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Development Of Precast Barrier Wall System For Bridge Decks

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DEVELOPMENT OF PRECAST BARRIER WALL SYSTEM FOR BRIDGE DECKS

By

Gaurang Patel

B.Eng. (Civil), Bhavnagar University, India, 1998

A Thesis

Presented To Ryerson University

In Partial Fulfillment Of The
Requirements For The Degree Of
Master Of Applied Science

In

Civil Engineering

Toronto, Ontario, Canada, 2008

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DEVELOPMENT OF PRECAST BARRIER WALL SYSTEM FOR BRIDGE DECKS

By

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August 2008

ABSTRACT

As Ontario transportation infrastructure enters the era of maintenance, rehabilitation and replacement, it needs more use of prefabricated elements and systems, which can quickly assembled to overcome relatively longer time to open bridge to the traffic. This thesis reports on both analytical and experimental investigations conducted on various aspects of precast bridge barrier, including design loads, structural design, sequence of construction, connection details between the barriers and bridge, joints between barrier segments and water leakage control. In this study, thru post-tensioned bolted connection system was developed to connect the precast barrier wall with bridge deck. The primary intension behind these connections is to expedite construction cycle simultaneously with minimizing traffic disruption, improve work-zone safety, quality, and constructible and lower down lifecycle cost of bridges. A total of five experimental tests on full-scale specimens of the developed system were conducted to verify and substantiate the design procedure used to develop the barrier wall system. Good correlation between theoretical ultimate loads, in the form of bending, shear and punching shear resistance, and the experimental findings was achieved.

ACKNOWLEDGEMENTS

I wish to express my deep appreciation to my co-advisor Dr. Khaled Sennah at Ryerson University, for his constant support, excellent guidance, advice and valuable supervision for this research project. Dr. Khaled Sennah devoted his time and effort to make this study a success. Their most helpful guidance is greatly appreciated. Also, I would like to thank my co-advisor, Dr. Reza Kianoush.

I wishes to thank Dr. Clifford Lam and Dr. Bala Tharmabala from Ministry Of Transportation of Ontario for their valuable suggestions during this study and I also wish to thank the Highway Infrastructure Innovation Funding Program of Ministry of Transportation of Ontario (MTO) for financing this project.

I wish to thank Mr. Roger Smith, Mr. Mohamad Aldardari and Mr. Ali for their excellent assistance during the experimental work. I would also like to acknowledge my colleagues and friends for their contribution in accomplishing the objectives of this research study.

There are not enough words to express my gratitude to my mother – Avantikaben and sister – Bijal for their excellent support and encouragement to enable me to complete this research project successfully.

Finally, I would like to express my heartedly feelings of joy of almighty God for the spirit and courage to accomplish this research work.

DEDICATION

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LIST OF ABBREVIATIONS AND SYMBOLS

ϕ_c	= Resistance factor for concrete
ϕ_p	= Resistance factor for prestressed reinforcement bars
ϕ_s	= Resistance factor for non-prestressed reinforcement bars
A	= Cross sectional area of concrete structure, mm ²
a	= Depth of equivalent rectangular stress block, mm
<i>AASHTO</i>	= American Association Of State Highway Transportation officials Specification
A_{bh}	= Area of plate excluding diameter of hole inside
<i>ACI</i>	= American concrete institute
A_N	= Projected area of the failure surface for the anchor, mm ²
A_{NO}	= Projected area of the failure surface of a single anchor remote from edges
A_s	= Area of tensile reinforcement per unit width, mm ² /m
A_{se}	= Cross sectional area of bolt, mm ²
b	= Unit width of section, mm
b_1	= Short side of concentrated load or reaction area, mm
b_2	= Long side of concentrated load or reaction area, mm
b_o	= Perimeter of critical section for slab and footing, mm
<i>CHBDC</i>	= Canadian Highway Bridge Design Code
<i>CPCI</i>	= Canadian Precast Concrete Institute
<i>CSA</i>	= Canadian Standard Association
d	= Average effective depth of section, mm
d	= Effective depth of the section, mm
E	= Modulus of elasticity

