td>
</tr>
<tr>
<td>4th Symmetric</td>
<td>781 Hz</td>
<td>![Image]</td>
</tr>
</tbody>
</table>

Figure 4.18 Theoretical Resonant Modes and Frequency Response for the C.N. Rail Prestressed Concrete Bridge Tie
motion. This frequency is also close to the theoretical second resonant frequency of 240 Hz, which indicates the presence of the second resonant frequency mode.
CHAPTER 5
PROPOSED DESIGN MODEL

5.1 SYSTEM DYNAMICS
In most cases designers of prestressed concrete bridge ties will not have direct access to field measured strains. As such it may be difficult for the designer to determine the level of strength and magnitude of prestress required for a properly designed tie. See for instance chapter 10 of (1).

The current method employed by most designers is to specify an impact loading by which the wheel loading is factored and increased. The main disadvantages in this method are that it is difficult to determine the correct impact factor and the chosen impact factor does not help in determining the amount of reverse bending to which the ties are subjected. There are other disadvantages in the use of impact factors which have been identified by others (20).

In order to assist designers in the design of prestressed concrete bridge ties two mathematical models which predict the strain levels under any desired loading, have been developed. The first model predicts the strains due to positive bending of the tie, while the second model predicts the strains due to rebound of the tie.

Before presenting the models, an explanation of the interaction of the rail vehicles and the track system is
perhaps in order to provide a better understanding of the mechanics behind the models. The models are based on the principal of conservation of energy and momentum. These methods are less complex and easier to use than equations of a time dependant nature.

The impact imparted to the track structure is generally caused by irregularities in the running surface of the rail, such as crushed heads, corrugation or such defects and by the wheels of the vehicles themselves. Wheel defects such as out-of-round wheels, ovate wheels, built-up-treads or skid flats tend to cause the greatest impacts to the track structure. Therefore a railway which keeps its vehicle wheels in generally good condition would be able to use a lower impact factor and design a more economical tie.

Vehicles with normal wheels would cause the ties and rail to deflect an amount proportional to the vehicle loading as if under static loading, in actuality the deflection would be quasi-static. However, if the vehicle has a wheel defect, the deflection is now two part. There is a deflection due to the static loading of the vehicle with an additional deflection due to the impact loading.

As the rail is impacted by the wheel, the tie deflects downwards beyond its static deflection point to a maximum
deflection whereupon it moves upwards and striking the wheel set of the vehicle at the statically loaded deflection point. This is because the frequency response of the vehicle is between 5 to 10 Hz, whereas the frequency response of the bridge ties is 88 Hz (in the first mode). Therefore the car and wheel set do not move and can be considered motionless or stationary in the vertical plane.

After the track has contacted the wheel set it forces the wheels and axles upward against the springs of the vehicle. This continues until the tie reaches its maximum upward deflection whereupon it is acted upon by the vehicle's sprung mass which forces the tie down to its statically deflected position. The deflection of the tie upward (rebound) is currently causing the CN type A modified ties to crack at their top centres.

5.2 MATHEMATICAL MODEL OF THE CONCRETE BRIDGE TIE RESPONSE IN POSITIVE BENDING

There are three possibilities of wheel rail impact modes from a skid flat wheel. These possibilities are illustrated in Figure 5.1.

In all three impact cases shown, the dynamic forces involved are described in Figure 5.2 below:
1) Complete contact over wheel flat

2) Edge contact of wheel flat

3) Skip over flat - no contact

Figure 5.1 Schematic of Wheel Impacts

71
Figure 5.2 Dynamic Forces During Impact

Where:

\( V \) = linear velocity of the wheel
\( \omega \) = angular velocity of rotation of the wheel
\( R \) = reaction of rail beneath the wheel
\( T \) = traction force of coupler
\( \dot{F} \) = dynamic impulse loading: \( \dot{F} = \int T_0 F_0(t) \, dt \), where \( T \) is the period of impact and \( F_0 \) is the dynamic load
\( \alpha \) = the angular deceleration of axle due to impact
\( \Delta \) = the change in angular velocity due to the impact, such that \( \Delta \omega = \int \alpha(t) \, dt \)

This mathematical model for the impact response of concrete bridge ties is based on a set of assumptions which are presented in the Appendix C. This model also relies on the previous work and findings of (4), which determined the fraction of the impact force on the main tie beneath the wheel to be relatively constant at 50 percent.
Before presenting the model, an outline of the assumptions that have been made are presented as follows:

**Basic Set of Assumptions**

1) The axle and wheel's horizontal velocity is constant, i.e.: the railcar does not change in velocity with a flat wheel.

2) The impact occurs directly over a tie.

3) The impact occurs completely, as the wheel is over the tie and the dynamic deflection takes place about the statically deflected position of the track. This was shown in time traces of tie strains obtained in the CN field tests.

4) The axle and wheel has little to drop, before the impact occurs; (see Appendix C); a period for the wheel flat to rotate past is 6 times greater than that of the wheel to fall.

5) The impact occurs simultaneously on both rails.

6) The rail and tie are rigidly connected.

7) That linear superposition of dynamic and static deformation of tie holds.

8) Consider the worst cause scenario (for design purposes), i.e: the wheel flat impacts the rail with the two surfaces parallel; there is always contact between wheel and rail which is the case for heavily loaded vehicles running up to 40 mph (see Appendix C).

9) The upward velocity of rail prior to impact, is
negligible.

10) For design purposes in calculating positive and negative bending strains at tie center, the first flexural mode of the tie in bending is used in the mathematical development.

11) The impact vertical velocities of wheel, rails and ties prior to impact are assumed negligible compared to their velocities after impact.

12) The change in wheel angular velocity \( \Delta \omega \) is small compared to \( \omega \), due to impact couple and slipping.

**Kinematic Mechanism Of Wheel - Rail Impacts:**

![Diagram](image)

**Figure 5.3 Kinematic Mechanism of Wheel - Rail Impact**

Description of mechanism in wheel - rail kinematics during
impact and up to the moment of separation.

displacement: \[ Y_v = -Y_{TR} + r_Y \]  \hspace{1cm} (1)

velocity: \[ \dot{Y}_v = -\dot{Y}_{TR} + \dot{r}_Y \]  \hspace{1cm} (2)

Where:

\( v \) = velocity of wheel and axle
\( \omega \) = angular velocity of wheel
\( r \) = radius of wheel
\( v = \omega \cdot r \)
\( d \) = length of skid flat
\( Y_w \) = vertical displacement of wheel
\( Y_{TR} \) = vertical displacement of the tie at the rail seat
\( r_Y \) = vertical displacement between axle and rail

\[ r_Y = r \cos \theta_0 - \omega t \]  \hspace{1cm} (3)

\[ \dot{r}_Y = r \omega \sin \theta_0 - \omega t \]  \hspace{1cm} (4)

\[ \dot{Y}_w = -\dot{Y}_{TR} + \dot{r}_Y = r \omega \sin (\theta_0 - \omega t) - \dot{Y}_{TR} \]  \hspace{1cm} (5)

Dynamic Analysis Using Conservation of Momentum and Impulsive Forces

Figure 5.4 Distribution of Impact Forces
\[ \hat{R} = \int_{0}^{T} R(t) \, dt = [\dot{Y}_{W}^{II} - \dot{Y}_{W}^{I}] \, M_{W} \]  \hspace{1cm} (6)

where superscript "I" and "II" denote before and after impact, respectively.

\[ \hat{R} = \sum_{n=1}^{5} \hat{R}_{n} + \lambda \int_{0}^{\lambda} [\dot{Y}_{R}^{II} - \dot{Y}_{R}^{I}] M_{R} \, dz \]  \hspace{1cm} (7)

where \( Y_{R} \) is the deflection of rail and \( \lambda \) is the centre to centre line of 7 ties, and \( M_{R} \) is the mass of rail per unit length.

\[ \hat{R} = \int_{-a}^{t+a} \left[ \dot{Y}_{Tn}^{II} - \dot{Y}_{Tn}^{I} \right] M_{T} \, dx \]  \hspace{1cm} (8)

Where \( M_{T} \) is the mass of tie per unit length and \( \dot{Y}_{Tn} \) is the vertical velocity of tie \( n^{th} \) as a function of position \( x \).

\[ \left[ \dot{Y}_{W}^{II} - \dot{Y}_{W}^{I} \right] M_{W} = \sum_{n=1}^{5} \int_{-a}^{t+a} \left[ \dot{Y}_{Tn}^{II} - \dot{Y}_{Tn}^{I} \right] M_{T} \, dx + \int_{0}^{\lambda} \left[ \dot{Y}_{R}^{II} - \dot{Y}_{R}^{I} \right] M_{R} \, dz \]  \hspace{1cm} (9)

\[ \dot{Y}_{W}^{II} \cdot M_{W} = \sum_{n=1}^{5} \int_{-a}^{t+a} \dot{Y}_{Tn}^{II} \cdot M_{T} \, dx + \int_{0}^{\lambda} \dot{Y}_{R}^{II} \cdot M_{R} \, dz \]  \hspace{1cm} (10)

Simplifying yields:

\[ \dot{Y}_{W} \cdot M_{W} = \sum_{n=1}^{5} \int_{-a}^{t+a} \dot{Y}_{Tn} \cdot M_{T} \, dx + \int_{0}^{\lambda} \dot{Y}_{R} \cdot M_{R} \, dz \]  \hspace{1cm} (11)

The same impact period for all ties is assumed and only vertical linear motion is considered. Therefore, from (4), upon impact, 50 percent of the impulse force is on the center tie.
Figure 5.5 Assumed Deflected Shape of Rail After Impact

\[ \hat{P} = \int_{0}^{T} F(t) \, dt = \sum_{n=1}^{5} \int_{-a}^{t+a} \left[ \dot{\gamma}^{II}_{Tn} - \dot{\gamma}^{I}_{Tn} \right] M_{tie(n)} \, dx + \int_{0}^{\lambda} \left[ \dot{\gamma}^{II}_{R} - \dot{\gamma}^{I}_{R} \right] M_{R} \, dx \]  \hspace{1cm} (12)

but \[ \hat{R}_{n} = \int_{0}^{T} R_{n}(t) \, dt = \int_{0}^{t+a} \left[ \dot{\gamma}^{II}_{Tn} - \dot{\gamma}^{I}_{Tn} \right] M_{tie(n)} \, dx \]  \hspace{1cm} (13)

where \( \dot{\gamma}_{R} = \dot{\gamma}_{rail} \) and \( \dot{\gamma}_{Tn} = \dot{\gamma}_{tie(n)} \), where \( n = 1, 2, 3, 4, 5 \).

The equation can be simplified knowing that:
\( \dot{\gamma}_{Tn} = 0 \) for ties 1 to 5 and \( \dot{\gamma}_{R} = \dot{\gamma}_{W} = 0 \)
and: \( \hat{R} = (\dot{Y}_w^{II}) M_w \)

\[
\hat{R} = \sum_{n=1}^{5} \int_{-a}^{a} \dot{Y}_T^{II} M_T \, dx + 2 \int_{0}^{\lambda} \dot{Y}_R^{II} M_R \, dr
\]  

(14)

For the sake of clarity the superscript "II" can be omitted.

\[
\dot{Y}_w M_w = \sum_{n=1}^{5} \int_{-a}^{a} (\dot{Y}_T M_T) \, dx + 2 \int_{0}^{\lambda} (\dot{Y}_R M_R) \, dr
\]  

(15)

Note that for a given tie the summation is the same as that of the static condition:

\[
2 [\hat{R}_3 + 2 R_{2,4} + 2 R_{1,5}] = \sum_{n=1}^{5} \int_{-a}^{a} (\dot{Y}_T M_T) \, dx
\]  

(16)

Therefore:

\[
\dot{Y}_w M_w = 2[\hat{R}_3 + 2 R_{2,4} + 2 R_{1,5}] + 2 \int_{0}^{\lambda} (Y_R M_R) \, dr
\]  

(17)

From (9), for 132 lb. rail and 136 lb. rail, with tie spacings from 16" (406 mm) to 24" (610 mm), it can be determined that:

\[
\frac{R_2}{R_3} = \frac{R_4}{R_3} = \frac{36}{86.36} = 0.42
\]

so that \( R_2 = R_4 = 0.42 \, R_3 \)

\[
\frac{R_1}{R_3} = \frac{R_5}{R_3} = \frac{5.68}{86.36} = 0.06
\]

so that \( R_1 = R_5 = 0.6 \, R_3 \)

Therefore from the above:

\( \hat{R}_3 = 0.5 \, \hat{R} \)
\[ \hat{R}_2 = \hat{R}_4 = 0.42 \hat{R}_3 = 0.42(0.5) \hat{R} = 0.21 \hat{R} \]
\[ \hat{R}_1 = \hat{R}_5 = 0.06 \hat{R}_3 = 0.06(0.5) \hat{R} = 0.03 \hat{R} \]

Substituting these values into equation 17 we obtain:

\[ \dot{Y}_M N = 2 \hat{R}_3 \left[ 1 + 2(0.42) + 2(0.06) \right] + 2 \int_0^1 (Y_R M_R) \, dx \quad (18) \]

\[ \dot{Y}_M N = 2 \hat{R}_3 (1.96) + 2 \int_0^1 (Y_R M_R) \, dx \quad (19) \]

In general terms and in the case of tie number 3:

\[ 2 \hat{R}_3 = (Y_T M_T) \, dx \quad , \text{ for each tie including the central tie.} \]

![Diagram of assumed deflected shape of the tie in the first mode](image)

**Figure 5.6 Assumed Deflected Shape of the Tie in the First Mode**

The above diagram represents the deflected shape of the tie in the first mode. The frequency of the tie in the first
mode is 88 Hz (from field measurements). The first mode gives the largest impact deflection.

The assumed displacement is given by:

\[
Y_T = Y_{\text{max}} \sin\left(\frac{\pi x}{L}\right) \sin 2\pi F t
\]  

(20)

The velocity is given by:

\[
\dot{Y}_T = Y_{\text{max}} 2\pi F \sin\left(\frac{\pi x}{L}\right) \cos 2\pi F t
\]

(21)

at \( t = 0 \), the velocity is a maximum.

For conservation of momentum for a tie with zero initial velocity:

\[
\dot{R}_j = \int_{-a}^{l} \rho_c A Y_{\text{max}} 2\pi F \sin\left(\frac{\pi x}{L}\right) \cos (2\pi F t) \, dx
\]

(22)

Where:

\( \rho_c \) = mass density of concrete

\( A \) = cross-sectional area of the bridge tie

\( F \) = frequency of the tie in the first mode = 88 Hz

now let: \( Y_T = 2\pi F y_{\text{max}} \)

substituting into equation 22 yields.

\[
2 \dot{R}_j = \rho_c A Y_T \cos 2\pi F t \int_{-a}^{l} \sin\left(\frac{\pi x}{L}\right) \, dx
\]

(23)

\[
2 \dot{R}_j = \rho_c A Y_T (\cos 2\pi F t) \left( \frac{-L}{\pi} \cos \frac{\pi x}{L} \right) \Bigg|_{-a}^{l}
\]

(24)

evaluating by parts yields:

\[
-\frac{L}{\pi} \cos \frac{\pi x}{L} = -(21.6 + 21.6) = -43.21
\]
\[ \rho_c A = 0.029 \text{ slugs} \]

\[ 2 \dot{R}_3 = (0.029) (43.21) (\cos 2 \pi ft) \ddot{Y}_T \]

\[ 2 \dot{R}_3 = 1.25 \cos 2 \pi ft \ddot{Y}_T \]

at the point of impact \( t = 0 \)

therefore:

\[ 2 \dot{R}_3 = 1.25 \ddot{Y}_T, \]

so \( 2 \dot{R}_3 = 1.25 \ddot{Y}_T \) for the central tie.

To obtain the deflection at the rail seat, the following expression must be used:

\[ \dot{Y}_{TR} = \ddot{Y}_T \sin \frac{\pi b}{L} \] (at rail seat) \hspace{1cm} (25)

\[ 2 \dot{R}_3 = \frac{1.25 \dot{Y}_{TR}}{\sin \left( \frac{\pi b}{L} \right)} \text{ where: } \sin \left( \frac{\pi b}{L} \right) = 0.555 \]

\[ 2 \dot{R}_3 = 2.25 \dot{Y}_{TR} \] \hspace{1cm} (26)

\[ \dot{Y}_W M_W = 2.25 \dot{Y}_{TR} (1.96) + 2 \int_0^1 (\dot{Y}_R M_R) \, dr \]

\[ \dot{Y}_W M_W = 4.41 \dot{Y}_{TR} + 2 \int_0^1 (\dot{Y}_R M_R) \, dr \] \hspace{1cm} (27)

but according to kinematics (from equation 2 and 5)

\[ \dot{Y}_W = \dot{Y}_Y - \dot{Y}_{TR} \]

\[ \dot{Y}_W = \dot{Y}_Y - \dot{Y}_{TR} = r \omega \sin (\Theta_0 - \omega t) - \dot{Y}_{TR} \]

81
but from equation 4, \( \dot{y} = rw \sin (\Theta_o - \omega t) \)

at: \( t = 0 \), \( \dot{y} = rw \sin \Theta_o \), where maximum velocity occurs

\[ \dot{y}_w = rw \sin \Theta_o - \dot{y}_{TR} \quad (28) \]

\[ M_w \left[ rw \sin \Theta_o - \dot{y}_{TR} \right] = 4.41 \dot{y}_{TR} + 2 \int_0^\lambda \left( \dot{y}_R M_R \right) \, dr \quad (29) \]

where \( \Theta_o = 1/2 \) angle of wheel flat.

The apparent linear mass (\( M_w \)) involved in the impact is:

1) mass of wheel set, plus

2) 1/3 the mass of 2 side frames

\( M_w = 8.67 \) slugs (for average 100 ton car)

\[ \dot{y}_{TR} \times 13.0 = 8.67 \, rw \sin \Theta_o - 2 \int_0^\lambda \left( \dot{y}_R M_R \right) \, dr \quad (30) \]

Assuming that the rail and the ties deflect together at the rail seats, the following configuration is obtained:

---

**Figure 5.7 Assumed Displacement of Rail and Ties**

Therefore \( y_R = y_{TR} \)
note that:\[ \frac{\hat{R}_n}{\hat{R}_3} = \frac{\int_a^b \hat{Y}_{TN} M_r \, dx}{\int_a^b \hat{Y}_{TR} M_r \, dx} \] (31)

with \[ \hat{Y}_{TN} (x,t) = Y_{\text{max}} (2\pi f) \sin \frac{\pi x}{l} \cos 2\pi ft, \] (32)

where \( f = 88 \text{ Hz} \).

Substituting yields:

\[ \frac{\hat{R}_n}{\hat{R}_3} = \frac{Y_{\text{max}} (2\pi fT) \cos 2\pi ft \int_a^b \sin \frac{\pi x}{l} \, dx}{Y_{\text{max}, (2\pi f) \cos 2\pi ft} \int_a^b \sin \frac{\pi x}{l} \, dx} = \frac{\hat{Y}_{TN}}{\hat{Y}_{TR}} \] (33)

but \( \hat{Y}_{TR} = \hat{Y}_{TN} \sin \frac{\pi b}{l} \)

so \[ \frac{\hat{R}_n}{\hat{R}_3} = \frac{\bar{Y}_T}{\bar{Y}_{TR}} = \frac{\hat{Y}_R (16,4)}{\hat{Y}_R (48)} \] (34)

Now, assuming the deflected rail shape is compatible with the deflection of the rail seat on the tie, the deflection of the rail is established as:
\[ \bar{Y}_{TR} = \hat{Y}_R (16, n) \]

where the tie spacing = 16" (406 mm)
\[ Y_R = Y_{TR} F_{(r)} \sin 2\pi ft, \] (35)

where \( F_{(r)} \) is a function of the vertical deflection along the track
\[ \hat{Y}_R = Y_{TR} (2\pi f) F_{(r)} \cos 2\pi ft \] (36)
\[ \dot{y}_R = \bar{y}_R \sin \frac{bn}{\ell} F_{(x)} \cos 2\pi ft \]  

(37)

Therefore:

\[ \frac{\dot{y}_R (16 \times n)}{y_R (48)} = \frac{F(16 \times n)}{F(48)} \]

The function of \( F_{(r)} \) must be chosen such that it is compatible with the tie displacement at the rail seat:

Set \( F_{(x)} = \sin^6 \frac{\pi X}{\lambda} \) where \( \lambda = 96'' (2.44m) \)

The following is a table for the ratios:

\[ \frac{\dot{R}_n}{\dot{R}_3} \text{ and } \frac{F(16 \times n)}{F(48)} \]

<table>
<thead>
<tr>
<th>( n )</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>( r )</td>
<td>16</td>
<td>32</td>
<td>48</td>
</tr>
<tr>
<td>( \dot{R}_n/\dot{R}_3 )</td>
<td>0.06</td>
<td>0.42</td>
<td>1.00</td>
</tr>
<tr>
<td>( F_n/F_3 )</td>
<td>0.02</td>
<td>0.42</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Table 5.1 Summary of Ratios \( R_n/\dot{R}_3 \) and \( F(16 \times n)/F(48) \)

From the above table it can be seen that the chosen function \( F_{(r)} = \sin^6 \frac{\pi X}{\lambda} \) is appropriate.

Therefore:

\[ \dot{y}_R = \bar{y}_R \sin \frac{bn}{\ell} \sin^6 \frac{\pi X}{\lambda} \cos 2\pi ft \]  

(38)

at \( t = 0 \) \[ \dot{y}_R = \bar{y}_R \sin^6 \frac{\pi X}{\lambda} \]  

(39)
\[ \int_0^1 \sin^6 \frac{\pi x}{\lambda} \, dx \]  

where \( \lambda = 96" \) (2.44 m)

Solving yields

\[ \int_0^1 \sin^6 \frac{\pi x}{\lambda} = 30 \]  

(41)

Substituting into the left hand side of equation 30 yields:

\[ \int_0^1 \dot{y}_R M_R \, dr = 30 M_R \ddot{y}_T \sin \frac{b\pi}{l} \cos 2\pi ft \]  

(42)

\[ \int_0^1 \dot{y}_R M_R \, dr = 30 M_R \dot{y}_T \cos 2\pi ft \]  

(43)

Therefore:

\[ \int_0^1 M_R \ddot{y}_T \sin \frac{b\pi}{l} \sin^6 \frac{\pi x}{\lambda} \cos 2\pi ft \, dx \]  

(44)

\[ = 30 M_R \dot{y}_T \sin \frac{b\pi}{l} \cos 2\pi ft \]  

(45)

\[ = 30 M_R \dot{y}_T \cos 2\pi ft \]  

(46)

\[ M_R = \rho_R A_R = (0.283 \text{ lbs/in}^3 \times 12.91 \text{ in}^2) / 386.4 \]  

(47)

\[ M_R = 0.0095 \text{ slugs} \quad \text{say} \quad M_T = 0.01 \text{ slugs} \]

85
\[
\int_0^l \dot{Y}_R M_R \, dr = 30.0 \times 0.01 \times \dot{Y}_{TR} \times \cos 2\pi F t
\]  
(48)

At impact, the maximum momentum occurs at \( t = 0 \) therefore,

\( 30 \times 0.01 \times \dot{Y}_{TR} \),

therefore, from equation 30

\[
\dot{Y}_{TR} \times 13.0 = 8.67 \, r \, \omega \sin \Theta_0 - 2 \int_0^l \dot{Y}_R M_R \, dr
\]  
(49)

we now obtain

\[
\dot{Y}_{TR} \times (13.0 + 0.60) = 8.67 \, r \, \omega \sin \Theta_0
\]  
(50)

assuming 36" (914 mm) Heuman wheels, \( r = 18" \) (457 mm)
\( \sin \Theta_0 = 0.028 \) for a 1" (25.4 mm) skid flat
\( \omega = V/r \)

\[
13.60 \, \dot{Y}_{TR} = 8.67 \times 18 \times V \sin \Theta_0
\]  
(51)

Assuming a freight train velocity of 40 mph (66 km/h), which
was the average speed across the bridge (all monitored
trains)

\[
13.60 \, \dot{Y}_{TR} = 8.67 \times 706 \times 0.028 = 171.39
\]  
(52)

\[
\dot{Y}_{TR} = 12.60 \text{ in/sec (0.23 m/sec)}
\]

Knowing the maximum tie velocity from the wheel impact, a
rail seat and therefore the tie center, the maximum strain
at tie center can be computed from the assumed deflected
shape. The vertical velocity along the tie is expressed as:

\[
\dot{Y}_T(x, t) = \overline{Y} \frac{\sin \pi x}{\ell} \cos 2\pi F t
\]  
(53)

\[\text{Where } \overline{Y} \sin \frac{\pi b}{\ell} = \dot{Y}_{TR} \text{ and } \overline{Y} = Y_{max} \cdot 2\pi F\]
To express maximum strain at the tie centre, set $30 = \ell/2$

and $t = 0$ (immediately)

\[
\text{Therefore } \dot{Y} = \frac{\dot{Y}_{TR}}{\sin \frac{\pi \beta}{\ell}} = \frac{12.60}{\sin \frac{\pi 18}{96}}
\]

\[\dot{Y} = 22.68 \text{ in/sec (0.58 m/sec) at tie centre}\]

Maximum displacement $Y_{\text{max}}$ at the tie center

\[Y_{\text{max}} = \frac{Y}{2\pi F} = \frac{22.68}{2\pi 85} = 0.042'' (1.1 \text{ mm}) \text{ at tie centre.}\]  

(55)

The expression of vertical tie displacement along its length is expressed as:

\[Y_T(x,t) = Y_{\text{max}} \sin \left(\frac{\pi x}{\ell}\right) \sin 2\pi Ft\]

(56)

To express the bending strain in terms of vertical displacement of the beam.

\[\sigma = \frac{MY}{I} \text{ or } \epsilon = \frac{MY}{EI}, \text{ also } M = EI \frac{d^2 Y_T}{dx^2}\]

\[\epsilon = Y \frac{d^2 Y_T}{dx^2}\]

(57)

\[\epsilon_{\text{max}} \text{ at the tie centre:}\]

\[\epsilon_{\text{max}} = \frac{d^2}{dx^2} \left[ Y_{\text{max}} \sin \frac{\pi x}{\ell} \sin 2\pi Ft \right]\]

(58)

where $\epsilon_{\text{max}}$ is taken at maximum deflection of the tie from impact.

At maximum deflection of tie, the time component "sin 2\pi Ft" of the assumed mode expression is:

\[t = \frac{1}{4f}\]
Therefore:
\[ \sin 2\pi ft = 1 \]

so
\[ \varepsilon_{\text{max}} = Y_b Y_{\text{max}} \frac{d^2}{dx^2} \left[ \sin \frac{\pi x}{L} \right] = Y_b Y_{\text{max}} \left( \frac{\pi}{L} \right)^2 \sin \frac{\pi x}{L} \] (59)

for \( x = \frac{L}{2} \)

\[ \varepsilon_{\text{max}} = Y_b Y_{\text{max}} \left( \frac{\pi}{L} \right)^2, \text{ where } Y_{\text{max}} = 0.042'' (1.1 \text{mm}) \]

Considering the bottom fibre tensile strain, for positive bending:

\[ Y_{\text{max}} = 0.042'' (1.1 \text{ mm}) \]
\[ L = 96'' (2.44 \text{ m}) \]
\[ Y_b = 5.84'' (148.3 \text{ mm}) \]

\[ \varepsilon = Y_b Y_{\text{max}} \left( \frac{\pi}{L} \right)^2 \]

\[ \varepsilon_{\text{max}} = 5.84 \times 0.042 \left( \frac{\pi}{96} \right)^2 \] (60)

\[ \varepsilon_{\text{max}} = 262 \times 10^{-6} \text{ in./in. dynamic strain} \]

The static strain must now be added to this component such that:

\[ \varepsilon_{\text{max}} = \varepsilon_{\text{max}} \text{ (dynamic) + } \varepsilon_{\text{max}} \text{ (static)} \]

\[ \varepsilon_{\text{max}} \text{ (static)} = 153 \times 10^{-6} \text{ in./in.} \]

\[ \varepsilon_{\text{max}} = (262 \times 10^{-6}) + (153 \times 10^{-6}) \]

\[ \varepsilon_{\text{max}} = 415 \times 10^{-6} \text{ in./in.} \]
5.3 MATHEMATICAL MODEL OF THE CONCRETE BRIDGE TIE RESPONSE IN NEGATIVE BENDING

Using the same basic assumptions as that of the preceding mathematical model for the tie response in the positive bending, the negative bending response has been developed. This method has been based on the conservation of energy and is considered to occur in the first flexural mode.

From the energy method:

\[ \sigma = \frac{MY}{I} \quad \text{and} \quad \sigma = \varepsilon E \]

Therefore \( \varepsilon E = \frac{MY}{I} \)

and \( \varepsilon = \frac{MY}{EI} \); \( \frac{M}{EI} = \frac{\varepsilon}{Y} \)

but \( \frac{M}{EI} = \frac{-d^2Y}{dx^2} \) and \( \frac{\varepsilon}{Y} = \frac{-d^2y}{dx^2} \)

The assumed mode of deflection is given by:

\[ Y = Y_0 \sin \left( \frac{\pi x}{L} \right) \] (1)

\[ \frac{d^2Y}{dx^2} = -\left( \frac{\pi}{L} \right)^2 Y_0 \sin \left( \frac{\pi x}{L} \right) \] (2)

Now setting \( \left( \frac{\pi}{L} \right)^2 Y_0 = A \)

Yields \( \frac{d^2Y}{dx^2} = -A \sin \left( \frac{\pi x}{L} \right) \) (3)
Figure 5.8 Bending Moment Diagram for Tie

From the above bending moment diagram

\[ a = 24" \text{ (610 mm)} \]
\[ b = 18" \text{ (457 mm)} \]
\[ \ell = 96" \text{ (2.44 m)} \]

From the energy method

\[ u = \int \frac{M^2(x) \, dx}{2EI} \]  

\[ U_D = U_U + W\gamma_o' + 1/2 \, k \left( \frac{\gamma_o'}{2} \right)^2 \]

Where:

\( U_o \) = energy stored in the tie due to the downward deflection.

\( U_U \) = energy stored in the tie due to the upward deflection.

\( W \) = wheel load.

\( \gamma_o' \) = upward deflection of tie.

\( k \) = spring constant for railway vehicles

Evaluating each term separately yields:
\[ U_p = \int_{-a}^{a} \frac{EI}{2} \left( \frac{M}{EI} \right)^2 \, dx \]  

\[ = \int_{-a}^{a} \frac{EI}{2} \left[ A \sin \left( \frac{\pi x}{L} \right) \right]^2 \, dx \]

\[ = \int_{-a}^{a} \frac{EIA^2}{2} \sin^2 \left( \frac{\pi x}{L} \right) \, dx \]

\[ = \frac{EIA^2}{2} \int_{-a}^{a} \sin^2 \left( \frac{\pi x}{L} \right) \, dx \]

Similarly:

\[ U_U = \int_{-a}^{a} \frac{EI}{2} \left( \frac{M}{EI} \right)^2 \, dx \]  

\[ = \int_{-a}^{a} \frac{EI}{2} \left[ B \sin \left( \frac{\pi x}{\ell} \right) \right]^2 \, dx \]

\[ = \int_{-a}^{a} \frac{EIB^2}{2} \sin^2 \left( \frac{\pi x}{\ell} \right) \, dx \]

\[ = \frac{EIB^2}{2} \int_{-a}^{a} \sin^2 \left( \frac{\pi x}{\ell} \right) \, dx \]  

\[ \text{Where: } B = \left( \frac{\pi}{\ell} \right)^2 Y_0' \]

\[ WY = 2 WY_0' \sin \left( \frac{bn}{\ell} \right) \]
\[ \text{and } 1/2 \ K \ Y_s^2 = 1/2 \ K \left( B \frac{L^2}{2 \pi^2} \right)^2 \]  

\[ 1/2 \ K \ Y_s^2 = 1/2 \ K B^2 \left( \frac{L}{2 \pi} \right)^2 \]

Where \( Y_s \) = Compression of the vehicle springs. Taking the above terms and evaluating yields:

\[ U_b = \frac{EIA^2}{2} \left[ \left( \frac{1}{2} \ (120) - \frac{96}{4\pi} \sin \left( \frac{240\pi}{96} \right) \right) - \left( \frac{1}{2} \ (-24) - \frac{96}{\pi} \sin \left( \frac{-48\pi}{96} \right) \right) \right] \]

\[ \left[ \frac{1}{2} \ (-24) - \frac{96}{\pi} \sin \left( \frac{-48\pi}{96} \right) \right] \]

\[ U_b = \frac{EIA^2}{2} \left[ \left( 60 - \frac{24}{\pi} \sin 2.5\pi \right) - \left( -12 - \frac{96}{\pi} \sin \left( \frac{-\pi}{2} \right) \right) \right] \]

\[ U_b = \frac{EIA^2}{2} \left[ \left( 60 - \frac{24}{\pi} \sin 7.854 \right) - \left( -12 - \frac{24}{\pi} \sin -1.571 \right) \right] \]

\[ U_b = \frac{EIA^2}{2} \left[ \left( 60 - 7.6394 \right) - (-12 + 7.6394) \right] \]

\[ U_b = \frac{EIA^2}{2} \left[ 52.3606 + 4.3606 \right] \]

\[ U_b = \frac{EIA^2}{2} (56.7212) \]

Similarly:

\[ U_u = \frac{EIA^2}{2} (56.7212) \]

\[ 2 \ WY_o' \sin \left( \frac{bn}{L} \right) = 2 \ WY_o' \sin \left( \frac{18\pi}{96} \right) \]
\[ = 2 \omega Y_0' \left( 0.5336 \right) \]  \hspace{1cm} (20)

but \( Y_0' = B \left( \frac{l}{\pi} \right)^2 \)  \hspace{1cm} (21)

Therefore, substituting equation 21 into equation 10 yields:
\[ 2 W B \left( \frac{l}{\pi} \right)^2 \sin \left( \frac{b \pi}{l} \right) \]  \hspace{1cm} (22)

Where \( W \) = Weight of wheel loading. Normal 100 ton (890 kN) car vehicle bogie (truck) weighs 8,000 lbs. (35.6 kN).

Under impact the central tie receives 50 percent of the load. Only 1 wheel set is being considered.

So: \( W = 8,000 \times \frac{1}{2} \times \frac{1}{2} = 2,000 \text{ lbs. (8.9 kN)} \)

Therefore:
\[ 2 \omega Y_0' \sin \left( \frac{b \pi}{l} \right) = 2 \left( \frac{2,000}{2} \right) B \left( \frac{96}{\pi} \right)^2 \sin \left( \frac{18 \pi}{96} \right) \]  \hspace{1cm} (23)

\[ 2 \omega Y_0' \sin \left( \frac{b \pi}{l} \right) = 2.075 \times 10^6 B \]  \hspace{1cm} (24)

The term \( \frac{1}{2} k \left( Y_s' \right)^2 \) considers the compression of the bogie (truck) springs after the ties have been impacted by the rebounding tie.

\( k = 37,000 \text{ lbs/in (6,479 N/mm)} \)

\( Y_0' = 2 Y_s \) where \( Y_s \) is the compression of the springs.
\[ \frac{1}{2} K \left( \frac{Y_0'}{2} \right)^2 = \frac{1}{8} K Y_0' \]  \hspace{1cm} (25)

but \( Y_0' = B \left( \frac{l}{\pi} \right)^2 \)  \hspace{1cm} (26)
therefore \( Y_0'^2 = B^2 \left( \frac{\ell}{\pi} \right)^4 \) \hspace{1cm} (27)

Substituting equation 27 into 25 and evaluating yields:

\[
\frac{1}{8} k B^2 \left( \frac{\ell}{\pi} \right)^4 = \frac{1}{8} \times 37,000 \times \left( \frac{96}{\pi} \right)^4 B^2
\]

\[
= 4,032.711 \times 10^6 B^2
\]

\[
A = \frac{\varepsilon}{Y \sin \left( \frac{\pi x}{l} \right)} \quad \text{where} \quad x = \frac{l}{2}
\]

\[
(29)
\]

Therefore: \( A = \frac{\varepsilon}{Y} \) when \( Y = 5.84'' \) (148 mm)

The maximum positive strain (from the model for the ties response in positive bending) is \( 262 \times 10^{-6} \).

Therefore \( A = \frac{\varepsilon}{Y} = \frac{262 \times 10^{-6}}{5.84} \)

\[
A = 44.86 \times 10^{-6}.
\]

Substituting this value for A into the equation for \( U_b \) and evaluating for B yields:

\[
U_b = \frac{(4.15 \times 10^6) (1560) (44.86 \times 10^{-6})^2 (56.7212)}{2}
\]

\[
U_b = 369.5
\]

Similarly:

\[
U_v = \frac{(4.15 \times 10^6) (1,560) (56.7212) B^2}{2}
\]

\[
U_v = 1.8361 \times 10^{11} B^2
\]

\[
1/2 K \left( \frac{Y_0'}{2} \right)^2 = 4,032.71 \times 10^6 B^2
\]

(35)
\[ 2 \bar{W}B Y_0' \sin \left( \frac{bt}{l} \right) = 2.075 \times 10^6 B \]  

Therefore:

\[ 369.5 = 1.8361 \times 10^{11} B^2 + 4,032.71 \times 10^6 B^2 + 2.075 \times 10^6 B \]

\[ 369.5 = 1.87639 \times 10^{11} B^2 + 2.075 \times 10^6 B \]  

(37)

Using the quadratic equation and solving yields.

\[ B = 41.5 \times 10^{-6} \text{ in.}^{-1} \]  

(38)

but \( B = \frac{\varepsilon}{Y_T} \) \( \text{where } Y_T = 6.16" \text{ (157 mm)} \)

\[ \varepsilon = B Y \lambda = (42 \times 10^{-6}) (6.16) \]  

(39)

\[ \varepsilon = 259 \times 10^{-6} \text{ in./in.} \]

The maximum static strain \( = 153 \times 10^{-6} \). Therefore the dynamic strain in negative bending is:

\[ (259 \times 10^{-6}) - (153 \times 10^{-6}) = 106 \times 10^{-6}. \text{ in./in.} \]  

(40)

### 5.4 Discussion on the Models

From the above models the strain levels can be determined in the tie for both positive and negative bending.

The model for the positive bending tie response predicts the maximum positive strain (static and dynamic) for the bottom fibre of the tie.

The tie can be designed using the strain levels as:

\[ \sigma_b = \varepsilon_b E_c \text{ (for positive bending)} \]

\[ \sigma_t = \varepsilon_t E_c \text{ (for negative bending)} \]
CHAPTER 6
DESIGN OF THE CONCORDIA TEST TIES

6.1 CONVERSION OF MODEL STRAINS TO STRESSES

Using the mathematical model of the preceding chapter, full scale ties for experimental laboratory testing were designed. Three separate tie designs using the original C.N.R. tie envelope were undertaken.

The design of the ties is based on using the maximum strains calculated in the previous chapter and converting them to design stress levels at the outer fibres of the tie. The maximum strain (combined static and dynamic) at the bottom of the tie due to positive bending is $415 \times 10^{-6}$.

The corresponding stress level can be calculated from:

$\sigma_b = \epsilon_b \ E_c$ and $\sigma_t = \epsilon_t \ E_c$

Where

$E_c = 57,000 \ \sqrt{f_c'}$

and $f_c'$ is specified as 6,000 psi (41.4 MPa)

Therefore: $E_c = 4.42 \times 10^6$ Psi (3,048 MPa)

$\sigma_b = 4.42 \times 10^6 \times 4.15 \times 10^{-6}$

$\sigma_b = 1,835$ psi (12.7 MPa)

Use 1,850 psi (12.8 MPa) compression

The tie is subject to negative bending (reverse bending) caused by dynamic impact. Therefore, the tie does not feel the effects of static loading when subjected to negative
bending and the top fibre stress level can be calculated from the maximum dynamic strain value.

\[ \sigma_t = \varepsilon_t E_c \]

\[ \sigma_t = 106 \times 10^{-6} \times 4.42 \times 10^6 \]

\[ \sigma_t = 469 \text{ psi (3.3 MPa)} \]

Use \( \sigma_t = 500 \text{ psi (3.4 MPa)} \) compression

It was decided to design two tie types with eccentric prestress (Type 1 and Type 2) and one type with uniform prestress (Type 3). For the Type 2 tie an arbitrary value of \( \sigma_t = 1,000 \text{ psi (6.90 Mpa)} \) was chosen because \( \sigma_t = \frac{1}{2} \sigma_b \).

Summarizing the stress value for each type of tie:

<table>
<thead>
<tr>
<th>Type 1</th>
<th>Type 2</th>
<th>Type 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \sigma_t )</td>
<td>500 psi</td>
<td>1,000 psi</td>
</tr>
<tr>
<td>( \sigma_t )</td>
<td>3.4 MPa</td>
<td>6.9 MPa</td>
</tr>
<tr>
<td>( \sigma_p )</td>
<td>1,850 psi</td>
<td>1,850 psi</td>
</tr>
<tr>
<td>( \sigma_p )</td>
<td>12.8 MPa</td>
<td>12.8 MPa</td>
</tr>
</tbody>
</table>

In practice, it is best to design the tie such that when it is subjected to maximum service loading, the concrete within the tie remains in compression even though the concrete can carry tension, usually limited to approximately 0.1 \( f_c' \) in uniaxial tension.

As the original C.N.R. tie envelope was maintained, the
properties of the tie can now be summarized.

\[ A = 130.28 \text{ in.}^2 (84,057 \text{ mm}^2) \]

\[ I = 1,560 \text{ in.}^4 (6.49 \times 10^6 \text{ mm}^4) \]

\[ Y_t = 6.16 \text{ in.} (156.6 \text{ mm}) \]

\[ Y_b = 5.84 \text{ in.} (148.3 \text{ mm}) \]

6.2 DESIGN OF THE TYPE 1 TIE

By knowing the desired design stresses \( \sigma_t = 500 \text{ psi} (3.4 \text{ MPa}) \) and \( \sigma_b = 1,850 \text{ psi} (12.8 \text{ MPa}) \), and by using the following well known formulas (3, 5):

\[ \sigma_t = \frac{P}{A} - \frac{P e Y_t}{I} \]

\[ \sigma_b = \frac{P}{A} + \frac{P e Y_b}{I} \]

the required prestressing force, \( P \), and the eccentricity, \( e \), can be determined. Substituting the above values into the two equations and solving yields:

\( P = 155,420 \text{ lbs.} \) (691.3 KN)

Back-substitution yields:

\( e = 1.13'' \) (28.7 mm)

An assumed loss in prestressing force due to relaxation of wires, creep and shrink of concrete, was taken as 27 percent. This value is based on the C.N.R. experience with concrete track ties, whose values ranged from 25 to 30 percent.

\[ \text{Therefore } P_i = \frac{P}{1 - 0.27} = \frac{P}{0.73} \]
\[ P_i = \frac{155,420}{0.73} = 212,900 \text{ lbs (947 kN)} \]

The use of 3/8" (9.55 mm) diameter prestressing strand with a tensile strength, \( f_{pu} \), of 270 ksi (1,860 MPa) was selected.

The normal range of prestressing force initially applied to each strand is in the order of 0.7 \( f_{pu} A_s \) to 0.8 \( f_{pu} A_s \). The value of 0.7 \( f_{pu} A_s = 16,100 \text{ lbs (71.6 kN)} \) is chosen as the initial prestressing force.

Therefore the number of strands required is:
212,900 / 16,100 = 13.22 strands.