f_b	= Concrete stress at bottom fibre
f_c'	= Ultimate compressive strength
f_c'	= Average compressive strength of concrete, MPa
f_{ut}	= Ultimate tensile capacity, MPa
f_y'	= Specified yield strength of reinforcement, MPa
H	= height of barrier wall
h_{ef}	= Effective height
K	= 10, constant (CSA23.3-04)
k_t	= Tensional stiffness of member
L_{ce}	= Length of end region, as per yield analysis
L_{ci}	= Length of interior region, as per yield analysis
$LRFD$	= Load and Resistance Factored Design
L_t	= Length of interior region, 2.4 m (LRFD and CHBDC)
M_{bint}	= Flexural capacity of the cap beam
M_{cint}	= Flexural capacity of the railing about a horizontal axis
M_{max}	= Maximum bending moment at particular section
M_r	= Nominal moment of resistance of the section, KN-m/m
M_{wint}	= Flexural capacity of the railing about its vertical axis
n	= Number of bolts
N_{br}	= Factored concrete breakout of a single anchor in tension in cracked concrete
N_{cbr}	= Factored concrete breakout of an anchor or anchor group in tension

LIST OF ABBREVIATIONS AND SYMBOLS

N_{pr}	= Pullout capacity in tension of a single headed stud or headed bolt
N_{sr}	= Factored resistance of an anchor in tension
PCI	= Precast Concrete Institute
P_f	= Prestressing force in tendon, KN
R	= Constant for ductile member
R_{wi}	= Resistance of the mechanism
S_b	= Section modulus, mm ³
V_c	= Punching shear capacity, N
β_c	= Ratio of short side to long side of concentrated load
θ	= actual angle between the horizontal and the assumed failure plane
$\psi_{c, N}$	= Modification factor (CSA23.3-04 D.6.2.6)
$\psi_{cp, N}$	= Modification factor for post-installed anchors designed for uncracked concrete (CSA23.3-04 D.6.2.7)
$\psi_{ed, N}$	= Modification factor for edge effect

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Chapter-1

INTRDUCTION

1.1 Introduction

Aging bridges of North America requiring repair, rehabilitation, or replacement represent serious problems that have important consequences for bridge users. Introduction of prefabricated elements and systems in bridge construction consists primarily of providing ready-made factory produced elements through mass production of the materials, repeated use of forms, and reduction of on-site construction time and labour by concentrating the construction effort in a fabrication facility rather than at the bridge site. Work zone safety rapid construction is the most important issue to consider in bridge construction and replacement. This thesis deals with the development of precast barrier wall system on which the barrier wall is fabricated in the plant and then shipped to the bridge site to connect it mechanically with the bridge deck.

1.2 The Problem

The design process of traffic and bridge barrier walls specified in the Canadian Highway Bridge Design Code (CHBDC, 2000) is based on the AASHTO Guide Specification for bridge railings (AASHTO, 1989) and the AASHTO Guide for Selecting, Locating and Designing Traffic Barriers (AASHTO, 1977). They include concrete barriers; steel W-beam guardrail on timber or steel posts, steel box-beam on steel posts, cable guardrail with steel or timber post and chain-link fence arresting barrier. Design of barrier wall is based on two aspects. The first is that the traffic barrier can differ from site to site and traffic barrier performance should match specific bridge site requirements. This is called the multiple performance level concepts. CHBDC specified

these requirements in the form of barrier performance level 1,2 and 3. These performance level are depends on traffic volume, percentage of trucks in traffic mix, highway type, barrier clearance, highway curvature, highway design speed, superstructure height above ground level, number of people at risk beneath the bridge and hazards existing beneath the bridge. The second aspect is that traffic barriers should be crash tested to confirm that they will meet the requirements of a specific performance level. A traffic barrier serves dual and often conflicting roles. It must be capable of redirecting and/or containing an errant vehicle without imposing intolerable conditions on the occupants of the vehicle. It should be able to do this for a range of vehicle size and weights, impact speed and angle.