Use 14 - 3/8" (9.55 mm) diameter strands. Therefore, the initial prestressing force applied to each strand is:
212,900 / 14 = 15,210 lbs (67.6 kN)
15,210 lbs (67.6 kN) = 0.66 \( f_{pu} A_s \)

The final prestressing force after losses in each strand is:
15,210 x 0.73 = 11,103 lbs (49.4 kN)

Through a trial and error process, the arrangement of 14 strands as shown in Figure 6.1, was chosen.

6.3 DESIGN OF THE TYPE 2 AND TYPE 3 TIES
The Type 2 and Type 3 test ties were designed using the same general procedure as for the Type 1 ties. Table 6.1 indicates
the tabulated values for all 3 tie designs. Again a trial and error process is used in order to arrive at an acceptable arrangement of prestressing strands. The Type 2 and Type 3 ties are shown in Figures 6.2 and 6.3 respectively.

Figure 6.4 indicates the comparison of the various tie cross sections. A comparison of the effective prestress levels in these various tie designs is shown in Figure 6.5.

6.4 ULTIMATE MOMENT CAPACITY CHECK OF THE TYPE 1 TIE

In order to check the ultimate moment capacity of the ties, the stress - strain compatibility method has been used. This method more accurately predicts the ultimate moment capacity of the tie. This was useful in determining the required strength of the load frame and capacity of the jacks for laboratory testing.
TIE TYPE No.1  (ECCENTRIC PRESTRESS)

1) 14 x 3/8" (9.5mm) DIA. - 7 WIRE STRAND WITH PRESTRESSING FORCE OF 15,210 lbs. (67.7 kN) EACH.
2) 15,210 lbs. (67.7 kN) = 0.66 fpu As
3) TOTAL PRESTRESSING FORCE = 212,940 lbs. (947.2 kN).
4) EFFECTIVE PRESTRESSING FORCE = 155,445 lbs. (691.4 kN).
5) STRESSES IN CONCRETE AFTER TRANSFER:
   \[ f_t = 500 \text{ psi (3.4 MPa) COMPRESSION} \]
   \[ f_s = 1850 \text{ psi (12.8 MPa) COMPRESSION} \]
6) ECCENTRICITY (e) = 1.13 in. (28.7mm)

Figure 6.1 Strand Arrangement of the Type 1 Tie
TIE TYPE No. 2 (ECCENTRIC PRESTRESS)

1) 16 x 3/8" DIA. (9.5mm) - 7 WIRE STRAND WITH PRESTRESSING FORCE OF 16,021 lbs. (71.3 kN) EACH.
2) 16,021 lbs. (71.3 kN) = 0.7 fpu As
3) TOTAL PRESTRESSING FORCE = 256,337 lbs. (1140.2 kN).
4) EFFECTIVE PRESTRESSING FORCE = 187,126 lbs. (832.3 kN).
5) STRESSES IN CONCRETE AFTER TRANSFER:
   \[ f_t = 1000 \text{ psi (6.9 MPa)} \text{ COMPRESSION} \]
   \[ f_b = 1850 \text{ psi (12.8 MPa)} \text{ COMPRESSION} \]
6) ECCENTRICITY \( (e) = 0.59 \text{ in. (15.0mm)} \)

Figure 6.2 Strand Arrangement of the Type 2 Tie
TIE TYPE No. 3  (UNIFORM PRESTRESS)

1) 20 x 3/8" DIA. (9.5mm) - 7 WIRE STRAND WITH PRESTRESSING FORCE OF 16,510 lbs. (73.4 kN) EACH.
2) 16,510 lbs. (73.4 kN) = 0.72 fpu As
3) TOTAL PRESTRESSING FORCE = 330,162 lbs. (1468.6 kN).
4) EFFECTIVE PRESTRESSING FORCE = 241,046 lbs. (1072.7 kN).
5) STRESSES IN CONCRETE AFTER TRANSFER:
   \[ f_e = 1850 \text{ psi} (12.8 \text{ MPa}) \text{ COMPRESSION} \]
   \[ f_b = 1850 \text{ psi} (12.8 \text{ MPa}) \text{ COMPRESSION} \]
6) ECCENTRICITY \( (e) = 0 \)

Figure 6.3 Strand Arrangement of the Type 3 Tie

103
<table>
<thead>
<tr>
<th>TYPE</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>130.28 in²</td>
<td>130.28 in²</td>
<td>130.28 in²</td>
</tr>
<tr>
<td></td>
<td>84,057 mm²</td>
<td>84,057 mm²</td>
<td>84,057 mm²</td>
</tr>
<tr>
<td>I</td>
<td>1560 in⁴</td>
<td>1560 in⁴</td>
<td>1560 in⁴</td>
</tr>
<tr>
<td></td>
<td>6.49 x 10⁻⁴m⁴</td>
<td>6.49 x 10⁻⁴m⁴</td>
<td>6.49 x 10⁻⁴m⁴</td>
</tr>
<tr>
<td>Yₜ</td>
<td>6.16 in</td>
<td>6.16 in</td>
<td>6.16 in</td>
</tr>
<tr>
<td></td>
<td>156.5 mm</td>
<td>156.5 mm</td>
<td>156.5 mm</td>
</tr>
<tr>
<td>Yᵦ</td>
<td>5.84 in</td>
<td>5.84 in</td>
<td>5.84 in</td>
</tr>
<tr>
<td></td>
<td>148.3 mm</td>
<td>148.3 mm</td>
<td>148.3 mm</td>
</tr>
<tr>
<td>NO. STRANDS</td>
<td>14</td>
<td>16</td>
<td>20</td>
</tr>
<tr>
<td>% LOSSES</td>
<td>27</td>
<td>27</td>
<td>27</td>
</tr>
<tr>
<td>σₜ</td>
<td>500 psi</td>
<td>1,000 psi</td>
<td>1,850 psi</td>
</tr>
<tr>
<td></td>
<td>3.45 MPa</td>
<td>690 MPa</td>
<td>12.75 MPa</td>
</tr>
<tr>
<td>σᵦ</td>
<td>1,850 psi</td>
<td>1,850 psi</td>
<td>1,850 psi</td>
</tr>
<tr>
<td></td>
<td>12.75 MPa</td>
<td>12.75 MPa</td>
<td>12.75 MPa</td>
</tr>
<tr>
<td>e</td>
<td>1.13 in</td>
<td>0.59 in</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>28.70 mm</td>
<td>14.99 mm</td>
<td>0</td>
</tr>
<tr>
<td>Pᵦ</td>
<td>212,940 lbs</td>
<td>256,337 lbs</td>
<td>330,162 lbs</td>
</tr>
<tr>
<td></td>
<td>947.2 kN</td>
<td>1,140 kN</td>
<td>1,469 kN</td>
</tr>
<tr>
<td>Pₑₑₑ</td>
<td>155,440 lbs</td>
<td>187,126 lbs</td>
<td>241,018 lbs</td>
</tr>
<tr>
<td></td>
<td>691.4 kN</td>
<td>832 kN</td>
<td>1,072 kN</td>
</tr>
</tbody>
</table>

Table 6.1 Characteristics of Tie Designs
Figure 6.4 Comparison of Various Tie Cross Sections
Figure 6.5 Comparison of Effective Prestress Levels in Various Ties
Figure 6.6 Stress - Strain Compatibility Diagram

From the above diagram the following is obtained:

\[ f_{se} = \text{effective prestress in steel after losses} \]

\[ f_{se} = \frac{155,440}{14} \times 0.085 = 130,622 \text{ psi (900.6 MPa)} \]

\[ \varepsilon_{se} = \frac{f_{se}}{E_s} = \frac{130,622}{29,000,000} = 0.0045 \]

\[ \varepsilon_{su} = \frac{f_{pu}}{E_s} = \frac{270,000}{29,000,000} = 0.0093 \]

From the placement of strand within the tie the following can be determined:

\[ d_1 = 9'' \ (229 \text{ mm)}, \ d_2 = 7'' \ (178 \text{ mm)} \text{ and } d_3 = 3'' \ (76 \text{ mm)} \]

Assume the balanced case where concrete ruptures as the steel yields, and solve for \( X_{\text{max}} \), where:
\[ a = 0.80 \times_{\text{max}} \]

\[ X_{\text{max}} = \frac{e_{\text{cu}} \times d_1}{e_1 + e_{\text{cu}}} \]

Where: \( \epsilon_1 = \epsilon_{\text{psi}} - \epsilon_{\text{se}} = 0.0093 - 0.0045 = 0.0048 \)

and \( \epsilon_{\text{cu}} = 0.003 \)

\[ X_{\text{max}} = \frac{(0.003)(9)}{0.0048 + 0.003} \]

\[ X_{\text{max}} = 3.46" \text{ (87.9 mm)} \]

Now assume that the steel fails first:

\[ \epsilon_2 = \frac{e_2 (d_2 - x)}{(d_1 - x)} = 0.0048 \times \frac{(7 - x)}{(9 - x)} \]

\[ \epsilon_3 = \frac{-e_1 (d_3 - x)}{(d_1 - x)} = -0.0048 \times \frac{(3 - x)}{(9 - x)} \]

\[ T_1 = A_{e_1} f_{pu} = (6 \times 0.085) \times 270,000 = 137,700 \text{ lbs (612.5 kN)} \]

\[ T_2 = A_{e_2} f_{pu} (e_2/e_1) = 6 \times 0.085 \times 270,000 \times \left(\frac{7 - x}{9 - x}\right) \]

\[ T_2 = 137,700 \times \left(\frac{7 - x}{9 - x}\right) \]

\[ C_1 = 0.085 f' c \times a \times b \]

set \( a = 0.8x \) and \( b = 10" \text{ (25 mm)} \)

\[ C_1 = 0.85 \times 6,000 \times (0.8x) \times 10 \]

\[ C_1 = 40,800 \times x \]
\[ C_2 = A_{s_3} \left( -\varepsilon_3/\varepsilon_1 \right) = -2 \left( 0.085 \right) 270,000 \left( \frac{3 - x}{9 - x} \right) \]

\[ C_2 = -45,900 \left( \frac{3 - x}{9 - x} \right) \]

Collecting terms yields:

\[ T_1 + T_2 = C_1 + C_2 \]

Therefore:

\[ 137,700 + 137,700 \left( \frac{7 - x}{9 - x} \right) = 40,800x - 45,900 \left( \frac{3 - x}{9 - x} \right) \]

Solving the above by use of the quadratic equation yields:

\[ x = 4.29" \ (109.0 \ mm) \]

but \( x > X_{\text{max}} \) (ie \( 4.29" > 3.46" \))

Therefore use \( x = X_{\text{max}} = 3.46" \ (87.9 \ mm) \)

Now substitute \( X_{\text{max}} = 3.46 \ (87.9 \ mm) \) to obtain:

\[ \varepsilon_2 = \varepsilon_1 \left( \frac{d_2 - X_{\text{max}}}{d_1 - X_{\text{max}}} \right) - \varepsilon_1 \left( \frac{7 - 3.46}{9 - 3.46} \right) \]

\[ \varepsilon_2 = 0.639 \varepsilon_1 \]

\[ \varepsilon_3 = \varepsilon_1 \left( \frac{d_3 - X_{\text{max}}}{d_1 - X_{\text{max}}} \right) - \varepsilon_1 \left( \frac{3 - 3.46}{9 - 3.46} \right) \]

\[ \varepsilon_3 = -0.083 \varepsilon_1 \]

Solving for \( T_1, T_2, C_1 \) and \( C_2 \) in terms of \( f_{s_1} \) yields:

\[ T_1 = A_{s_1} f_{s_1} = 6 \ (0.085) f_{s_1} = 0.510 f_{s_1} \]

\[ T_2 = A_{s_2} f_{s_1} \left( \varepsilon_2/\varepsilon_1 \right) = 6 \ (0.085) \ (0.639) f_{s_1} = 0.326 f_{s_1} \]

\[ C_1 = 0.85 f'_c \ a \ b \]

\[ C_1 = 0.85 \ (6,000) \ (0.80 \times 3.46) \ (10.0) = 141,168 \ \text{lbs} \ (627.9 \ \text{kN}) \]

\[ C_2 = A_{s_3} f_{s_1} \left( \varepsilon_3/\varepsilon_1 \right) = 2 \ (0.085) \ (0.083) f_{s_1} \]

\[ C_2 = 0.014 f_{s_1} \]

109
Collecting terms and solving for \( f_{s1} \) yields:
\[
T_1 + T_2 = C_1 + C_2
\]
\[
0.510 f_{s1} + 0.326 f_{s1} = 141,168 + 0.014 f_{s1}
\]
\[
f_{s1} = 171,737 \text{ lbs (763.9 kN)}
\]
Therefore:
\[
T_1 = 87,586 \text{ lbs (389.6 kN)}
\]
\[
T_2 = 55,986 \text{ lbs (249.0 kN)}
\]
\[
C_1 = 141,168 \text{ lbs (627.9 kN)}
\]
\[
C_2 = 2,404 \text{ lbs (10.7 kN)}
\]
Now by taking moments about \( T_1 \) the ultimate capacity of the tie can be obtained.
\[
\Sigma M_{T1} = 0
\]
\[
M_u = C_1 (d_1 - a/2) + C_2 (d_1 - d_3) - T_2 (d_1 - d_2)
\]
\[
M_u = 141,168 \left( 9 - \left( \frac{2.77}{2} \right) \right) + 2,404 (9 - 3) - 55,986 (9 - 7)
\]
\[
M_u = 977,446 \text{ in \cdot lbs (110.5 kN\cdot m)}
\]
But \( M_u = 18 \) \( P_u \)
Therefore: \( P_u = 54.3 \text{ kips (241.5 kN)} \)

6.5 ULTIMATE MOMENT CAPACITY OF THE TYPE 2 AND TYPE 3 TIES
The ultimate moment capacity of the Type 2 and Type 3 ties was calculated in a similar manner to that of the Type 1 tie. In addition the ultimate moment capacity of the ties was also calculated for the ties in the inverted position (subjected to rebound). The ultimate moment capacity of all three tie types
in both the right-side up and inverted positions is summarized in Table 6.2.

6.6 CRACKING MOMENT OF THE TYPE 1 TIE

In order to determine the cracking moment of the tie, it is necessary to convert the prestressing strand to an equivalent concrete area, such that:

\[ A_r = A_c + A_s (n - 1) \]

Where: \( n \) = modular ratio = \( E_s/E_c \) and

\[ E_c = 57,000 \sqrt{F_c'} \]

\[ E_s = 29 \times 10^6 \text{ Psi (200 kPa)} \]

\[ A_r = 130.5 + 14(0.085) \left( \frac{29 \times 10^6}{4.42 \times 10^6} - 1 \right) \]

\[ A_r = 130.5 + 6.62 \]

\[ A_r = 137.12 \text{ in}^2 (88,470 \text{ mm}^2) \]

\[ A_t = 137.12 = \frac{d (b + b_i)}{2} \]

Assume that the height of the tie remains constant at 12" (304.8 mm) and that the effect of the steel is uniformly distributed over the width of the tie.

Therefore:

\[ 137.12 = 12 \left( \frac{b + b_i}{2} \right) \]

so \( b + b_i = 22.85" \) (580 mm)
22.85 - 21.75 = 1.10" (28 mm)
1.10/2 = 0.55" (14 mm) top and bottom
b = 10 + 0.53 = 10.53" (268 mm)

substituting and solving for \( I_T \) yields:

\[
I_T = d^3 \frac{(b^2 + 4b b_1 + b_1^2)}{36 (b + b_1)}
\]

\( I_T = 1,642 \text{ in}^4 \) (683.45 x 10^6 mm^4)

Volume of tie = \( \frac{130.50}{144} \times 12 = 10.88 \text{ ft}^3 \) (0.31 m^3)

weight of concrete = 150 lbs / ft^3 (2,400 kg/m^3)

Therefore, the tie weighs 1,632 lbs (7.3 Kn)

\( M_{self} = 816 \text{ ft} - \text{lbs} \) (1.1 kN-m) (for tie supported on a 8 ft (2.44 m) span)

Top fibre stress due to prestress and self weight only is:

\[
f_{t, self} = \frac{155,440}{130.50} - \frac{155,446 (1.13) (6.16)}{1,642} + \frac{(816 \times 12) (6.16)}{1,642}
\]

\( f_{t, self} = 569 \text{ psi} \) (3.9 MPa) compression

Bottom fibre stress due to prestress and self weight only is:

\[
f_{b, self} = \frac{155,440}{130.50} + \frac{155,400 (1.13) (5.84)}{1,642} - \frac{(816 \times 12) (5.84)}{1,642}
\]

\( f_{b, self} = 1781 \text{ psi} \) (12.3 MPa)

The modulus of rupture for concrete is given as:

\( f_r = 7.5 \sqrt{f_c'} \)

\( f_r = 580 \text{ psi} \) (2.58 MPa)

The cracking moment for both the upright and the inverted tie (supported on a 8 ft (2.44 m) span and loaded on a 5 ft (1.52 m) span) can now be calculated from the following:
$$M_{cr, pos} = \left( \frac{f_r + f_t}{Y_b} \right) I_T$$

$$M_{cr, pos} = \frac{(580 + 1,781)}{5.84} \cdot 1,642$$

$$M_{cr, pos} = 663,829 \text{ in} - \text{lbs (75.0 kN-m)}$$

$$M_{cr, pos} = 18 \ P_{cr}$$

$$P_{cr} = 36,879 \text{ lbs (164.0 kN)}$$

$$P_{cr} = 36.8 \text{ kips (164.0 kN)}$$

$$M_{cr, neg} = \left( \frac{f_r + f_t}{Y_c} \right) I_T$$

$$M_{cr, neg} = \left( \frac{580 + 569}{6.16} \right) 1642$$

$$M_{cr, neg} = 306,276 \text{ in} - \text{lbs (34.6 kN-m)}$$

$$M_{cr, neg} = 18 \ P_{cr}$$

$$P_{cr} = 17,015 \text{ lbs (75.7 kN)}$$

$$P_{c} = 17.0 \text{ kips (75.7 kN)}$$

### 6.7 CRACKING MOMENTS FOR THE TYPE 2 AND 3 TIES

The cracking moment for both the Type 2 and the Type 3 tie were calculated in a similar manner as the Type 1 tie. The values have been summarized in Table 6.2.
<table>
<thead>
<tr>
<th>TIE TYPE</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_{cr, \text{pos}}$ (in.k)</td>
<td>663.8</td>
<td>780.2</td>
<td>966.0</td>
</tr>
<tr>
<td>$P_{cr, \text{pos}}$ (kips)</td>
<td>36.8</td>
<td>43.3</td>
<td>53.7</td>
</tr>
<tr>
<td>$M_{cr, \text{neg}}$ (in.k)</td>
<td>306.3</td>
<td>333.5</td>
<td>391.5</td>
</tr>
<tr>
<td>$P_{cr, \text{neg}}$ (kips)</td>
<td>17.0</td>
<td>18.5</td>
<td>21.7</td>
</tr>
<tr>
<td>$M_{u, \text{pos}}$ (in.k)</td>
<td>977.5</td>
<td>1267.6</td>
<td>1156.5</td>
</tr>
<tr>
<td>$P_{u, \text{pos}}$ (kips)</td>
<td>54.3</td>
<td>70.4</td>
<td>64.3</td>
</tr>
<tr>
<td>$M_{u, \text{neg}}$ (in.k)</td>
<td>622.0</td>
<td>782.9</td>
<td>956.0</td>
</tr>
<tr>
<td>$P_{u, \text{neg}}$ (kips)</td>
<td>34.6</td>
<td>43.5</td>
<td>53.1</td>
</tr>
</tbody>
</table>

**TABLE 6.2 CRACKING AND ULTIMATE CAPACITY FOR TIE TYPES**

### 6.8 CALCULATION OF SPLITTING FORCE

In order to determine the shear strength of the ties the method outlined in Figure 6.7 has been used. This method will be used to determine if the ties will fail in flexure or shear.

**TYPE 1 TIES**

\[
f_{ct} = \frac{V}{ba} - \frac{54,300}{10.88 \times 18} = 277 \text{ psi (1.91 MPa)}
\]

\[
f_{ct} = \frac{Va}{bh^2} - \frac{54,300}{10.88 \times (12)^2} = 624 \text{ psi (4.30 MPa)}
\]
TAN \( \alpha = \frac{h}{a} \)

\( f_{te} = \frac{V}{ba} \)

\( f_{et} = \frac{V_0}{bh^3} \)

\( \frac{f_{et}}{f_{te}} = \frac{a^2}{h^3} \)

\[ S = \frac{a}{\cos \alpha} \]

\[ l = \frac{h}{\sin \alpha} \]

Figure 6.7 Diagram of Splitting Forces
\[ P_{\text{eff}} = 155,420 \text{ lbs (691.3 kN)} \]

\[ S = a^2 + h^2 = 18^2 + 12^2 = 468 \text{ psi (3.2 MPa)} \]

\[ f_{\text{cts}} = P_{\text{eff}} \frac{a}{S^2 h} = \frac{155,420 \times 18}{468 \times 12} = 498 \text{ psi (3.5 MPa)} \]

\[ f_{\text{tcs}} = \frac{P_{\text{eff}} h}{b S^2} = \frac{155,420 \times 12}{10.88 \times 468} = 366 \text{ psi (2.5 MPa)} \]

\[ f_c = f_{\text{ct}} + f_{\text{cts}} = 624 + 498 - 1,122 \text{ psi (7.7 MPa) tension} \]

\[ f_t = f_{\text{tc}} - f_{\text{tcs}} = 277 - 366 = -89 \text{ psi (0.6 MPa) compression} \]

\[ f_{t0} = 0.08 f_c' = 480 \text{ psi (3.3 MPa)} \]

\[ f_c = f_c' \left(1 - \frac{f_t}{f_{t0}}\right) \]

\[ f_c = 6,000 \left(1 - \frac{-89}{480}\right) = 7,112.5 \text{ (49.0 MPa)} \]

use the lesser of \( f_c' \) or \( f_c \)

\( f_c' < f_c \) therefore use \( f_c' \)

\[ V_{\text{split}} = f_c' \text{ hb} \]

\[ V_{\text{split}} = 6,000 \times (12) (10.88) \]

\[ V_{\text{split}} = 783 \text{ kips (3,484 kN)} \]

**Type 2 Ties**

\[ P_{\text{eff}} = 187,126 \text{ lbs (8.32 kN)} \]

\[ f_{\text{tc}} = \frac{V}{b a} = \frac{70,400}{(10.88) (144)} = 809 \text{ psi (2.5 MPa)} \]
\[ f_{ct} = \frac{V_a}{bh^2} = \frac{70,400 \times 18}{(10.88) (144)} = 809 \text{ psi (5.6 MPa)} \]

\[ s = A^2 + h^2 = 468 \text{ in}^2 (0.30 \text{ m}^2) \]

\[ f_{cts} = P_{eff} \frac{a}{s^2} h = \frac{187,126 \times 18}{468 \times 12} = 600 \text{ psi (4.1 MPa)} \]

\[ f_{tcs} = \frac{P_{eff} x h}{b s^2} = \frac{187,126 \times 12}{10.88 \times 468} = 441 \text{ psi (3.0 MPa)} \]

\[ f_c = f_{ct} + f_{cts} = 809 + 600 = 1,409 \text{ psi (9.7 MPa)} \text{ (tension)} \]

\[ f_t = f_{tc} - f_{tcs} = 360 - 441 = -81 \text{ psi (0.6 MPa)} \text{ (compression)} \]

\[ f_{to} = 480 \text{ psi (3.3 MPa)} \]

\[ f_c = f_c' \left(1 - \frac{f_t}{f_{to}}\right) = 6000 \left(1 - \frac{-81}{480}\right) \]

\[ f_c = 7,012.5 \text{ psi (48.3 MPa)} \]

\[ f_c' > f_c' \text{ therefore use } f_c' \]

\[ V_{\text{split}} = f_c' h b \]

\[ V_{\text{split}} = 6,000 (12) (10.88) = 783,360 (3,484 \text{ kN}) \]

\[ V_{\text{split}} = 783 \text{ kips (3,484 kN)} \]

**TYPE 3 TIES**

\[ P_{eff} = 241,046 \text{ lbs (1,072.1 kN)} \]

\[ f_{tc} = \frac{V}{ba} = 64, \frac{300}{(10.88 \times 18)} = 328 \text{ psi (2.3 MPa)} \]

\[ f_{ct} = \frac{V_a}{bh^2} = \frac{64,300 \times 18}{(10.88) (144)} = 739 \text{ psi (5.1 MPa)} \]

\[ s = 468 \text{ in}^2 (0.30 \text{ m}^2) \]
\[ f_{c_{ts}} = \frac{P_{eff} \cdot a}{s^2 \cdot h} = \frac{241,046 \times 18}{(468) \times (12)} = 773 \text{ psi (5.3 MPa)} \]

\[ f_{t_{cs}} = \frac{P_{eff} \times h}{b \cdot s^2} = \frac{241,046 \times 12}{(10.88) \times (468)} \]

\[ f_{t_{cs}} = 568 \text{ psi (3.9 MPa)} \]

\[ f_c = f_{ct} + f_{c_{ts}} = 739 + 773 = 1,512 \text{ psi (10.4 MPa)} \]

\[ f_t = f_{tc} - f_{t_{cs}} = 328 - 568 = -240 \text{ psi (1.7 MPa)} \]

\[ f_{to} = 480 \text{ psi (3.3 MPa)} \]

\[ f_c = f_c' \left(1 - \frac{f_t}{f_{to}}\right) = 6,000 \left(1 - \frac{240}{480}\right) \]

\[ f_c = 9,000 \text{ psi (62.0 MPa)} \]

Use \( f_c' = 6,000 \text{ psi (62.0 MPa)} \) as \( f_c' < f_c \)

\[ V_{split} = f_c' \times h \times b \]

\[ V_{split} = 6,000 \times (12) \times (10.88) \]

\[ V_{split} = 783 \text{ kips (3,484 kN)} \]

### 6.9 Calculation of Equivalent Impact Factor

In order to compare the relative strength of this tie in terms of an impact factor a calculation of the equivalent impact factor has been undertaken.

Knowing that the bottom stress of the tie is 1,850 psi (12.8 MPa), the total applied moment was calculated from:

\[ M_{total} = \frac{\sigma_b \cdot I}{Y_b} \]
\[ M_{\text{total}} = \frac{1,850 \times 1,560}{5.84} \]

\[ M_{\text{total}} = 494,178 \text{ in-lbs (55.8 kN.m)} \]

\[ M_{\text{total}} = 494.18 \text{ in-k (55.8 kN.m)} \]

\[ M_{\text{total}} = M_{\text{self}} + M_{\text{LL}} + M_{\text{adj}} \]

\[ M_{\text{self}} = 9,792 \text{ in-lbs (1.1 kN.m)} \]

\[ M_{\text{adj}} = 3,000 \text{ in-lbs (0.3 kN.m)} \]

\[ M_{\text{adj}} + M_{\text{self}} = 9.79 + 3.0 = 12.79 \text{ in-k (1.4 kN.m)} \]

\[ 494.18 - 12.79 = 481.39 \text{ in-k (54.4 kN.m)} \]

but \[ M_{\text{LL}} = 18 P_{\text{LL}} \]

\[ P_{\text{LL}} = 26.74 \text{ kips (118.9 kN)} \]

Consider a 263,000 lb (1,170 kN) railcar on 4 axles, which is equivalent to a 65,750 lb (301.4 kN) axle loading. The load is spread over 3 ties.

Therefore:

\[ \frac{65,750}{2 \times 3} = 10,958 \text{ lbs (48.7 kN)} \]

\[ 26.74 = (10.96)(1 + I) \]

\[ 2.44 = (I + 1) \]

\[ I = 144 \text{ percent} \]

In terms of Cooper's E-80 loading:

\[ M_{\text{LL}} = 13.33 \]

\[ 26.73 - 13.33(I + 1) \]

\[ I = 100 \text{ percent} \]

**6.10 FATIGUE CRACKING MOMENT**

The fatigue cracking moment of the concrete ties is
calculated in a similar manner to that given in section 2.9. The fatigue cracking moment (both positive and negative) for each type of tie is given as follows:

**TYPE 1 TIE**

Positive fatigue cracking moment:

\[ M_t = \frac{f_b \times I}{Y_b} = \frac{1,850 \times 1,560}{5.84} \]

\[ M_t = 494.2 \text{ in.K (55.8 kN.m)} \]

Negative fatigue cracking moment:

\[ M_t = \frac{f_t \times I}{Y_t} = \frac{500 \times 1,560}{6.16} \]

\[ M_t = 126.6 \text{ in.K (14.3 kN.m)} \]

**TYPE 2 TIE**

Positive fatigue cracking moment:

\[ M_t = \frac{f_b \times I}{Y_b} = \frac{1,850 \times 1,560}{5.84} \]

\[ M_t = 494.2 \text{ in.K (55.8 kN.m)} \]

Negative fatigue cracking moment:

\[ M_t = \frac{f_t \times I}{Y_t} = \frac{1,000 \times 1,560}{6.16} \]

\[ M_t = 253.3 \text{ in.K (28.6 kN.m)} \]

**TYPE 3 TIE**

Positive fatigue cracking moment:
\[ M_t = \frac{f_b \times I}{Y_b} = \frac{1,850 \times 1,560}{5.84} \]

\[ M_t = 494.2 \text{ in.K (55.8 kN.m)} \]

Negative fatigue cracking moment:

\[ M_t = \frac{f_c \times I}{Y_t} = \frac{1,850 \times 1,560}{6.16} \]

\[ M_t = 468.5 \text{ in.K (52.9 kN.m)} \]

### 6.11 ANALYSIS FOR DISTURBED REGIONS (END BLOCK ANALYSIS)

Analysis for the ties for several concentrated forces acting on a prismatic member indicated that the resultant tension splitting forces in the ties did not require application of stirrups.

However, to account for weaker than specified concrete or too early a release of the prestressing strand, it was decided to apply stirrups to the ties. This was done primarily to decrease the probability of bursting the ends of the ties and keep rejected ties to a minimum, especially since all ties were donated.
CHAPTER 7

TEST PROCEDURE FOR THE CONCORDIA TIES

7.1 TIE SPECIMENS
The prestressed concrete ties designed in Chapter 6 were fabricated in a single form at a precasting plant in Laval, Quebec. Six (6) full scale ties of each type were fabricated, for a total of 18 ties.

The tie specimens have been given designations in order to aid in their identification during the testing procedure and in the presentation of the results. The Type 1 ties have been designated by T1, while the six samples of this type have been sequentially numbered from B1 to B6. Therefore, the Type 1 ties have been designated T1B1 to T1B6.

Similarly, the Type 2 ties have been designated T2B1 to T2B6 and the Type 3 ties have been designated T3B1 to T3B6.

7.2 LOADING SYSTEM
The loading system consisted of simply supporting one end of the tie and roller supporting the other end at 8'- 0" (2.44 m) centre to centre and placing 2 - 0.2 in. (5mm) EVA tie pads and 2 - 132 lb. RE profile rails at standard railway gauge, 56.5" (1435 mm). The rails were approximately 12" (305 mm) long. Directly above the rails were two 120 kip (534 kN) ENERPAC RRH-6010 hydraulic jacks used for applying the loading. Immediately above one jack was a Strain-Sert 100 kip
(445 kN)) - 2 mv, 350 ohm compression load cell with a safety factor of 1.5, positioned below a load distribution beam. The load cell contained internal electrical resistance strain gauges connected to an Intertechnology P-3500 strain gauge box with an oscilloscope output to an HP 3852A Data Acquisition unit with an HP 310 Computer System.

The lateral movement of the ties was not prevented and the ties were allowed to rotate under the applied loading. This was because of the previously documented problems experienced by others performing like tests (8, 14) wherein the clamping systems failed under high loadings. Figure 7.1 illustrates the loading system used to test the ties.

7.3 INSTRUMENTATION

Two 125 ohm electrical resistance strain gauges were installed on the top and bottom vertical faces of the ties at midspan, in order to monitor the tie centre strains.

Three electrical linear velocity displacement transducers (LVDT's) were used to measure deflections of the ties at three locations. One gauge was placed directly above each of the supports of the ties and the third placed at the midspan of the tie.
Figure 7.1 Loading System Used in Laboratory Testing of Ties
7.4 TESTING PROCEDURE

The ties were individually placed in the load frame as described in section 7.2 and illustrated in Figure 7.1.

The testing program consisted of subjecting the three types of ties to increasing static loading forces noting both cracking and ultimate loads. The test program was a compilation of three tests as outlined below:

**TEST 1a** - an inverted tie was loaded in increments approximately 5 kips (22.2 kN) until first cracking was achieved. The crack location along with the load causing cracking was clearly marked on the tie. Loading was continued. The cracks propagated approximately half way up through the tie. The propagation of the cracks was indicated on the tie at 5 kip (22.2 kN) intervals. Ties tested in this manner: T1B1, T1B4, T2B1, T3B1, T3B5 and T3B6.

**TEST 1b** - the tie from Test 1a was installed in the load frame right side up (in reverse from 1a) and loaded in approximately 5 kip (22.2 kN) increments until first cracking was reached. The location of first cracking and its magnitude was clearly marked on the tie. Loading continued in 5
kip (22.2 kN) intervals, with the propagation of cracks marked, until ultimate capacity was reached. Ties tested in this manner: T1B1, T1B4, T2B1, T3B1, T3B5 and T3B6.

**TEST 2** – An inverted tie was placed in the load frame and loaded in 5 kip (22.2 kN) increments until first cracking was reached. The location of first cracking and its magnitude was clearly marked on the tie. Loading continued in 5 kip (22.2 kN) intervals, with the propagation of cracks marked, until ultimate capacity was reached. Ties tested in this manner: T1B3, T2B3, T2B4 and T3B3.

**TEST 3** – a right side up tie was placed in the load frame and loaded in 5 kip (22.2 kN) increments until first cracking was reached. The location of first cracking and its magnitude was clearly marked on the tie. Loading continued in 5 kip (22.2 kN) intervals, with the propagation of cracks marked, until ultimate capacity was reached. Ties tested in this manner: T1B5, T2B2, T2B5, T3B2 and T3B4.

In total 16 of the 18 ties fabricated were tested to destruction. The remaining two have been retained for future tests.
7.5 CONCRETE STRENGTH

The specified 28 day compressive strength of the concrete to be used in the ties was set at 6,000 psi (41.4 MPa). In order to obtain the actual compressive strength of the concrete, 4" by 8" (101.6 mm by 203.2 mm) test cylinders were poured at the same time as the ties. These cylinders were tested on a Tinius Olsen compression machine.

The mean compressive strength of the concrete from cylinder testing was found to be 7,094 psi (48.9 MPa) with a standard deviation of 1,329 psi (9.2 MPa). Table 7.1 indicates the compressive cylinder strengths of the concrete for the ties tested.
<table>
<thead>
<tr>
<th>TIE NO.</th>
<th>COMPRESSION STRENGTH psi</th>
<th>COMPRESSION STRENGTH MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1B1</td>
<td>6876</td>
<td>47.43</td>
</tr>
<tr>
<td>T1B2</td>
<td>7018</td>
<td>48.39</td>
</tr>
<tr>
<td>T1B3</td>
<td>6724</td>
<td>46.36</td>
</tr>
<tr>
<td>T1B4</td>
<td>7054</td>
<td>48.64</td>
</tr>
<tr>
<td>T1B5</td>
<td>7616</td>
<td>52.51</td>
</tr>
<tr>
<td>T2B1</td>
<td>5761</td>
<td>39.72</td>
</tr>
<tr>
<td>T2B2</td>
<td>8005</td>
<td>55.19</td>
</tr>
<tr>
<td>T2B3</td>
<td>6520</td>
<td>44.96</td>
</tr>
<tr>
<td>T2B4</td>
<td>4646</td>
<td>32.03</td>
</tr>
<tr>
<td>T2B5</td>
<td>6565</td>
<td>45.27</td>
</tr>
<tr>
<td>T3B1</td>
<td>9390</td>
<td>64.74</td>
</tr>
<tr>
<td>T3B2</td>
<td>8937</td>
<td>61.62</td>
</tr>
<tr>
<td>T3B3</td>
<td>9554</td>
<td>65.87</td>
</tr>
<tr>
<td>T3B4</td>
<td>6027</td>
<td>41.56</td>
</tr>
<tr>
<td>T3B5</td>
<td>6433</td>
<td>44.36</td>
</tr>
<tr>
<td>T3B6</td>
<td>6382</td>
<td>44.00</td>
</tr>
</tbody>
</table>

Table 7.1 Summary of Concrete Compressive Strength - $f_c$.

7.6 PRESTRESSING STEEL

The prestressing strand used in the manufacture of the ties was seven wire strand of nominal diameter 3/8" (9.5 mm) and nominal area of 0.085 in.$^2$ (55 mm$^2$). The ultimate tensile
strength of the strand is 270,000 psi (1,860 MPa).

The total prestressing force applied to each tie was measured at the precasting plant. Each strand was marked, pulled to the specified prestressing force with the amount of extension of each measured. Upon release of the prestressing jacks, the elongation in each strand was again measured. This last measurement was used in determining the total prestressing force applied to the ties.

The total elongation, after release of the jacks, was divided by the known jacking length (from anchoring bulkhead to anchoring bulkhead) to obtain the strain in each strand. The average strain for all the strands of a tie was taken and applied to the graph of the stress-strain curve, Figure 7.2, supplied by the tie manufacturer. From this an applied force per strand was obtained. The total applied force per tie was obtained by simply multiplying by the number of strands per tie.

The losses for creep and shrink of concrete were assumed as 15 percent. The total applied prestressing force was then multiplied by 0.85 to obtain the effective prestressing force per tie.

Table 7.2 indicates the actual prestressing force applied to
Figure 7.2 Stress - Strain Curve for 3/8" (9.5 mm) 7 Wire Strand
<table>
<thead>
<tr>
<th>TIE NO</th>
<th>PRESTRESSING FORCE</th>
<th>PRESTRESSING FORCE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LBS.</td>
<td>kN</td>
</tr>
<tr>
<td>T1B1</td>
<td>160,650</td>
<td>714.6</td>
</tr>
<tr>
<td>T1B2</td>
<td>154,700</td>
<td>688.1</td>
</tr>
<tr>
<td>T1B3</td>
<td>154,700</td>
<td>688.1</td>
</tr>
<tr>
<td>T1B4</td>
<td>157,675</td>
<td>701.3</td>
</tr>
<tr>
<td>T1B5</td>
<td>164,220</td>
<td>730.4</td>
</tr>
<tr>
<td>T2B1</td>
<td>248,200</td>
<td>1,104.0</td>
</tr>
<tr>
<td>T2B2</td>
<td>248,200</td>
<td>1,104.0</td>
</tr>
<tr>
<td>T2B3</td>
<td>244,800</td>
<td>1,088.9</td>
</tr>
<tr>
<td>T2B4</td>
<td>234,600</td>
<td>1,043.5</td>
</tr>
<tr>
<td>T2B5</td>
<td>261,800</td>
<td>1,164.5</td>
</tr>
<tr>
<td>T3B1</td>
<td>289,000</td>
<td>1,285.5</td>
</tr>
<tr>
<td>T3B2</td>
<td>272,000</td>
<td>1,209.9</td>
</tr>
<tr>
<td>T3B3</td>
<td>272,000</td>
<td>1,209.9</td>
</tr>
<tr>
<td>T3B4</td>
<td>301,750</td>
<td>1,342.2</td>
</tr>
<tr>
<td>T3B5</td>
<td>280,500</td>
<td>1,247.7</td>
</tr>
<tr>
<td>T3B6</td>
<td>280,500</td>
<td>1,247.7</td>
</tr>
</tbody>
</table>

Table 7.2 Summary of Effective Prestressing Forces $P_{eff}$
each tie. The average effective prestressing force applied to the Type 1 ties was 158,389 lbs (704.5 kN); the Type 2 ties 247,520 lbs (1,101 kN) and the Type 3 ties 282,625 lbs (1,257.1 kN). As can be seen from Table 7.2, the effective prestressing force applied to each tie was consistent within each group.
CHAPTER 8

COMPARISON OF THEORETICAL AND EXPERIMENTAL RESULTS FOR INDIVIDUAL TIES

8.1 GENERAL TEST OBSERVATIONS

The ties were individually loaded as outlined in Chapter 7. As the load increased, first (flexural) cracking occurred in all cases between the loaded rail locations. Thereafter as the load increased new cracks began forming, which were fairly uniformly distributed between the rails while the originally occurring cracks propagated.

In all cases the ties cracked at a lower cracking moment, $M_{cr}$, than calculated. With further increase of the load, diagonal cracks began to appear between the load point and support (shear span). Shortly thereafter the ties failed.

Failure of the ties in all cases occurred with the crushing of the concrete at the top of the tie in the rail seat area. This flexural failure was brittle and tended towards being termed explosive in nature.

All three types of ties exhibited similar cracking patterns. Figure 8.1, 8.2 and 8.3, illustrates the cracking pattern of the ties. The cracking pattern commenced with first cracking which led to uniformly distributed flexural cracking, flexural-shear cracks and finally diagonal shear cracks. In
Figure 8.1 Diagram of Typical Failure Mode of a Laboratory Tested Tie
Figure 8.2  Failure Mode of Tie T2B4 Inverted Position
Figure 8.3  Failure Mode of Tie T3B2 Right Side up Position
all cases diagonal cracks appeared just prior to failure occurring.

8.2 LOAD-DEFLECTION

Load-deflection curves, for the midspan deflection, have been plotted for selected ties and are illustrated in Figures 8.4 to 8.10. From these graphs the energy absorption capacity of the individual ties can be obtained. The ability of the prestressed ties to resist impact can be measured by the amount of energy absorbed by the ties as they deflect under load. The elastic energy absorbed by the tie can be calculated by obtaining the area under the load-deflection curve up to the point of first cracking. The total energy that can be absorbed by the tie is calculated from the total area under the load-deflection curve up to the point of rupture.

According to the research studies (13), the load-deflection curves, obtained by static load tests, may be used to obtain the energy absorption capacity of the ties under dynamic loading. Therefore, load-deflection curves can be used as a measure of the impact resistance of the ties. Table 8.1 indicates both the elastic and total energy absorbed by each tie illustrated in Figures 8.4 to 8.10. The greater the ties' resilience, the greater impact resistance it will display.
Figure 8.4 Load - Deflection Curve for Tie T1B2
Figure 8.5 Load - Deflection Curve for Tie T1B3
Figure 8.6 Load - Deflection Curve for the T2Bl
Figure 8.7 Load - Deflection Curve for Tie T2B2
Figure 8.8 Load - Deflection Curve for Tie T2B3
Figure 8.9 Load - Deflection Curve for Tie T3B1
LOAD-DEFLECTION CURVE
T3B2

Figure 8.10 Load - Deflection Curve for Tie T3B2
## Table 8.1 Elastic and Total Energy Absorption of Ties

<table>
<thead>
<tr>
<th>TIE NO.</th>
<th>ELASTIC ENERGY</th>
<th>TOTAL ENERGY</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>in.-k</td>
<td>Kn.m</td>
</tr>
<tr>
<td>T1B2</td>
<td>8.83</td>
<td>1.00</td>
</tr>
<tr>
<td>T1B3</td>
<td>8.39</td>
<td>0.90</td>
</tr>
<tr>
<td>T2B1</td>
<td>6.58</td>
<td>0.75</td>
</tr>
<tr>
<td>T2B2</td>
<td>6.53</td>
<td>0.73</td>
</tr>
<tr>
<td>T2B3</td>
<td>1.71</td>
<td>0.19</td>
</tr>
<tr>
<td>T3B1</td>
<td>7.02</td>
<td>0.80</td>
</tr>
<tr>
<td>T3B2</td>
<td>2.47</td>
<td>0.28</td>
</tr>
</tbody>
</table>

The amount of ductility displayed in each beam was fairly consistent. The inverted ties generally showed less ductility than the ties tested in the normal position. From the results the Type 1 ties displayed the greatest energy absorption capability in both the inverted and normal positions.