Based on CHBDC provisions, the barrier should be designed to stand a specified lateral static loads simulating vehicle impact at the ultimate limit state. As such, for precast barrier wall system, horizontal and vertical reinforcement should be designed in addition to the connection between the barrier and the deck slab. Also, the connection between the barrier wall and the deck should not leak water over thru traffic under the bridge in the winter times. Therefore, in this thesis, different scenarios of structural details of the precast concrete barrier-to-concrete bridge deck slab connection are investigated including, including changes to the anchorage system size, location and arrangements, thickening, or otherwise strengthening, the rigid barrier, changing material properties, presence of asphalt layer, distance from the barrier to the edge of the deck and shear locking provided by the interface between the top of the deck slab and the bottom surface of the barrier. Joint configuration between barrier segments is of great importance to maintain structural continuity of the barrier wall resistance to vehicle impact load.

Special attention are to be directed to examine the difference between using snug-tight bolted joint and pre-tensioned bolted joint on ultimate strength and water leakage control.

The proposed barrier wall system is expected to have the following advantages:

- 1- Factory made and inspected barriers are certain to have better appearance and quality than C.I.P.
- 2- Bridge barrier walls can be installed in one day.
- 3- No protruding rebars cages or step/keys interfere with the flat-finished bridge deck surface.
- 4- It may be scheduled to be installed any time of the year with no rain or snow but based on the lowest possible temperature for the cement grout to harden.
- 5- Because the barrier is factory made, there is no costly labour-intensive rebar installation in the field.
- 6- No expensive barrier forms are needed.
- 7- In case of connecting the precast barrier wall with the existing deck slab, drilling down the deck slab can be used. If the rebar is encountered while drilling the hole, a hole can be drilled beside the rebar, and this offset can be taken by the difference in size between the connecting threaded rod and its encased sleeve. The rod can be taken of 25 mm diameter while the sleeve can be of 75 mm diameter.
- 8- With use of pretension in the connecting rods, no chance of salt and water trapping around the anchor rods or corroding bottom plates or nuts.

1.3 Objectives of This Study

The objective of the proposed research project is to develop precast concrete bridge barrier system that meets the CHBDC design requirements with respect to strength and durability (leakage).

1.4 Scope of Work

The proposed research study includes following tasks:

- 1) Conducting literature survey on state-of-the-art and state-of-practice of traffic and bridge barrier systems.
- 2) Conducting Structural design the precast concrete barrier wall and connection between the barrier wall and the deck slab per CHBDC design requirements.
- 3) Conducting static load tests to complete collapse on full-size specimens of the proposed precast barrier wall system to verify and substantiate the structural design.
- 4) Conducting water leakage (durability) test on the proposed barrier wall system.

1.5 Content and Arrangement of This Study

Chapter # 2 emphasizes on brief historical background of the development of traffic and bridge barrier walls and their connection details. Chapter # 3 summarizes the theoretical design of barrier walls based on CHBDC and LRFD design codes. Chapter # 4 presents the experimental program conducted on five full-size specimens of the proposed bridge barrier wall system. This chapter includes model details, material used in the experiments, test setup, instrumentation and test procedure. Chapter # 5 presents the discussion of results obtained from the experiment testing along with their correlation with the theoretical findings. Chapter # 6 presents summary of the research findings and recommendations for future research.

Chapter-2

LITERATURE REVIEW

2.1 Introduction

This chapter presents (i) brief background about bridge barrier theoretical and experimental investigations pertained to the subject of the current research work, (ii) literature review of available connection details between the deck slab and the barrier wall, and (iii) literature review of available connection details at barrier-to-barrier joints.