### 8.3 Comparison of Cracking Moments

Both the positive and negative theoretical cracking moment for each tie have been calculated by the method outlined in Chapter 6. These results have been compared to the actual cracking moments outlined by testing. Table 8.2 illustrates the comparison of the theoretical negative cracking moments with the experimental cracking moments. Table 8.3 illustrates the comparison of the theoretical positive cracking moments to
the experimental cracking moments.

The theoretical cracking moment calculation are by no means an exact figure, but rather give an estimation of the cracking moment. The cracking moment is most dependant on the stress state of the concrete, however, the cement mix, strength and size of aggregate and water cement ratio also have an influence.

<table>
<thead>
<tr>
<th>TIE NO.</th>
<th>THEORETICAL CRACKING MOMENT</th>
<th>EXPERIMENTAL CRACKING MOMENT</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>in.K</td>
<td>kN.m</td>
</tr>
<tr>
<td>T1B1</td>
<td>302</td>
<td>34.2</td>
</tr>
<tr>
<td>T1B3</td>
<td>296</td>
<td>33.4</td>
</tr>
<tr>
<td>T1B4</td>
<td>302</td>
<td>34.2</td>
</tr>
<tr>
<td>T2B1</td>
<td>510</td>
<td>57.6</td>
</tr>
<tr>
<td>T2B3</td>
<td>513</td>
<td>57.9</td>
</tr>
<tr>
<td>T2B4</td>
<td>478</td>
<td>54.0</td>
</tr>
<tr>
<td>T3B1</td>
<td>788</td>
<td>89.1</td>
</tr>
<tr>
<td>T3B3</td>
<td>755</td>
<td>85.3</td>
</tr>
<tr>
<td>T3B5</td>
<td>748</td>
<td>84.5</td>
</tr>
<tr>
<td>T3B6</td>
<td>747</td>
<td>84.4</td>
</tr>
</tbody>
</table>

TABLE 8.2 Comparison of Negative Cracking Moments
<table>
<thead>
<tr>
<th>TIE NO.</th>
<th>THEORETICAL CRACKING MOMENT</th>
<th>EXPERIMENTAL CRACKING MOMENT</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>in.k</td>
<td>kN.m</td>
</tr>
<tr>
<td>T1B1</td>
<td>682</td>
<td>77.1</td>
</tr>
<tr>
<td>T1B2</td>
<td>665</td>
<td>75.1</td>
</tr>
<tr>
<td>T1B4</td>
<td>675</td>
<td>76.3</td>
</tr>
<tr>
<td>T1B5</td>
<td>702</td>
<td>79.3</td>
</tr>
<tr>
<td>T2B1</td>
<td>819</td>
<td>92.6</td>
</tr>
<tr>
<td>T2B2</td>
<td>847</td>
<td>95.7</td>
</tr>
<tr>
<td>T2B5</td>
<td>866</td>
<td>97.9</td>
</tr>
<tr>
<td>T3B1</td>
<td>795</td>
<td>89.8</td>
</tr>
<tr>
<td>T3B2</td>
<td>755</td>
<td>85.3</td>
</tr>
<tr>
<td>T3B4</td>
<td>783</td>
<td>88.5</td>
</tr>
<tr>
<td>T3B5</td>
<td>745</td>
<td>84.2</td>
</tr>
<tr>
<td>T3B6</td>
<td>744</td>
<td>84.1</td>
</tr>
</tbody>
</table>

**TABLE 8.3 Comparison of Positive Cracking Moments**

Due to the variation in the compressive strength of the concrete in each tie, all experimental results have been divided by the compression strength of concrete in order to obtain comparative relative cracking data. Figure 8.11 illustrates the relationship between $M_{cr}/bh^2f'_c$ and the bottom fibre stress, $f_b/f'_c$ for both the inverted and normal positions. These values are shown in Table 8.4.
<table>
<thead>
<tr>
<th>TIE NO.</th>
<th>$f_y/f'_c$</th>
<th>$M_{cr}/(bh^2f'_c)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1B1</td>
<td>0.2782</td>
<td>0.0502</td>
</tr>
<tr>
<td>T1B2</td>
<td>0.2623</td>
<td>0.0491</td>
</tr>
<tr>
<td>T1B3</td>
<td>0.0788</td>
<td>0.0244</td>
</tr>
<tr>
<td>T1B4</td>
<td>0.2642</td>
<td>0.0326</td>
</tr>
<tr>
<td>T1B5</td>
<td>0.2567</td>
<td>0.0453</td>
</tr>
<tr>
<td>T2B1</td>
<td>0.4258</td>
<td>0.0698</td>
</tr>
<tr>
<td>T2B2</td>
<td>0.4258</td>
<td>0.0502</td>
</tr>
<tr>
<td>T2B3</td>
<td>0.2074</td>
<td>0.0264</td>
</tr>
<tr>
<td>T2B4</td>
<td>0.2752</td>
<td>0.0247</td>
</tr>
<tr>
<td>T2B5</td>
<td>0.3942</td>
<td>0.0700</td>
</tr>
<tr>
<td>T3B1</td>
<td>0.2362</td>
<td>0.0367</td>
</tr>
<tr>
<td>T3B2</td>
<td>0.2336</td>
<td>0.0257</td>
</tr>
<tr>
<td>T3B3</td>
<td>0.2249</td>
<td>0.0301</td>
</tr>
<tr>
<td>T3B4</td>
<td>0.3844</td>
<td>0.0381</td>
</tr>
<tr>
<td>T3B5</td>
<td>0.3347</td>
<td>0.0536</td>
</tr>
<tr>
<td>T3B6</td>
<td>0.3374</td>
<td>0.0450</td>
</tr>
</tbody>
</table>

Table 8.4 Values for $M_{cr}/bh^2f_c$, and $f_y/f_c$.

As can be seen from this graph, an increase in the bottom fibre compressive prestress level leads to an increase in the cracking moment. This merely confirms what is previously known, but has been included here for completeness.
$f_b / f_c$ vs $M_{cr} / (bh^2 f_c)$

Figure 8.11 Relationship Between $M_{cr}/bh^2 f_c$, and $f_b/f_c$. 
8.4 ULTIMATE MOMENT CAPACITY

Both the theoretical ultimate positive and ultimate negative moments for each tie have been calculated by the stress-strain compatibility method outlined in Chapter 6. Table 8.5 illustrates the comparison of the theoretical ultimate moment capacity with the experimental ultimate moment capacity for both negative and positive bending.

As can be seen from Table 8.5, the average experimental ultimate moment was 91.7 percent of the theoretical ultimate moment. This indicates that the experimental and theoretical values compare favourably.

It should be noted that no capacity reduction factor was used in the theoretical ultimate moment capacity calculations. Normal practice (4, 7) is to use a capacity reduction factor, $\phi$, to allow for variations in the steel, concrete and in construction dimensions. However, had a capacity reduction factor $\phi$ (sometimes called $\phi$ factor) of 0.90 for flexure been used in the calculation, very close agreement between the theoretical and experimental values would have been obtained.

Again, due to the variation in the compressive strength of concrete in each tie, the experimental results have been divided by $bh^2f'_c$ in order to obtain comparative relative
<table>
<thead>
<tr>
<th>TIE NO.</th>
<th>THEORETICAL</th>
<th>EXPERIMENTAL</th>
<th>( \frac{\mu_{\text{exp}}}{\mu_{\text{theo}}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1B1</td>
<td>1,148 in.K</td>
<td>1,221.3 in.K</td>
<td>1.06</td>
</tr>
<tr>
<td></td>
<td>129.7 kN.m</td>
<td>138.0 kN.m</td>
<td></td>
</tr>
<tr>
<td>T1B2</td>
<td>1,145 in.K</td>
<td>1,245.6 in.K</td>
<td>1.09</td>
</tr>
<tr>
<td></td>
<td>129.4 kN.m</td>
<td>140.8 kN.m</td>
<td></td>
</tr>
<tr>
<td>T1B3</td>
<td>646 in.K</td>
<td>869.4 in.K</td>
<td>1.35</td>
</tr>
<tr>
<td></td>
<td>73.0 kN.m</td>
<td>98.2 kN.m</td>
<td></td>
</tr>
<tr>
<td>T1B4</td>
<td>1,165 in.K</td>
<td>1,038.8 in.K</td>
<td>0.89</td>
</tr>
<tr>
<td></td>
<td>131.6 kN.m</td>
<td>117.4 kN.m</td>
<td></td>
</tr>
<tr>
<td>T1B5</td>
<td>1,289 in.K</td>
<td>1,422.9 in.K</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td>145.7 kN.m</td>
<td>160.8 kN.m</td>
<td></td>
</tr>
<tr>
<td>T2B1</td>
<td>1,427 in.K</td>
<td>1,266.8 in.K</td>
<td>0.89</td>
</tr>
<tr>
<td></td>
<td>161.3 kN.m</td>
<td>143.2 kN.m</td>
<td></td>
</tr>
<tr>
<td>T2B2</td>
<td>1,561 in.K</td>
<td>1,280.3 in.K</td>
<td>0.82</td>
</tr>
<tr>
<td></td>
<td>176.3 kN.m</td>
<td>144.7 kN.m</td>
<td></td>
</tr>
<tr>
<td>T2B3</td>
<td>940 in.K</td>
<td>898.9 in.K</td>
<td>0.96</td>
</tr>
<tr>
<td></td>
<td>106.2 kN.m</td>
<td>101.6 kN.m</td>
<td></td>
</tr>
<tr>
<td>T2B4</td>
<td>869 in.K</td>
<td>672.1 in.K</td>
<td>0.77</td>
</tr>
<tr>
<td></td>
<td>98.2 kN.m</td>
<td>75.9 kN.m</td>
<td></td>
</tr>
<tr>
<td>T2B5</td>
<td>1,480 in.K</td>
<td>1,443.1 in.K</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>167.2 kN.m</td>
<td>163.1 kN.m</td>
<td></td>
</tr>
<tr>
<td>T3B1</td>
<td>1,494 in.K</td>
<td>1,496.5 in.K</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>168.8 kN.m</td>
<td>169.1 kN.m</td>
<td></td>
</tr>
<tr>
<td>T3B2</td>
<td>1,472 in.K</td>
<td>986.0 in.K</td>
<td>0.67</td>
</tr>
<tr>
<td></td>
<td>166.4 kN.m</td>
<td>114.4 kN.m</td>
<td></td>
</tr>
<tr>
<td>T3B3</td>
<td>1,440 in.K</td>
<td>1,150.0 in.K</td>
<td>0.80</td>
</tr>
<tr>
<td></td>
<td>162.7 kN.m</td>
<td>130.0 kN.m</td>
<td></td>
</tr>
<tr>
<td>T3B4</td>
<td>1,312 in.K</td>
<td>672.1 in.K</td>
<td>0.51</td>
</tr>
<tr>
<td></td>
<td>148.3 kN.m</td>
<td>75.9 kN.m</td>
<td></td>
</tr>
<tr>
<td>T3B5</td>
<td>1,337 in.K</td>
<td>1,217.9 in.K</td>
<td>0.91</td>
</tr>
<tr>
<td></td>
<td>151.1 kN.m</td>
<td>137.6 kN.m</td>
<td></td>
</tr>
<tr>
<td>T3B6</td>
<td>1,332 in.K</td>
<td>1,161.0 in.K</td>
<td>0.87</td>
</tr>
<tr>
<td></td>
<td>150.5 kN.m</td>
<td>131.2 kN.m</td>
<td></td>
</tr>
</tbody>
</table>

Table 8.5 Comparison of Theoretical and Experimental Ultimate Moment Capacities
ultimate capacity data. Figure 8.12 indicates the graph of $M_y/bh^2f'_c$ versus $f_y/f'_c$. From this graph it can be seen that increasing the bottom prestress increases the ultimate moment capacity of the tie. This is previously known and been included here for the sake of completeness. It should be noted that inverting the tie causes the top prestress to become the bottom prestress and has therefore been denoted as $f_b$. 
<table>
<thead>
<tr>
<th>TIE NO.</th>
<th>$f_t$ (psi)</th>
<th>$M_u$ (in.k)</th>
<th>$f_t/f_{c'}$</th>
<th>$M_u/bh^2f_{c'}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1B1</td>
<td>516</td>
<td>1,221</td>
<td>0.0750</td>
<td>0.1134</td>
</tr>
<tr>
<td>T1B2</td>
<td>497</td>
<td>1,246</td>
<td>0.0708</td>
<td>0.1133</td>
</tr>
<tr>
<td>T1B3</td>
<td>1,851</td>
<td>869</td>
<td>0.2753</td>
<td>0.0826</td>
</tr>
<tr>
<td>T1B4</td>
<td>506</td>
<td>1,039</td>
<td>0.0717</td>
<td>0.0904</td>
</tr>
<tr>
<td>T1B5</td>
<td>527</td>
<td>1,423</td>
<td>0.0692</td>
<td>0.1193</td>
</tr>
<tr>
<td>T2B1</td>
<td>1,327</td>
<td>1,267</td>
<td>0.2303</td>
<td>0.1404</td>
</tr>
<tr>
<td>T2B2</td>
<td>1,327</td>
<td>1,280</td>
<td>0.1658</td>
<td>0.1021</td>
</tr>
<tr>
<td>T2B3</td>
<td>2,420</td>
<td>899</td>
<td>0.3712</td>
<td>0.0880</td>
</tr>
<tr>
<td>T2B4</td>
<td>2,319</td>
<td>672</td>
<td>0.4991</td>
<td>0.0924</td>
</tr>
<tr>
<td>T2B5</td>
<td>1,401</td>
<td>1,443</td>
<td>0.2134</td>
<td>0.1403</td>
</tr>
<tr>
<td>T3B1</td>
<td>2,218</td>
<td>1,497</td>
<td>0.2362</td>
<td>0.1018</td>
</tr>
<tr>
<td>T3B2</td>
<td>2,088</td>
<td>986</td>
<td>0.2336</td>
<td>0.0704</td>
</tr>
<tr>
<td>T3B3</td>
<td>2,088</td>
<td>1,150</td>
<td>0.2185</td>
<td>0.0769</td>
</tr>
<tr>
<td>T3B4</td>
<td>2,317</td>
<td>834</td>
<td>0.3844</td>
<td>0.0884</td>
</tr>
<tr>
<td>T3B5</td>
<td>2,153</td>
<td>1218</td>
<td>0.3347</td>
<td>0.1209</td>
</tr>
<tr>
<td>T3B6</td>
<td>2,153</td>
<td>1161</td>
<td>0.3374</td>
<td>0.1162</td>
</tr>
</tbody>
</table>

Table 8.6 Values for $M_u/bh^2f_{c'}$ and $f_t/f_{c'}$.  

153
Figure 8.12 Relationship Between $M_u/(bh^2f_{c'})$ and $f_b/f_{c'}$. 

$f_b/f_{c'}$ vs $M_u/(bh^2f_{c'})$
Figure 8.13 Relationship Between $M_u/bh^2f_c'$ and $f_t/f_c'$. 
Figure 8.13 illustrates the relationship between $M_u/bh^2f_c'$ and $f_t/f_c'$ for positive loading only from values contained in Table 8.6. This graph indicates that increasing the level of prestress in the top of the tie, $f_t$, has a negligible effect on the ultimate positive moment capacity, $M_u$, of the tie.

This is important as it suggests that in the design of prestressed ties (or other members), subjected to reverse bending, additional prestress may be added to the tie to prevent negative cracking, without appreciably affecting the positive ultimate moment capacity.

It should be noted that the Type 3 ties, which had uniform prestress ($e = 0$), exhibited only a 10% average decrease in positive ultimate moment capacity over the Type 2 ties.

8.5 STRAIN GAUGE READINGS

Although strain readings were recorded for the laboratory portion of the research, they could not be used in any analysis. On closer review of the measured strains, it became apparent that these strains were meaningful up to the point of first cracking and thereafter became meaningless. First cracking of the ties occurred at a loading of 15 kips (66.7 kN) to 30 kips (133.4 kN), while the ultimate load was between 60 kips (266.8 kN) to 80 kips (355.8 kN). This indicates that the strain gauges were only effective for a short period of the test performed on each tie.
CHAPTER 9

CONCLUSIONS AND RECOMMENDATIONS

9.1 CONCLUSIONS

The following conclusions may be drawn from the field testing performed in 1984 and 1986 and the laboratory testing conducted at Concordia University in 1987:

1) The combination of soft rail-tie pad (SBR 60 durometer) with soft tie-girder pad (AASHTO 60 durometer) gave the lowest dynamic strains in the tie. This indicates that this combination of pads attenuates the most impact.

2) The combination of hard rail-tie pad (EVA) and hard tie-girder pad (SA-17) gave the largest dynamic strains in the ties.

3) The hard rail-tie and hard tie-girder pad combination gave strains 5% percent higher than that of the soft rail-tie and soft tie-girder pad combination.

4) The use of a hard rail-tie pad with a soft tie-girder pad showed negligible difference in strains in the tie when compared to the use of a hard rail-tie pad and a hard tie-girder pad. This indicates that the dynamic strains experienced by the ties are influenced the greatest by the rail-tie pad, and that the tie-girder pad has a insignificant effect on the strains.

5) The use of a soft rail-tie pad however, contributes to higher negative bending strains when the tie rebounds.

6) The maximum recorded negative strain experienced by the
C.N.R. bridge tie was 129.9 $\times 10^{-6}$ in/in in tension. The existing tie design is based on a compression strain of 47.5 $\times 10^{-6}$ in/in. Therefore, the existing C.N.R. concrete bridge tie is under designed in negative bending by at least 270 percent.

7) The use of the soft 6.5 mm Hytrel rail-tie pads gave dynamic strain readings approximately 20 percent less than the use of the soft 5 mm SBR rail-tie pads.

8) The use of the 6.5 mm Hytrel rail-tie pads and the installation of the 50 concrete track ties on the approaches to the bridge reduced the positive strains by 18 percent.

9) Both lightly and heavily loaded vehicles generated similar dynamic strain levels in the tie for the same velocity. This indicates that impacts are more a function of the unsprung mass and velocity of the vehicles.

10) Because lightly loaded and heavily loaded cars generate the same dynamic strain levels in the concrete ties, impact factors are not a practical design tool.

11) Impact factors do not assist in the design of concrete ties subjected to rebound, as they do not help predict the level of precompression required in the top of the tie.

12) The strains obtained from field testing and the strains obtained from the model for both positive and negative
bending show good correlation. For instance, the maximum negative bending strain (from rebound) obtained from the field data is $129.9 \times 10^{-6}$ in/in and the strains obtained from the model is $106 \times 10^{-6}$ in/in.

13) Failure of the ties in all cases was one of compression, with crushing of the concrete at the top of the tie.

14) Load-deflection curves obtained by static load tests are an indicator of the energy absorption capacity of the ties. The greater the ties' resilience, the greater its' impact resistance.

15) The experimental ultimate moment capacities compare favourably with the theoretical calculations. The average experimental ultimate moment capacity was 91.7 percent of the theoretical capacity.

16) An increase in the prestress level in the tension zone (bottom) of the tie increases the cracking strength of the tie.

17) An increase in the prestress level in the compression zone (top) of the tie has negligible effect on the positive ultimate moment capacity of the tie and greatly increases the cracking strength of the tie in rebound.

18) The Type 1 tie with a precompression level of 500 psi (3.4 MPa) in the top and 1,850 psi (12.86 MPa) in the bottom would be an ideal substitute for the current C.N.R. design.
9.2 RECOMMENDATIONS

From the research work undertaken during the course of this study, the following recommendations are presented:

1) The railway should continue to use the 6.5 mm Hytreel rail-tie pad as it attenuates the greatest impact.

2) The railway should adopt a tie design similar to the Concordia Type 1 tie with a 500 psi (3.4 MPa) precompression level in the top and a 1,850 psi (12.8 MPa) precompression level in the bottom.

3) The railway should discontinue the use of 5 mm wire as the number of wires required, from a practical viewpoint, would make jacking difficult because of interference. The use of 3/8" (9.55 mm) 7 wire strand is recommended.

4) The existing trapezoidal tie shape, 10" (254 mm) wide at the top, 11-3/4" (298 mm) wide at the bottom and 12" (305 mm) high, should be maintained.

5) Air entrained concrete of approximately 5 percent with a compression strength of 6,000 psi (41.4 MPa) minimum should be used in the manufacture of the ties to prevent the effects of freeze-thaw.

6) The use of an impact factor in design of future ties should be avoided, however, if this is not practical, the precompression level at the top of the tie should be a minimum 27 percent of the precompression level at the bottom of the tie to avoid cracking from rebound.
7) Concrete track ties should be installed on the approaches to any bridge with concrete bridge ties. This will help in the transition from ballasted track to the bridge. In addition, the track ties closest to the backwall of the bridge must be kept surfaced so as to prevent large impacts on the end bridge ties.

8) An economic analysis should be undertaken as an increased amount of prestress leads to an increase in the consumption of steel strand, which must be balanced with the improved performance and durability of the ties.

9) If the railway moves in the direction of increasing car loadings from the 100 ton car, 263,000 lbs. (1,170 kN) on 4 axles to the 125 ton car, 315,000 lbs. (1,401 kN) on 4 axles as is the trend among North American railways, then the design of the Type 1 tie should be reviewed.

10) As the statistical data available from the CN field testing is by no means complete, it is currently the best available. However, further statistical data should be obtained through cooperation with a Railway and the Concordia Type 1 tie re-evaluated in light of this data.
REFERENCES


162
REFERENCES


16) Scott, J. F., "Evaluation of Turnout Ties and Bridge Ties", Paper Presented to the American Railway Engineering
REFERENCES


21) Watstein, D., "Effect of Straining Rate on the Compressive Strength and Elastic Properties of Concrete", Journal of the American Concrete Institute, Detroit, Vol. 24, No. 8, April 1953.

APPENDIX A

TWO WIRE TRANSMITTER BOX

OUTPUT VOLTAGE FORMULATION
APPENDIX A

TWO WIRE TRANSMITTER BOX OUTPUT VOLTAGE FORMULATION

1.0 GENERAL CONDITIONS OF A WHEATSTONE BRIDGE

An accurate method of measuring the resistance is by means of a circuit using a Wheatstone Bridge. The formulation for this is as outlined below. See Figures A1.1 and A1.2.

\[ V_b = I_1 (R_1 + R_2) \quad \text{and} \quad e_1 = V_b - I_1 R_1 \]
\[ V_b = I_2 (R_3 + R_4) \quad \text{and} \quad e_2 = V_b - I_2 R_4 \]

Therefore:

\[ e_1 - e_2 = V = (V_b - I_1 R_1) - (V_b - I_2 R_4) \]
\[ e_1 - e_2 = V = I_2 R_4 - I_1 R_1 \]

but: \[ I_1 = \frac{V_b}{(R_1 + R_2)} \quad \text{and} \quad I_2 = \frac{V_b}{(R_3 + R_4)} \]

\[ e_1 - e_2 = \frac{V_b R_4}{(R_3 + R_4)} - \frac{V_b R_1}{(R_1 + R_2)} \]

Initially it is assumed that there is no strain on the system so that when \( R_1 = R_2 \) and \( R_3 = R_4 \), \( e_1 = e_2 = 0 \)

This means that the output of the bridge can be set to zero. The bridge under strain is affected by the following:

1) forces applied to the system and Poisson's effect.

2) temperature variations.

The strains due to stresses in three dimensions are as follows:

\[ \varepsilon_x = \frac{\sigma_x}{E} - \mu \frac{\sigma_y}{E} - \mu \frac{\sigma_z}{E} \]
\[ \varepsilon_y = \frac{\sigma_y}{E} - \mu \frac{\sigma_x}{E} - \mu \frac{\sigma_z}{E} \]
\[ \varepsilon_z = \frac{\sigma_z}{E} - \mu \frac{\sigma_x}{E} - \mu \frac{\sigma_y}{E} \]

The strains due to temperature variations are given by:

\[ \varepsilon_1 = \alpha \Delta T \]
Figure A 1.0 Schematic of a Wheatstone Bridge

Figure A 1.1 Schematic of Strain Gauge Rosettes
If the material is homogeneous and isotropic, the strain will be the same in all directions of the system. The resistance values under the effects of strain and temperature are as outlined below:

if \( R_1 = R_2 = r \) and \( R_3 = R_4 = R \) (assumed when the system is initially installed) the following conditions apply:

\[
\begin{align*}
R_1 &= r + \Delta R_f^a + \Delta R_t^a \\
R_2 &= r - \Delta R_\mu^a + \Delta R_t^a \\
R_3 &= R + \Delta R_f^b + \Delta R_t^b \\
R_4 &= R - \Delta R_\mu^b + \Delta R_t^b \\
\end{align*}
\]

where:

Superscripts \( a \) and \( b \) indicate the particular surface of the tie.

Subscript \( F \) indicates the strain due to applied force.

Subscript \( \mu \) indicates strain due to Poisson's effect.

Subscript \( T \) indicates strain due to temperature variations.

Therefore the equation for \( e_1 - e_2 \) can be written as:

\[
e_1 - e_2 = V_B \left[ \frac{R - \Delta R_\mu^b + \Delta R_T^b}{2R + \Delta R_f^b - \Delta R_\mu^b + 2\Delta R_T^b} - \frac{r + \Delta R_f^a + \Delta R_T^a}{2r + \Delta R_f^a - \Delta R_\mu^a + 2\Delta R_T^a} \right]
\]

For stresses due to forces, we have the following:

\[
\begin{align*}
\frac{\Delta R}{R} &= F \epsilon_x^b \quad \text{and} \quad \frac{\Delta R}{r} = F \epsilon_x^a \\
\Delta R_T^b &= RF \epsilon_x^b \quad \text{and} \quad \Delta R_T^a = rF \epsilon_x^a \\
\Delta R_\mu^b &= RF \mu \epsilon_x^b \quad \text{and} \quad \Delta R_\mu^a = rF \mu \epsilon_x^a \\
\end{align*}
\]

therefore:
\[ e_1 - e_2 = V_B \left[ \frac{1 - F \mu e_x^b + (\Delta R_T^b/R)}{2 + F e_x^b (1 - \mu) + (2 \Delta R_T^b/R)} - \frac{1 + F e_x^a + (\Delta R_T^a/R)}{2 + F e_x^a (1 - \mu) + (2 \Delta R_T^a/R)} \right] \]

It should be noted that in the above equation it is assumed that the initial value of \( e_1 - e_2 \) prior to any change in strain is zero.

### 2.1 The Half Bridge

In the situation where a half bridge is used, \( R_3 \) and \( R_4 \) are completion resistors and their effect on surfaces \( a \) and \( b \) are not considered.

Therefore:

\( \Delta R_f^b = \Delta R_x^b = \Delta R_t^b = 0 \)

\( \Delta R_f^a = \Delta R_f \)

\( \Delta R_x^a = \Delta R_x \)

\( \Delta R_t^a = \Delta R_t \)

Equation \( e_1 - e_2 \) now becomes:

\[ e_1 - e_2 = V_B \left[ \frac{1}{2} - \frac{r + \Delta R_f + \Delta R_t}{2r + \Delta R_f - \Delta R_x + 2 \Delta R_t} \right] \]

\[ e_1 - e_2 = V_B \left[ \frac{1}{2} - \frac{1 + F e_x + (\Delta R_T/r)}{2 + F e_x (1 - \mu) + (2 \Delta R_T/r)} \right] \]

\[ e_1 - e_2 = V_B \left[ \frac{-F e_x (1 - \mu)}{2(2 + F e_x (1 - \mu) + (2 \Delta R_T/r)} \right] \]

The following is assumed:
1) the temperature variation during the test is small, such that \[ \Delta R_t/r = << 1 \]

2) the maximum strain is \( \epsilon_x = 500 \mu \epsilon \), such that:
\[ F \epsilon_x (1 - \mu) = 0.04 \text{ percent} \]

therefore:
\[ e_1 - e_2 = V_b - \left( \frac{F \epsilon_x (1 + \mu)}{4} \right) \]

The output voltage at the 25 ohm resistors of the T.W.T. is as follows:
\[ E_0 = - \frac{V_b F \epsilon_x A (1 + \mu)}{4} + V_{set} \]

where:
\( A \) is the gain of the T.W.T. = 500
\( V_{set} \) is the set voltage level, which was 2 volts
\( E_0 \) is the voltage change on a 25 ohm load resistor
\( F \) is the gauge factor, 2.055 for concrete
2.000 for prestressing wire
2.120 for the steel girders
\( \epsilon_x \) is the strain in the active gauge
\( \mu \) is Poisson's ratio, 0.18 for concrete
0.30 for steel

1.3 MODIFYING THE BRIDGE VOLTAGE
To modify the bridge voltage \( V_b \), padding resistors designated \( R_p \) must be added. See Figure A.1.2
Figure A 1.2 Schematic of a Wheatstone Bridge With Padding Resistors
\[ \frac{1}{R_T} = \frac{1}{2R_V} + \frac{1}{2R_C} \]

\[ R_T = \frac{2R_V R_C}{R_V + R_C} \]

\[ V_{ex} = I \ (R_T + 2R_p) \]

\[ V_{ex} = I \left[ \frac{2R_V R_C + 2R_p(R_V + R_C)}{R_V + R_C} \right] \]

\[ V_{ex} = 2IR_p + V_B \text{ and } V_B = V_{ex} - 2IR_p \]

\[ V_B = V_{ex} \left[ 1 - \frac{2R_p (R_V + R_C)}{2R_V R_C + 2R_p (R_V + R_C)} \right] \]

\[ V_B = V_{ex} \left[ \frac{R_V R_C}{R_V R_C + R_p (R_V + R_C)} \right] \]

Knowing the values of both \( V_B \) and \( V_{ex} \) the value of the padding resistors can be found as follows:

\[ V_{ex} = V_B \left[ 1 + \frac{R_p (R_V + R_C)}{R_V R_C} \right] \]

\[ R_p = \frac{V_{ex} - V_B}{V_B} \times \frac{R_V R_C}{R_V + R_C} \]

### 1.4 The Effect of the Calibration Switch in the T.W.T. Box

When the calibration switch on the receiver is in the "on" position, the resistor \( R_{ec} \) is shunted across the \( R_c \) resistor.
Figure A 1.3 Schematic of a Wheatstone Bridge With Calibration Switch
See Figure A 1.3.

The new resistance of the arm AB now becomes:

\[ \frac{1}{R_c'} = \frac{1}{R_c} + \frac{1}{R_{cc}} \]

\[ R_c' = \frac{R_{cc}R_c}{R_{cc} + R_c} \]

therefore:

\[ R_c' = R_c + \Delta R_c \text{ where } \Delta R_c = R_c' - R_c \]

\[ R_c' - R_c = \frac{R_{cc}R_c}{R_{cc} + R_c} - \frac{R_c(R_{cc} + R_c)}{R_{cc} + R_c} = \frac{-R_c^2}{R_{cc} + R_c} \]

\[ \Delta R_c' = \frac{-R_c^2}{R_{cc} + R_c} \]

therefore:

\[ R_c' = R_c + \Delta R_c = R_c(1 + \Delta R_c/R_c) \]

\[ R_c' = R_c \left[ 1 - \frac{R_c}{R_{cc} + R_c} \right] \]

It is known that,

\[ \epsilon \times F = \frac{\Delta R_c}{R_c} = \frac{R_c}{R_{cc} + R_c} \]

where \( \Delta R_c/R_c \ll 1 \)

The expression for strain, when the calibration switch at the receiver is in the on position, depends on if there was previously a test calibrating the applied exitation or load versus either:

1) the total bridge strain , comprising of Poisson's ratio (in the case of a half bridge rosette) = \( \epsilon_x(1 + \mu) \), or
2) the actual longitudinal strain in the member (for example in the x direction) = $\varepsilon_x$ without Poisson's effect.

Therefore, in the case of a half bridge rosette for longitudinal stress in one direction (x) the following applies:

a) the total bridge strain $\varepsilon_x^B$:

$$\varepsilon_x^B = \varepsilon_x (1 + \mu)$$

$$\varepsilon_x^B = \frac{1}{F} x \frac{\Delta R_c}{R_c} = \frac{1}{F} x \frac{R_c}{R_{cc} + R_c}$$

b) the actual longitudinal strain in the member $\varepsilon_x$:

$$\varepsilon_x = \frac{1}{F} x \frac{1}{(1 + \mu)} x \frac{R_c}{(R_{cc} + R_c)}$$

where: $R_{cc} = 349,650$ ohms and $R_c = 350$ ohms

$$E_0 = \frac{V_B FA \varepsilon_x^B}{4}$$

$$E_B = \frac{-V_B A}{4(1 + \mu)} \left[ \frac{R_c}{R_{cc} + R_c} \right]$$
APPENDIX B

OCCURRENCE VERSUS STRAIN

AND

CUMULATIVE PERCENT EXCEEDING GRAPHS

FOR 1984 AND 1986 C.N. FIELD TESTS
% OCCURRENCE VS STRAIN

SUMMARY 3RB - POSITIVE BENDING 1984

Figure B.1.0 Summary Graph 3RB for 1984
Figure B 1.1 Cumulative Exceeding Graph for 3RB, 1984
Figure B 1.2 Summary Graph 8RB for 1984
Figure B 1.3 Cumulative Exceeding Graph for 8RB, 1984
Figure B 1.4 Summary Graph 15RB for 1984
%OCCURRENCE vs STRAIN
SUMMARY 20RB - POSITIVE BENDING 1984

Figure B 1.6 Summary Graph 20RB for 1984
CUM. PERCENT EXCEEDING vs STRAIN
SUMMARY 20RB – POSITIVE BENDING 1984

Figure B 1.7 Cumulative Exceeding Graph for 20RB, 1984
%OCCURRENCE vs STRAIN - POSITIVE BENDING
SUMMARY 3RB, 8RB, 15RB AND 20RB 1984

Figure B 1.8 Summary Graph 3, 8, 15, 20RB for 1984
%OCCURRENCE vs STRAIN
SUMMARY 8RT - REVERSE BENDING 1984

Figure B 2.0 Summary Graph 8RT for 1984
%OCCURRENCE vs STRAIN
SUMMARY 15RT - REVERSE BENDING 1984

Figure B 2.2 Summary Graph 15RT for 1984
CUM. PERCENT EXCEEDING vs STRAIN
SUMMARY 15RT - REVERSE BENDING

Figure B 2.3 Cumulative Exceeding Graph for 15RT, 1984
% OCCURRENCE vs STRAIN - REVERSE BENDING
SUMMARY 8RT, 15RT AND 20RT 1984

Figure B 2.6 Summary Graph 8, 15, 20RT for 1984
Cumulative Exceeding Graph for 8, 15, 20RT, 1984
Figure B 3.0 Summary Graph 3RB for 1986
Figure B 3.1 Cumulative Exceeding Graph for 3RB, 1986
Figure B 3.2  Summary Graph 15RB for 1986
Figure B.3.3 Cumulative Exceeding Graph for 15RB, 1986
% OCCURRENCE VS STRAIN – POSITIVE BENDING

SUMMARY 3 RB AND 15 RB 1986

Figure B 3.4 Summary Graph 3, 15RB for 1986
Figure B 3.5 Cumulative Exceeding Graph for 3, 15RB, 1986
Figure B 4.0  Summary Graph 1RB for 1984
CUM. PERCENT EXCEEDING vs STRAIN
SUMMARY 1RB - POSITIVE BENDING 1984

Figure B 4.1 Cumulative Exceeding Graph for 1RB, 1984
%OCURRENCE vs STRAIN
SUMMARY 22RB - POSITIVE BENDING 1984

Figure B 4.2 Summary Graph 22RB for 1984
CUM. PERCENT EXCEEDING vs STRAIN
SUMMARY 22RB - POSITIVE BENDING 1984

Figure B 4.3 Cumulative Exceeding Graph for 22RB, 1984
Figure B 4.4 Summary Graph 1RB for 1986
Figure B 4.7 Cumulative Exceeding Graph for 22RB, 1986
Figure B 5.0 Average Strain Versus Speed for Light Loads
AVERAGE STRAINS vs SPEED
HEAVY WHEEL LOADS

Figure B 5.1 Average Strain Versus Speed for Heavy Loads
Figure B 5.2 Average Impact Factors for Light Loads
Average Impact Factors for Various Speeds

Heavy Wheel Loads

Figure B 5.3 Average Impact Factors for Heavy Loads
APPENDIX C

ANCILLARY CALCULATIONS

USED IN THE MODELS
PERIOD OF TIE TO SPRING UP AND CLOSE GAP

Consider only the central tie under wheel load:

\[ Z = \frac{W L^3}{3EI} \frac{b^2}{L^2} \frac{(L - b)^2}{L^2} + \frac{W L^3}{6EI} \frac{(L - b)}{L} \frac{b^2}{L^2} \frac{b}{L} \left(1 + \frac{L}{b} - \frac{b}{a}\right) \]

\[ Z = \frac{W L^3}{3EI} \frac{b^2}{L^2} \frac{(L - b)}{L} \left\{ \frac{L - b}{L} + \frac{1}{2} \frac{b}{L} \left(1 + \frac{L}{b} - \frac{b}{a}\right) \right\} \]

\[ a = 24" \text{ (610 mm), } b = 18" \text{ (457 mm), } c = 96" \text{ (2,438 mm)} \]

\[ E = 4.42 \times 10^6 \text{ psi (3,048 MPa)} \]

\[ I = 1,560 \text{ in}^4 \text{ (6.49 x } 10^8 \text{ mm}^4) \]

\[ W = \frac{64,000}{8} = 8,000 \text{ lbs (35.6 kN)} \]

\[ Z = \frac{8,000 (96)^2}{3 \times 5 \times 10^6 \times 1,600} \left(\frac{18}{96}\right)^2 \frac{78}{96} \left(\frac{78}{96} + \frac{1}{2} \frac{18}{96} \left(1 + \frac{96}{18} - \frac{18}{24}\right)\right) \]

\[ Z = 0.0131" \text{ (0.3318 mm) for } W = 8,000 \text{ lbs (35.6 kN)} \]

\[ Z = 0.0539" \text{ (1.3688 mm) for } W = 33,000 \text{ lbs (146.8 kN)} \]

The gap "Δ" due to \( d = 1" \text{ (25.4 mm)} "skid" \text{ flat is:} \]

214
\[\Delta - r (1 - \cos \Theta_0)\]

\[\Delta = r \left(1 - \sqrt{1 - (0.5d/r)^2}\right)\]

\[\Delta = 0.00174'' (0.044 \text{ mm})\]

\[\frac{\Delta}{Z_{6,000}} = 13 \text{ percent} \quad \frac{\Delta}{Z_{33,000}} = 3 \text{ percent}\]

Therefore, when tie is vibrating at a fundamental frequency of 85 Hz, its period for a half cycle is 0.0059 sec. Assume a linear relationship between time and distance, then the time to close the gap is:

For a load of 8,000 lbs (35.6 kN) 13 percent of 0.0059 = 0.0008 sec.

For a load of 33,000 lbs (146.8 kN) 3 percent of 0.0059 = 0.0002 sec.

**Evaluate The Attenuation Property of Truck Springs**

![Diagram of truck springs](image)

**Figure C 1.1 Attenuation Properties of Railway Vehicle Springs**

\[\frac{I_0 \Theta^2}{2} + M (\Theta d/2)^2 - Mg \Theta d/2 + \frac{1}{2} k (\Theta d/2)^2\]

Ignoring gravity for dynamic analysis
\[ M \left[ C + \frac{d^2}{4} \right] \Theta^2 = k \frac{d^2}{4} \Theta^2 \]

in harmonic motion \( \Theta = -\omega \Theta \)

Therefore:

\[ \omega^2 = \frac{kd^2/4}{M \left[ C + \frac{d^2}{4} \right]} \]

or

\[ F = \frac{1}{2\pi} \sqrt{\frac{kd^2/4}{I_0 + M \frac{d^2}{4}}} \]

Where \( I \text{ pivot} = I_0 + \frac{Ma^2}{4} \)

\( d = 5'6" = 66" \) (152.4 mm)

\( k = 37,000 \text{ lbs/in.} \) (6.48 kN/mm) (for springs on 1 truck)

\( M = (1 \text{ axle} + 2 \text{ side frames}) = 6.68 + 2 \times 2.98 = 12.64 \text{ slug} \)

\( I \text{ pivot} = M \text{ axle} \times d^2 + I \text{ side frames} + M \text{ side frames} \times (d/2)^2 \)

\( I \text{ pivot} = 6.68 \times 66^2 + 2 \times 1370 + 2 \times 2.98 \times 33^2 \)

\( I \text{ pivot} = 38,328 \text{ slug-in}^2 \)

\[ F = \frac{1}{2\pi} \sqrt{\frac{k \frac{d^2}{4}}{I \text{ pivot}}} = \frac{1}{2\pi} \sqrt{\frac{37,000 \times 66^2/4}{38,328}} \]

\( F = 5.2 \text{ Hz} \) resonant frequency.

Conclusion: for impact forces of period from 2 to 5 m.sec most of the frequency is well above the resonant frequency, and is cut off.
Evaluate Duration of Motion of:

Wheel Fall: Rotation of Flat: Spring Up of Tie

Figure C 1.2 Duration of Motion of Wheel Vertically

A - Duration of Fall of Wheel

in closing gap created by skid flat

d = 1" (25.4 mm)
r = 18" (457.2 mm)

\[ r \left( 1 - \cos \Theta_0 \right) = \frac{1}{2} gt^2; \quad \sin \Theta_0 = \frac{d/2}{r}; \quad \cos \Theta_0 \sqrt{1 - \left( \frac{d/2}{r} \right)^2} \]

\[ t = \sqrt{\frac{2r[1-\sqrt{1 - \left( \frac{d/2}{r} \right)^2}]}{g}} = \sqrt{\frac{2 \times 18 \times [1 - \sqrt{1 - (0.5/18)^2}]}{386.4}} \]

\[ t = 0.0060 \text{ sec} \]
Figure C 1.3 Duration of Motion of Wheel Horizontally

B - Duration of Rotation of Wheel Flat

\[ t = \frac{2 \Theta_0}{\omega} \; ; \; \omega = \frac{V}{r} \]

\[ V = 40 \text{ mph or 720 in/sec} \]

\[ \Theta_0 = \sin^{-1} \left( \frac{d/2}{r} \right) - \left( \frac{0.5}{18} \right) \]

\[ \Theta_0 = 0.02778 \text{ radians} \]

\[ t = \frac{2 \Theta_0 r}{V} = \frac{2 \times 0.02778 \times 18}{720} \]

\[ t = 0.0010 \text{ sec} \]
APPENDIX D

DOCUMENTATION ON ELONGATION
OF WIRE STRANDS

(COURTESY OF MONARCH PRECO LTD.)
Figure D 1.0 Production Drawing of Type 1 Tie
Relève Des Opérations de Mise en Tension

Eléments Type #1 (BEAM#1) T.I.B.

Description des Eléments TIE (BEAMS)

Contrat # Contr. U.

Date 5 Fév-1991

Identification des torons

Torons

<table>
<thead>
<tr>
<th>Diamètre</th>
<th>Fabricant</th>
<th>Résistance maximale</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8&quot;</td>
<td>STELCO</td>
<td>270,000 psi</td>
</tr>
</tbody>
</table>

Capacité du Béton lb/ft²

<table>
<thead>
<tr>
<th>Coupage des torons</th>
<th>28 Jours</th>
</tr>
</thead>
<tbody>
<tr>
<td>G Fév-91</td>
<td>AMTPEL.30.44.HK 5 MAR.-91</td>
</tr>
</tbody>
</table>

Table D 1.0 Elongation Measurements for TIE #1B1
Table D 1.1 Elongation Measurements for Tie T1B2

<table>
<thead>
<tr>
<th>Toron #</th>
<th>Séquence pour la mise en traction</th>
<th>Lecture du monomètre</th>
<th>Tension</th>
<th>Elongation</th>
<th>Séquence pour le coupage</th>
<th>A(\varepsilon) (pp)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>760</td>
<td>152/10</td>
<td>16200</td>
<td>0.8725</td>
<td>7/8</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>760</td>
<td>152/10</td>
<td>16200</td>
<td>0.8725</td>
<td>3/4</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>760</td>
<td>152/10</td>
<td>16200</td>
<td>0.8725</td>
<td>5/16</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>760</td>
<td>152/10</td>
<td>16200</td>
<td>0.8725</td>
<td>3/4</td>
</tr>
<tr>
<td>5</td>
<td>2</td>
<td>760</td>
<td>152/10</td>
<td>16200</td>
<td>0.8725</td>
<td>13/16</td>
</tr>
<tr>
<td>6</td>
<td>3</td>
<td>760</td>
<td>152/10</td>
<td>16200</td>
<td>0.8725</td>
<td>17/16</td>
</tr>
<tr>
<td>7</td>
<td>1</td>
<td>760</td>
<td>152/10</td>
<td>16200</td>
<td>0.8725</td>
<td>7</td>
</tr>
<tr>
<td>8</td>
<td>4</td>
<td>760</td>
<td>152/10</td>
<td>16200</td>
<td>0.8725</td>
<td>9/8</td>
</tr>
<tr>
<td>9</td>
<td>5</td>
<td>760</td>
<td>152/10</td>
<td>16200</td>
<td>0.8725</td>
<td>3/4</td>
</tr>
<tr>
<td>10</td>
<td>4</td>
<td>760</td>
<td>152/10</td>
<td>16200</td>
<td>0.8725</td>
<td>12/12</td>
</tr>
<tr>
<td>11</td>
<td>7</td>
<td>760</td>
<td>152/10</td>
<td>16200</td>
<td>0.8725</td>
<td>13/16</td>
</tr>
<tr>
<td>12</td>
<td>6</td>
<td>760</td>
<td>152/10</td>
<td>16200</td>
<td>0.8725</td>
<td>14/14</td>
</tr>
<tr>
<td>13</td>
<td>6</td>
<td>760</td>
<td>152/10</td>
<td>16200</td>
<td>0.8725</td>
<td>14/14</td>
</tr>
<tr>
<td>14</td>
<td>7</td>
<td>760</td>
<td>152/10</td>
<td>16200</td>
<td>0.8725</td>
<td>14/14</td>
</tr>
</tbody>
</table>

Supervisor: 

Approved by:
### Table D 1.2 Elongation Measurements for Tie T1B3

<table>
<thead>
<tr>
<th>Toron#</th>
<th>Séquence pour la mise en tension</th>
<th>Lecture du monomètre (Lb)</th>
<th>Tension (Lb)</th>
<th>Expé. Finale</th>
<th>Calculée</th>
<th>Mesurée</th>
<th>Séquence pour le coupage</th>
<th>ΔR (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3</td>
<td>760</td>
<td>152.10</td>
<td>16220</td>
<td>0.8725</td>
<td>3/4</td>
<td>11 - 11 - 1/2</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>760</td>
<td>152.10</td>
<td>16220</td>
<td>0.7564</td>
<td>3/4</td>
<td>2 - 2 - 1/2</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>760</td>
<td>152.10</td>
<td>16220</td>
<td>0.7564</td>
<td>3/4</td>
<td>2 - 2 - 1/2</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>760</td>
<td>152.10</td>
<td>16220</td>
<td>0.7564</td>
<td>3/4</td>
<td>2 - 2 - 1/2</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>2</td>
<td>760</td>
<td>152.10</td>
<td>16220</td>
<td>0.7564</td>
<td>3/4</td>
<td>2 - 2 - 1/2</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>3</td>
<td>760</td>
<td>152.10</td>
<td>16220</td>
<td>0.7564</td>
<td>3/4</td>
<td>2 - 2 - 1/2</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>5</td>
<td>760</td>
<td>152.10</td>
<td>16220</td>
<td>0.7564</td>
<td>3/4</td>
<td>2 - 2 - 1/2</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>4</td>
<td>760</td>
<td>152.10</td>
<td>16220</td>
<td>0.7564</td>
<td>3/4</td>
<td>2 - 2 - 1/2</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>5</td>
<td>760</td>
<td>152.10</td>
<td>16220</td>
<td>0.7564</td>
<td>3/4</td>
<td>2 - 2 - 1/2</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>4</td>
<td>760</td>
<td>152.10</td>
<td>16220</td>
<td>0.7564</td>
<td>3/4</td>
<td>2 - 2 - 1/2</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>7</td>
<td>760</td>
<td>152.10</td>
<td>16220</td>
<td>0.7564</td>
<td>3/4</td>
<td>2 - 2 - 1/2</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>6</td>
<td>760</td>
<td>152.10</td>
<td>16220</td>
<td>0.7564</td>
<td>3/4</td>
<td>2 - 2 - 1/2</td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>6</td>
<td>760</td>
<td>152.10</td>
<td>16220</td>
<td>0.7564</td>
<td>3/4</td>
<td>2 - 2 - 1/2</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>7</td>
<td>760</td>
<td>152.10</td>
<td>16220</td>
<td>0.7564</td>
<td>3/4</td>
<td>2 - 2 - 1/2</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>16</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>17</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>18</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>19</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Rélevé Des Opérations de Mise en Tension

**Eléments**

<table>
<thead>
<tr>
<th>TYPE</th>
<th>#1</th>
<th>(Beam #4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1B4</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Description des Eléments**

*Tie (Beams)*

**Identification des torons**

<table>
<thead>
<tr>
<th>Toron</th>
<th>Séquence pour la mise en tension</th>
<th>Lecture du monomètre</th>
<th>Tension</th>
<th>Elongation</th>
<th>Séquence pour le coupage</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Initial</td>
<td>Étroit</td>
<td>Finale</td>
</tr>
<tr>
<td>1</td>
<td>3</td>
<td>760</td>
<td>1521</td>
<td>15220</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>760</td>
<td>1521</td>
<td>15220</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>760</td>
<td>1521</td>
<td>15220</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>760</td>
<td>1521</td>
<td>15220</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>2</td>
<td>760</td>
<td>1521</td>
<td>15220</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>3</td>
<td>760</td>
<td>1521</td>
<td>15220</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>4</td>
<td>760</td>
<td>1521</td>
<td>15220</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>5</td>
<td>760</td>
<td>1521</td>
<td>15220</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>6</td>
<td>760</td>
<td>1521</td>
<td>15220</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>4</td>
<td>760</td>
<td>1521</td>
<td>15220</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>7</td>
<td>760</td>
<td>1521</td>
<td>15220</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>6</td>
<td>760</td>
<td>1521</td>
<td>15220</td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>6</td>
<td>760</td>
<td>1521</td>
<td>15220</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>7</td>
<td>760</td>
<td>1521</td>
<td>15220</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>16</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>17</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>18</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>19</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

**Table D.1.3 Elongation Measurements for Tie T1B4**

---

**Superviseur**

---

**Approuvé par**
Relève Des Opérations de Mise en Tension

<table>
<thead>
<tr>
<th>Éléments</th>
<th>TYPE</th>
<th>BEAM*5</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1B5</td>
<td>T1B5</td>
<td></td>
</tr>
</tbody>
</table>

Contrat n°: CON. U.
Date: 12 FÉV. 1991

Description des Éléments: TIE (BEAMS)

Identification des torons

<table>
<thead>
<tr>
<th>Toron</th>
<th>Séquence pour la mise en tension</th>
<th>Lecture du manomètre</th>
<th>Tension</th>
<th>Elongation</th>
<th>Séquence pour le coupage</th>
<th>ΔR</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3</td>
<td>760</td>
<td>15210 15220</td>
<td>0.8725 0.8725</td>
<td>1/16 1/16</td>
<td>11 11</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>760</td>
<td>15210 15220</td>
<td>0.8725 0.8725</td>
<td>2/32 1/16</td>
<td>2 2</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>760</td>
<td>15210 15220</td>
<td>0.8725 0.8725</td>
<td>5/32 1/16</td>
<td>5 5</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>760</td>
<td>15210 15220</td>
<td>0.8725 0.8725</td>
<td>7/32 1/16</td>
<td>7 7</td>
</tr>
<tr>
<td>5</td>
<td>2</td>
<td>760</td>
<td>15210 15220</td>
<td>0.8725 0.8725</td>
<td>2/8 1/8</td>
<td>8 8</td>
</tr>
<tr>
<td>6</td>
<td>3</td>
<td>760</td>
<td>15210 15220</td>
<td>0.8725 0.8725</td>
<td>2/8 1/8</td>
<td>8 8</td>
</tr>
<tr>
<td>7</td>
<td>5</td>
<td>760</td>
<td>15210 15220</td>
<td>0.8725 0.8725</td>
<td>7/32 1/8</td>
<td>7 7</td>
</tr>
<tr>
<td>8</td>
<td>4</td>
<td>760</td>
<td>15210 15220</td>
<td>0.8725 0.8725</td>
<td>9/32 1/8</td>
<td>9 9</td>
</tr>
<tr>
<td>9</td>
<td>5</td>
<td>760</td>
<td>15210 15220</td>
<td>0.8725 0.8725</td>
<td>2/8 1/8</td>
<td>8 8</td>
</tr>
<tr>
<td>10</td>
<td>4</td>
<td>760</td>
<td>15210 15220</td>
<td>0.8725 0.8725</td>
<td>13/16 1/8</td>
<td>12 12</td>
</tr>
<tr>
<td>11</td>
<td>6</td>
<td>760</td>
<td>15210 15220</td>
<td>0.8725 0.8725</td>
<td>7/8 7/8</td>
<td>11 11</td>
</tr>
<tr>
<td>12</td>
<td>7</td>
<td>760</td>
<td>15210 15220</td>
<td>0.8725 0.8725</td>
<td>7/8 7/8</td>
<td>11 11</td>
</tr>
<tr>
<td>13</td>
<td>7</td>
<td>760</td>
<td>15210 15220</td>
<td>0.8725 0.8725</td>
<td>7/8 7/8</td>
<td>11 11</td>
</tr>
<tr>
<td>14</td>
<td>6</td>
<td>760</td>
<td>15210 15220</td>
<td>0.8725 0.8725</td>
<td>13/16 1/8</td>
<td>14 14</td>
</tr>
</tbody>
</table>

L = 159.25 m

Table D 1.4 Elongation Measurements for Tie T1B5

225
### Référence Des Opérations de Mise en Tension

**Eléments # TYPE #**

**T.B.**

**Description des Eléments**

**Tie (BEAMS)**

**Contrat #**

**CON. U.**

**Date**

**13 FÉV -1991**

---

#### Identification des torons

<table>
<thead>
<tr>
<th>Diamètre</th>
<th>Fabriquant</th>
<th>Résistance maximale</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8&quot;</td>
<td>STELCO</td>
<td>270,000 psi</td>
</tr>
</tbody>
</table>

#### Capacité du Béton lb/ft³

<table>
<thead>
<tr>
<th>Coupage des torons</th>
<th>28 Jours</th>
</tr>
</thead>
<tbody>
<tr>
<td>14 FÉV. / 4138 Poi</td>
<td>13 MAR. / 1438 Poi</td>
</tr>
</tbody>
</table>

#### Torons

| Toron # | Séquence pour la mise en tension | Lecture du monomètre | Tension | Elongation | Séquence pour le coupage |Ar
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>760</td>
<td>15210</td>
<td>16000</td>
<td>0.9195</td>
</tr>
<tr>
<td>2</td>
<td>3</td>
<td>760</td>
<td>15210</td>
<td>16000</td>
<td>0.9195</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>760</td>
<td>15210</td>
<td>16200</td>
<td>0.9172</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>760</td>
<td>15210</td>
<td>16200</td>
<td>0.9172</td>
</tr>
<tr>
<td>5</td>
<td>3</td>
<td>760</td>
<td>15210</td>
<td>16000</td>
<td>0.9195</td>
</tr>
<tr>
<td>6</td>
<td>2</td>
<td>760</td>
<td>15210</td>
<td>16000</td>
<td>0.9195</td>
</tr>
<tr>
<td>7</td>
<td>5</td>
<td>760</td>
<td>15210</td>
<td>16000</td>
<td>0.9195</td>
</tr>
<tr>
<td>8</td>
<td>4</td>
<td>760</td>
<td>15210</td>
<td>16200</td>
<td>0.9172</td>
</tr>
<tr>
<td>9</td>
<td>5</td>
<td>760</td>
<td>15210</td>
<td>16000</td>
<td>0.9195</td>
</tr>
<tr>
<td>10</td>
<td>4</td>
<td>760</td>
<td>15210</td>
<td>16200</td>
<td>0.9172</td>
</tr>
<tr>
<td>11</td>
<td>7</td>
<td>760</td>
<td>15210</td>
<td>16000</td>
<td>0.9195</td>
</tr>
<tr>
<td>12</td>
<td>6</td>
<td>760</td>
<td>15210</td>
<td>16200</td>
<td>0.9172</td>
</tr>
<tr>
<td>13</td>
<td>6</td>
<td>760</td>
<td>15210</td>
<td>16000</td>
<td>0.9195</td>
</tr>
<tr>
<td>14</td>
<td>7</td>
<td>760</td>
<td>15210</td>
<td>16000</td>
<td>0.9195</td>
</tr>
</tbody>
</table>

Superviseur: [Signature]

L = 159.25 in.

Approuvé par:

---

Table D 1.5 Elongation Measurements for Tie T1B6
Tie Type No.2 (Eccentric Prestress)

1) 16-3/8" dia. 7 wire strand. With a prestressing force of 14,021 lbs each.

2) 14,021 lbs = 0.70 f_{Pu} A_g

3) Total Prestressing force = 256,337 lbs.

4) Effective Prestressing force = 187,126 lbs.

5) Stresses in concrete after transfer:
   \( f_t = 1000 \text{ psi compression} \)
   \( f_b = 1850 \text{ psi compression} \)

6) Eccentricity (e) = 0.59 in.

Figure D 1.1 Production Drawing of Type 2 Tie
Table D 2.0 Elogation Measurements for Tie T2B1

<table>
<thead>
<tr>
<th>Passage</th>
<th>Tension</th>
<th>Elongation</th>
<th>Sequence for the coupon</th>
<th>Load After Cutt</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/16</td>
<td>1</td>
<td>1</td>
<td>1000 (6021)</td>
<td>1/2</td>
</tr>
<tr>
<td>3/16</td>
<td>2</td>
<td>2</td>
<td>1000 (6021)</td>
<td>1/2</td>
</tr>
<tr>
<td>3/16</td>
<td>3</td>
<td>3</td>
<td>1000 (6021)</td>
<td>1/2</td>
</tr>
<tr>
<td>3/16</td>
<td>4</td>
<td>4</td>
<td>1000 (6021)</td>
<td>1/2</td>
</tr>
<tr>
<td>3/16</td>
<td>5</td>
<td>2</td>
<td>1000 (6021)</td>
<td>3/16</td>
</tr>
<tr>
<td>5/32</td>
<td>6</td>
<td>1</td>
<td>1000 (6021)</td>
<td>2</td>
</tr>
<tr>
<td>5/32</td>
<td>7</td>
<td>4</td>
<td>1000 (6021)</td>
<td>7</td>
</tr>
<tr>
<td>5/32</td>
<td>8</td>
<td>5</td>
<td>1000 (6021)</td>
<td>7</td>
</tr>
<tr>
<td>5/32</td>
<td>9</td>
<td>4</td>
<td>1000 (6021)</td>
<td>8</td>
</tr>
<tr>
<td>7/32</td>
<td>10</td>
<td>5</td>
<td>1000 (6021)</td>
<td>10</td>
</tr>
<tr>
<td>7/32</td>
<td>11</td>
<td>6</td>
<td>1000 (6021)</td>
<td>11</td>
</tr>
<tr>
<td>7/32</td>
<td>12</td>
<td>7</td>
<td>1000 (6021)</td>
<td>13</td>
</tr>
<tr>
<td>7/32</td>
<td>13</td>
<td>7</td>
<td>1000 (6021)</td>
<td>12</td>
</tr>
<tr>
<td>7/32</td>
<td>14</td>
<td>6</td>
<td>1000 (6021)</td>
<td>14</td>
</tr>
<tr>
<td>7/32</td>
<td>15</td>
<td>8</td>
<td>1000 (6021)</td>
<td>15</td>
</tr>
<tr>
<td>7/32</td>
<td>16</td>
<td>9</td>
<td>1000 (6021)</td>
<td>16</td>
</tr>
</tbody>
</table>

Supervisor: 
Approved by: 

Contact Contr. 4.
Date 15 Fév. 1991
Table D 2.1 Elogation Measurements for Tie T2B2
# Table D 2.2  Elogation Measurements for Tie T2B3

<table>
<thead>
<tr>
<th>Toron#</th>
<th>Séquence pour la mise en tension</th>
<th>Lecture du manomètre</th>
<th>Tension</th>
<th>Elongation</th>
<th>Séquence pour le coupage</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/8</td>
<td>1</td>
<td>1000</td>
<td>16021</td>
<td>9000</td>
<td>0.906</td>
</tr>
<tr>
<td>9/64</td>
<td>2</td>
<td>1000</td>
<td>16021</td>
<td>9000</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>1000</td>
<td>16021</td>
<td>9000</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>3</td>
<td>1000</td>
<td>16021</td>
<td>9000</td>
<td>0</td>
</tr>
<tr>
<td>1/4</td>
<td>2</td>
<td>1000</td>
<td>16021</td>
<td>9000</td>
<td>0</td>
</tr>
<tr>
<td>5/64</td>
<td>2</td>
<td>1000</td>
<td>16021</td>
<td>9000</td>
<td>0</td>
</tr>
<tr>
<td>7/64</td>
<td>4</td>
<td>1000</td>
<td>16021</td>
<td>9000</td>
<td>0</td>
</tr>
<tr>
<td>8</td>
<td>5</td>
<td>1000</td>
<td>16021</td>
<td>9000</td>
<td>0</td>
</tr>
<tr>
<td>9/64</td>
<td>4</td>
<td>1000</td>
<td>16021</td>
<td>9000</td>
<td>0</td>
</tr>
<tr>
<td>10</td>
<td>5</td>
<td>1000</td>
<td>16021</td>
<td>9000</td>
<td>0</td>
</tr>
<tr>
<td>11</td>
<td>6</td>
<td>1000</td>
<td>16021</td>
<td>9000</td>
<td>0</td>
</tr>
<tr>
<td>12</td>
<td>7</td>
<td>1000</td>
<td>16021</td>
<td>9000</td>
<td>0</td>
</tr>
<tr>
<td>13</td>
<td>7</td>
<td>1000</td>
<td>16021</td>
<td>9000</td>
<td>0</td>
</tr>
<tr>
<td>14</td>
<td>6</td>
<td>1000</td>
<td>16021</td>
<td>9000</td>
<td>0</td>
</tr>
<tr>
<td>15</td>
<td>8</td>
<td>1000</td>
<td>16021</td>
<td>9000</td>
<td>0</td>
</tr>
<tr>
<td>16</td>
<td>9</td>
<td>1000</td>
<td>16021</td>
<td>9000</td>
<td>0</td>
</tr>
</tbody>
</table>

Superviseur

Approuvé par
### Table D 2.3 Elogation Measurements for Tie T2B4

<table>
<thead>
<tr>
<th>Toron#</th>
<th>Sequence pour la mise en tension</th>
<th>Lecture du manomètre (lb)</th>
<th>Tension (lb)</th>
<th>Elongation (%)</th>
<th>Sequence pour le countdown</th>
<th>dx (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/8</td>
<td>1</td>
<td>0.00</td>
<td>0.9063</td>
<td>1/8</td>
<td>1/8</td>
<td>1/8</td>
</tr>
<tr>
<td>3/32</td>
<td>2</td>
<td>0.00</td>
<td>0.9063</td>
<td>1/8</td>
<td>3/32</td>
<td>3/32</td>
</tr>
<tr>
<td>5/32</td>
<td>3</td>
<td>0.00</td>
<td>0.9063</td>
<td>1/8</td>
<td>5/32</td>
<td>5/32</td>
</tr>
<tr>
<td>7/32</td>
<td>4</td>
<td>0.00</td>
<td>0.9063</td>
<td>1/8</td>
<td>7/32</td>
<td>7/32</td>
</tr>
<tr>
<td>1/4</td>
<td>5</td>
<td>0.00</td>
<td>0.9063</td>
<td>1/8</td>
<td>1/4</td>
<td>1/4</td>
</tr>
<tr>
<td>9/32</td>
<td>6</td>
<td>0.00</td>
<td>0.9063</td>
<td>1/8</td>
<td>9/32</td>
<td>9/32</td>
</tr>
<tr>
<td>13/32</td>
<td>7</td>
<td>0.00</td>
<td>0.9063</td>
<td>1/8</td>
<td>13/32</td>
<td>13/32</td>
</tr>
<tr>
<td>15/32</td>
<td>8</td>
<td>0.00</td>
<td>0.9063</td>
<td>1/8</td>
<td>15/32</td>
<td>15/32</td>
</tr>
<tr>
<td>1/2</td>
<td>9</td>
<td>0.00</td>
<td>0.9063</td>
<td>1/8</td>
<td>1/2</td>
<td>1/2</td>
</tr>
</tbody>
</table>

---

Table D 2.3 Elogation Measurements for Tie T2B4

231
<table>
<thead>
<tr>
<th>Torons</th>
<th>Séquence pour la mise en tension</th>
<th>Lecture du manomètre</th>
<th>(lb) Tension</th>
<th>(in) Elongation</th>
<th>Calculée pour le coupage</th>
<th>Séquence pour le coupage</th>
<th>ARG</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4</td>
<td>1</td>
<td>1000 [600] 9000</td>
<td>1322 0.9063</td>
<td>1.8 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1</td>
</tr>
<tr>
<td>3/4</td>
<td>2</td>
<td>1000 [600] 9000</td>
<td>1322 0.9063</td>
<td>1.8 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1</td>
</tr>
<tr>
<td>3/16</td>
<td>3</td>
<td>1000 [600] 9000</td>
<td>1322 0.9063</td>
<td>1.8 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1</td>
</tr>
<tr>
<td>3/16</td>
<td>4</td>
<td>1000 [600] 9000</td>
<td>1322 0.9063</td>
<td>1.8 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1</td>
</tr>
<tr>
<td>1/4</td>
<td>5</td>
<td>1000 [600] 9000</td>
<td>1322 0.9063</td>
<td>1.8 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.5</td>
</tr>
<tr>
<td>3/16</td>
<td>6</td>
<td>1000 [600] 9000</td>
<td>1322 0.9063</td>
<td>1.8 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.5</td>
</tr>
<tr>
<td>3/16</td>
<td>7</td>
<td>1000 [600] 9000</td>
<td>1322 0.9063</td>
<td>1.8 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.5</td>
</tr>
<tr>
<td>3/16</td>
<td>8</td>
<td>1000 [600] 9000</td>
<td>1322 0.9063</td>
<td>1.8 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.5</td>
</tr>
<tr>
<td>7/16</td>
<td>9</td>
<td>1000 [600] 9000</td>
<td>1322 0.9063</td>
<td>1.8 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.5</td>
</tr>
<tr>
<td>7/16</td>
<td>10</td>
<td>1000 [600] 9000</td>
<td>1322 0.9063</td>
<td>1.8 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.5</td>
</tr>
<tr>
<td>7/32</td>
<td>11</td>
<td>1000 [600] 9000</td>
<td>1322 0.9063</td>
<td>1.8 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.5</td>
</tr>
<tr>
<td>7/32</td>
<td>12</td>
<td>1000 [600] 9000</td>
<td>1322 0.9063</td>
<td>1.8 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.5</td>
</tr>
<tr>
<td>7/32</td>
<td>13</td>
<td>1000 [600] 9000</td>
<td>1322 0.9063</td>
<td>1.8 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.5</td>
</tr>
<tr>
<td>7/32</td>
<td>14</td>
<td>1000 [600] 9000</td>
<td>1322 0.9063</td>
<td>1.8 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.5</td>
</tr>
<tr>
<td>7/32</td>
<td>15</td>
<td>1000 [600] 9000</td>
<td>1322 0.9063</td>
<td>1.8 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.5</td>
</tr>
<tr>
<td>7/32</td>
<td>16</td>
<td>1000 [600] 9000</td>
<td>1322 0.9063</td>
<td>1.8 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.5</td>
</tr>
<tr>
<td>17</td>
<td></td>
<td>1000 [600] 9000</td>
<td>1322 0.9063</td>
<td>1.8 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.5</td>
</tr>
<tr>
<td>18</td>
<td></td>
<td>1000 [600] 9000</td>
<td>1322 0.9063</td>
<td>1.8 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.5</td>
</tr>
<tr>
<td>19</td>
<td></td>
<td>1000 [600] 9000</td>
<td>1322 0.9063</td>
<td>1.8 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.5</td>
</tr>
<tr>
<td>20</td>
<td></td>
<td>1000 [600] 9000</td>
<td>1322 0.9063</td>
<td>1.8 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.8 1 1 1 1 1 1 1 1 1 1</td>
<td>1.5</td>
</tr>
</tbody>
</table>

Table D 2.4 Elongation Measurements for Tie T2B5

232
### Table D 2.5 Elogation Measurements for Tie T2B6

<table>
<thead>
<tr>
<th>Toron</th>
<th>Séquence pour la mise en tension</th>
<th>Lecture du manomètre (lb)</th>
<th>Tension (lb)</th>
<th>(in) Elongation</th>
<th>Séquence pour le coupage</th>
<th>A5 (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8</td>
<td>1</td>
<td>1000 G021 9000</td>
<td>1000 G021 9000</td>
<td>1.90 0.23 L/32 1 1-1</td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>5/32</td>
<td>2</td>
<td>1000 G021 9000</td>
<td>1000 G021 9000</td>
<td>1/32 3 2-2</td>
<td></td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>1000 G021 9000</td>
<td>1000 G021 9000</td>
<td>1/32 5 3-3</td>
<td></td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>1000 G021 9000</td>
<td>1000 G021 9000</td>
<td>1/32 6 4-4</td>
<td></td>
<td>4</td>
</tr>
<tr>
<td>5/32</td>
<td>5</td>
<td>1000 G021 9000</td>
<td>1000 G021 9000</td>
<td>1/32 4 5-5</td>
<td></td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>1000 G021 9000</td>
<td>1000 G021 9000</td>
<td>1/32 2 6-6</td>
<td></td>
<td>6</td>
</tr>
<tr>
<td>3/16</td>
<td>7</td>
<td>1000 G021 9000</td>
<td>1000 G021 9000</td>
<td>1/32 7 7-7</td>
<td></td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>1000 G021 9000</td>
<td>1000 G021 9000</td>
<td>3/32 9 8-8</td>
<td></td>
<td>8</td>
</tr>
<tr>
<td>1/4</td>
<td>9</td>
<td>1000 G021 9000</td>
<td>1000 G021 9000</td>
<td>1/32 8 9-9</td>
<td></td>
<td>9</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>1000 G021 9000</td>
<td>1000 G021 9000</td>
<td>1/32 10 10-10</td>
<td></td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>11</td>
<td>1000 G021 9000</td>
<td>1000 G021 9000</td>
<td>1/32 11 11-11</td>
<td></td>
<td>11</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>1000 G021 9000</td>
<td>1000 G021 9000</td>
<td>3/32 13 12-12</td>
<td></td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>13</td>
<td>1000 G021 9000</td>
<td>1000 G021 9000</td>
<td>3/32 12 13-13</td>
<td></td>
<td>13</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>1000 G021 9000</td>
<td>1000 G021 9000</td>
<td>3/52 14 14-14</td>
<td></td>
<td>14</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>1000 G021 9000</td>
<td>1000 G021 9000</td>
<td>1/32 15 15-15</td>
<td></td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>1000 G021 9000</td>
<td>1000 G021 9000</td>
<td>1/32 16 16-16</td>
<td></td>
<td>16</td>
</tr>
<tr>
<td>5/32</td>
<td>17</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>18</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>19</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure D 1.2  Production Drawing of Type 3 Tie
Table D 3.0 Elongation Measurements for Tie T3B1

<table>
<thead>
<tr>
<th>Toron</th>
<th>Série pour la mise en tension</th>
<th>Lecture du manomètre</th>
<th>Tension</th>
<th>Elongation</th>
<th>Séquence pour le coupage</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>0.9358</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>3/32</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>1/32</td>
</tr>
<tr>
<td>4</td>
<td>4</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>1/32</td>
</tr>
<tr>
<td>5</td>
<td>5</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>1/32</td>
</tr>
<tr>
<td>6</td>
<td>6</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>1/32</td>
</tr>
<tr>
<td>7</td>
<td>7</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>3/32</td>
</tr>
<tr>
<td>8</td>
<td>8</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>3/32</td>
</tr>
<tr>
<td>9</td>
<td>9</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>3/32</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>3/32</td>
</tr>
<tr>
<td>11</td>
<td>11</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>3/32</td>
</tr>
<tr>
<td>12</td>
<td>12</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>3/32</td>
</tr>
<tr>
<td>13</td>
<td>13</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>3/32</td>
</tr>
<tr>
<td>14</td>
<td>14</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>3/32</td>
</tr>
<tr>
<td>15</td>
<td>15</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>3/32</td>
</tr>
<tr>
<td>16</td>
<td>16</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>3/32</td>
</tr>
<tr>
<td>17</td>
<td>17</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>3/32</td>
</tr>
<tr>
<td>18</td>
<td>18</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>3/32</td>
</tr>
<tr>
<td>19</td>
<td>19</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>3/32</td>
</tr>
<tr>
<td>20</td>
<td>20</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>3/32</td>
</tr>
</tbody>
</table>

Note: T of 16510b increased to 19400 due to TUBE SUP LOUS 5/32
### Table D 3.1 Elongation Measurements for Tie T3B2

<table>
<thead>
<tr>
<th>Toron#</th>
<th>Série en tension pour le mesure</th>
<th>Lecture du manomètre</th>
<th>Tension</th>
<th>Elongation</th>
<th>Série pour le coupure</th>
<th>Séquence pour le coupure</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/2</td>
<td>7</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>0.9358</td>
<td>3/32</td>
</tr>
<tr>
<td>7/16</td>
<td>8</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>0.9358</td>
<td>3/32</td>
</tr>
<tr>
<td>3/8</td>
<td>7</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>0.9358</td>
<td>3/32</td>
</tr>
<tr>
<td>5/32</td>
<td>8</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>0.9358</td>
<td>3/32</td>
</tr>
<tr>
<td>5/32</td>
<td>2</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>0.9358</td>
<td>3/32</td>
</tr>
<tr>
<td>7/32</td>
<td>5</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>0.9358</td>
<td>3/32</td>
</tr>
<tr>
<td>7/32</td>
<td>6</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>0.9358</td>
<td>3/32</td>
</tr>
<tr>
<td>8/32</td>
<td>2</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>0.9358</td>
<td>3/32</td>
</tr>
<tr>
<td>9/32</td>
<td>1</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>0.9358</td>
<td>3/32</td>
</tr>
<tr>
<td>9/32</td>
<td>3</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>0.9358</td>
<td>3/32</td>
</tr>
<tr>
<td>11/32</td>
<td>4</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>0.9358</td>
<td>3/32</td>
</tr>
<tr>
<td>12/32</td>
<td>1</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>0.9358</td>
<td>3/32</td>
</tr>
<tr>
<td>13/32</td>
<td>2</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>0.9358</td>
<td>3/32</td>
</tr>
<tr>
<td>14/32</td>
<td>6</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>0.9358</td>
<td>3/32</td>
</tr>
<tr>
<td>15/32</td>
<td>5</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>0.9358</td>
<td>3/32</td>
</tr>
<tr>
<td>16/32</td>
<td>2</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>0.9358</td>
<td>3/32</td>
</tr>
<tr>
<td>17/32</td>
<td>1</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>0.9358</td>
<td>3/32</td>
</tr>
<tr>
<td>18/32</td>
<td>4</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>0.9358</td>
<td>3/32</td>
</tr>
<tr>
<td>19/32</td>
<td>3</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>0.9358</td>
<td>3/32</td>
</tr>
<tr>
<td>20/32</td>
<td>7</td>
<td>1000</td>
<td>16510</td>
<td>19400</td>
<td>0.9358</td>
<td>3/32</td>
</tr>
</tbody>
</table>
### Elongation Measurements for Tie T3B3

<table>
<thead>
<tr>
<th>Toron #</th>
<th>Séquence pour la mise en tension</th>
<th>Lecture du monomètre</th>
<th>Tension</th>
<th>Elongation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Initiale</td>
<td>Exige Finale</td>
<td>Calculée</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>1000</td>
<td>10510</td>
<td>11010</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>1000</td>
<td>10510</td>
<td>11010</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>1000</td>
<td>10510</td>
<td>11010</td>
</tr>
<tr>
<td>4</td>
<td>4</td>
<td>1000</td>
<td>10510</td>
<td>11010</td>
</tr>
<tr>
<td>5</td>
<td>5</td>
<td>1000</td>
<td>10510</td>
<td>11010</td>
</tr>
<tr>
<td>6</td>
<td>6</td>
<td>1000</td>
<td>10510</td>
<td>11010</td>
</tr>
<tr>
<td>7</td>
<td>7</td>
<td>1000</td>
<td>10510</td>
<td>11010</td>
</tr>
<tr>
<td>8</td>
<td>8</td>
<td>1000</td>
<td>10510</td>
<td>11010</td>
</tr>
<tr>
<td>9</td>
<td>9</td>
<td>1000</td>
<td>10510</td>
<td>11010</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
<td>1000</td>
<td>10510</td>
<td>11010</td>
</tr>
<tr>
<td>11</td>
<td>11</td>
<td>1000</td>
<td>10510</td>
<td>11010</td>
</tr>
<tr>
<td>12</td>
<td>12</td>
<td>1000</td>
<td>10510</td>
<td>11010</td>
</tr>
<tr>
<td>13</td>
<td>13</td>
<td>1000</td>
<td>10510</td>
<td>11010</td>
</tr>
<tr>
<td>14</td>
<td>14</td>
<td>1000</td>
<td>10510</td>
<td>11010</td>
</tr>
<tr>
<td>15</td>
<td>15</td>
<td>1000</td>
<td>10510</td>
<td>11010</td>
</tr>
<tr>
<td>16</td>
<td>16</td>
<td>1000</td>
<td>10510</td>
<td>11010</td>
</tr>
<tr>
<td>17</td>
<td>17</td>
<td>1000</td>
<td>10510</td>
<td>11010</td>
</tr>
<tr>
<td>18</td>
<td>18</td>
<td>1000</td>
<td>10510</td>
<td>11010</td>
</tr>
<tr>
<td>19</td>
<td>19</td>
<td>1000</td>
<td>10510</td>
<td>11010</td>
</tr>
<tr>
<td>20</td>
<td>20</td>
<td>1000</td>
<td>10510</td>
<td>11010</td>
</tr>
</tbody>
</table>

### Table D 3.2

<table>
<thead>
<tr>
<th>Identification des torons</th>
</tr>
</thead>
</table>

*Approved by* [Signature]

---

*Supervisor*
Table D 3.3 Elongation Measurements for Tie T3B4
<table>
<thead>
<tr>
<th>Toron#</th>
<th>Séquence pour le mélange en tension</th>
<th>Lecture du monomètre</th>
<th>Tension initiale</th>
<th>Tension finale</th>
<th>Élongation calculée</th>
<th>Élongation ligne 1</th>
<th>Suivi pour le coupage</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/8</td>
<td>1</td>
<td>1000</td>
<td>16510</td>
<td>14588</td>
<td>0.928</td>
<td>1/8</td>
<td>1</td>
</tr>
<tr>
<td>1/8</td>
<td>2</td>
<td>1000</td>
<td>16510</td>
<td>14588</td>
<td>0.928</td>
<td>1/8</td>
<td>1</td>
</tr>
<tr>
<td>1/8</td>
<td>3</td>
<td>1000</td>
<td>16510</td>
<td>14588</td>
<td>0.928</td>
<td>1/8</td>
<td>1</td>
</tr>
<tr>
<td>1/8</td>
<td>4</td>
<td>1000</td>
<td>16510</td>
<td>14588</td>
<td>0.928</td>
<td>1/8</td>
<td>1</td>
</tr>
<tr>
<td>1/8</td>
<td>5</td>
<td>1000</td>
<td>16510</td>
<td>14588</td>
<td>0.928</td>
<td>1/8</td>
<td>1</td>
</tr>
<tr>
<td>1/8</td>
<td>6</td>
<td>1000</td>
<td>16510</td>
<td>14588</td>
<td>0.928</td>
<td>1/8</td>
<td>1</td>
</tr>
<tr>
<td>1/8</td>
<td>7</td>
<td>1000</td>
<td>16510</td>
<td>14588</td>
<td>0.928</td>
<td>1/8</td>
<td>1</td>
</tr>
<tr>
<td>1/8</td>
<td>8</td>
<td>1000</td>
<td>16510</td>
<td>14588</td>
<td>0.928</td>
<td>1/8</td>
<td>1</td>
</tr>
<tr>
<td>1/8</td>
<td>9</td>
<td>1000</td>
<td>16510</td>
<td>14588</td>
<td>0.928</td>
<td>1/8</td>
<td>1</td>
</tr>
<tr>
<td>1/8</td>
<td>10</td>
<td>1000</td>
<td>16510</td>
<td>14588</td>
<td>0.928</td>
<td>1/8</td>
<td>1</td>
</tr>
<tr>
<td>1/8</td>
<td>11</td>
<td>1000</td>
<td>16510</td>
<td>14588</td>
<td>0.928</td>
<td>1/8</td>
<td>1</td>
</tr>
<tr>
<td>1/8</td>
<td>12</td>
<td>1000</td>
<td>16510</td>
<td>14588</td>
<td>0.928</td>
<td>1/8</td>
<td>1</td>
</tr>
<tr>
<td>1/8</td>
<td>13</td>
<td>1000</td>
<td>16510</td>
<td>14588</td>
<td>0.928</td>
<td>1/8</td>
<td>1</td>
</tr>
<tr>
<td>1/8</td>
<td>14</td>
<td>1000</td>
<td>16510</td>
<td>14588</td>
<td>0.928</td>
<td>1/8</td>
<td>1</td>
</tr>
<tr>
<td>1/8</td>
<td>15</td>
<td>1000</td>
<td>16510</td>
<td>14588</td>
<td>0.928</td>
<td>1/8</td>
<td>1</td>
</tr>
<tr>
<td>1/8</td>
<td>16</td>
<td>1000</td>
<td>16510</td>
<td>14588</td>
<td>0.928</td>
<td>1/8</td>
<td>1</td>
</tr>
<tr>
<td>1/8</td>
<td>17</td>
<td>1000</td>
<td>16510</td>
<td>14588</td>
<td>0.928</td>
<td>1/8</td>
<td>1</td>
</tr>
<tr>
<td>1/8</td>
<td>18</td>
<td>1000</td>
<td>16510</td>
<td>14588</td>
<td>0.928</td>
<td>1/8</td>
<td>1</td>
</tr>
<tr>
<td>1/8</td>
<td>19</td>
<td>1000</td>
<td>16510</td>
<td>14588</td>
<td>0.928</td>
<td>1/8</td>
<td>1</td>
</tr>
<tr>
<td>1/8</td>
<td>20</td>
<td>1000</td>
<td>16510</td>
<td>14588</td>
<td>0.928</td>
<td>1/8</td>
<td>1</td>
</tr>
</tbody>
</table>

Table 3.4 Elongation Measurements for Tie T3B5
### Table D 3.5 Elongation Measurements for Tie T3B6

<table>
<thead>
<tr>
<th>Toron#</th>
<th>Séquence pour la mise en tension</th>
<th>Lecture du manomètre</th>
<th>Tension</th>
<th>Elongation</th>
<th>Séquence pour le coupure</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7</td>
<td>1000</td>
<td>4510</td>
<td>0.926</td>
<td>2/52 1 11 11</td>
</tr>
<tr>
<td>2</td>
<td>8</td>
<td>1000</td>
<td>4510</td>
<td>0</td>
<td>4/52 1 11 11</td>
</tr>
<tr>
<td>3</td>
<td>7</td>
<td>1000</td>
<td>4510</td>
<td>0</td>
<td>4/52 1 11 11</td>
</tr>
<tr>
<td>4</td>
<td>8</td>
<td>1000</td>
<td>4510</td>
<td>0</td>
<td>4/52 1 11 11</td>
</tr>
<tr>
<td>5</td>
<td>2</td>
<td>1000</td>
<td>4510</td>
<td>0</td>
<td>4/52 1 11 11</td>
</tr>
<tr>
<td>6</td>
<td>5</td>
<td>1000</td>
<td>4510</td>
<td>0</td>
<td>4/52 1 11 11</td>
</tr>
<tr>
<td>7</td>
<td>6</td>
<td>1000</td>
<td>4510</td>
<td>0</td>
<td>4/52 1 11 11</td>
</tr>
<tr>
<td>8</td>
<td>2</td>
<td>1000</td>
<td>4510</td>
<td>0</td>
<td>4/52 1 11 11</td>
</tr>
<tr>
<td>9</td>
<td>1</td>
<td>1000</td>
<td>4510</td>
<td>0</td>
<td>4/52 1 11 11</td>
</tr>
<tr>
<td>10</td>
<td>3</td>
<td>1000</td>
<td>4510</td>
<td>0</td>
<td>4/52 1 11 11</td>
</tr>
<tr>
<td>11</td>
<td>4</td>
<td>1000</td>
<td>4510</td>
<td>0</td>
<td>4/52 1 11 11</td>
</tr>
<tr>
<td>12</td>
<td>1</td>
<td>1000</td>
<td>4510</td>
<td>0</td>
<td>4/52 1 11 11</td>
</tr>
<tr>
<td>13</td>
<td>2</td>
<td>1000</td>
<td>4510</td>
<td>0</td>
<td>4/52 1 11 11</td>
</tr>
<tr>
<td>14</td>
<td>6</td>
<td>1000</td>
<td>4510</td>
<td>0</td>
<td>4/52 1 11 11</td>
</tr>
<tr>
<td>15</td>
<td>5</td>
<td>1000</td>
<td>4510</td>
<td>0</td>
<td>4/52 1 11 11</td>
</tr>
<tr>
<td>16</td>
<td>2</td>
<td>1000</td>
<td>4510</td>
<td>0</td>
<td>4/52 1 11 11</td>
</tr>
<tr>
<td>17</td>
<td>1</td>
<td>1000</td>
<td>4510</td>
<td>0</td>
<td>4/52 1 11 11</td>
</tr>
<tr>
<td>18</td>
<td>4</td>
<td>1000</td>
<td>4510</td>
<td>0</td>
<td>4/52 1 11 11</td>
</tr>
<tr>
<td>19</td>
<td>3</td>
<td>1000</td>
<td>4510</td>
<td>0</td>
<td>4/52 1 11 11</td>
</tr>
<tr>
<td>20</td>
<td>7</td>
<td>1000</td>
<td>4510</td>
<td>0</td>
<td>4/52 1 11 11</td>
</tr>
</tbody>
</table>
APPENDIX E

PHOTOGRAPHS OF FAILURE MODE
OF LABORATORY TESTED TIES
Figure E 1.0 Photographs of Failure Mode of Tie T1B4
Figure E 1.1 Photographs of Failure Mode of Tie T2B1
Figure E 1.2 Photograph of Failure Mode of Tie T282
Figure E 1.3 Photographs of Failure Mode of Tie T2B3