2.2 Background

The design process of bridge barriers specified by the current Canadian Highway Bridge Design Code (2006), CHBDC, is based upon the AASHTO Guide Specifications for Bridge Railings (1989) and the AASHTO Guide for Selecting, Locating, and Designing Traffic Barriers (1977). The AASHTO Guide Specifications for Bridge Railings is a major departure from the static load design procedures traditionally used for the design of traffic barriers. It is based on two aspects. The first aspect is that traffic barrier can differ from site to site and that traffic barrier performance should match specific bridge site requirements. This is called the multiple performance level concepts. The second aspect is that traffic barriers should be crash tested to confirm that they will meet the requirements of a specific performance level. This aspect acknowledges that traditional static load design methods are inadequate for predicting traffic barrier performance. Traffic barrier requirements vary from bridge site to bridge site and are based on the expected frequency and consequences of vehicle accidents at a bridge site. CHBDC specifies these requirements in the form of barrier performance levels 1, 2 and 3.

These performance levels depend on traffic volume, percentage of trucks in traffic mix, highway type, barrier clearance, highway curvature, highway design speed, superstructure height above the ground level, number of people at risk beneath the bridge and hazards existing beneath the bridge.

A traffic barrier serves dual and often conflicting roles. It must be capable of redirecting and/or containing an errant vehicle without imposing intolerable conditions on the occupants of the vehicle. It should be able to do this for a range of vehicle sizes and weights, impact speed and impact angles. Compromises are necessary to achieve a balance between the structural and safety requirements. Crashworthiness focuses on the capability of a vehicle to protect its occupants in a collision. Crashworthy barrier is the one that can be impacted by a vehicle at or below the anticipated operating speed of the roadway with low probability of serious injury to the occupants of the vehicle. The evaluation of vehicle crashworthiness has involved numerous full-scale crash tests of the vehicle and highway hardware to verify the compliance with regulatory requirements (Alberson et al., 2005; Reid et al. 2001; Plaxico et al. 2000; Pfeifer and Sicking, 1997). Specific crash testing on bridge barriers was reported elsewhere (among them: Bielenberg et al., 2003; Buth, et al., 2000; Mak, et al., 1994). Few authors conducted pendulum impact testing as an alternative to vehicle crash testing (El-Gamal et al, 2007; El-Salakawy, et al., 2004; El-Salakawy, et al., 2003; Bank, et al., 1997). This test-guided product development process is very costly and time-consuming. As an alternative, computer simulation tools are increasingly being used for the upfront assessment of crashworthiness without going through multiple-cycles of prototype testing and iterative design changes.

Computer simulation of vehicle collisions has improved significantly over the past few years. With advances in computer technology and explicit large-displacement non-linear finite-element (FE) software, full-scale models and simulations of such sophisticated collisions are becoming ever more possible. With the availability of lower cost super-computers based on Symmetric Multi-Processor and Massively Parallel Processor technologies, simulations of frontal, offset, side and skew impacts of vehicles can be made more elaborate and efficient. Many authors dealt with the validation of these FE vehicle models by correlating the results of the finite-element simulation with those obtained from existing vehicle crash tests (among them: Zaouk, et al., 1998). Few types of software for large displacement, inelastic, dynamic, finite-element crash analysis is commercially available (among them: LSDYNA, MADYMO, and ABAQUS/ Explicit software). Several authors dealt with finite-element computer simulation of flexible highway barriers (Ranzo and Bonin, 2005; Tabiei and Wu 2000; Gentry and Bank, 1998; Plaxico, et al., 1998; Uddin and Hackett, 1998; Ross, et al., 1993), while others dealt with finite-element computer simulation of existing guardrail terminals (Wekezer et al., 2000; Ray and Patzner, 1997). Few authors dealt specifically with impact simulation, using finite-element modeling and multi-body dynamics, of rigid concrete bridge barrier and bridge railing (Consolazio, et al., 2003; Itoh, et al., 2000; Marzougui, et al., 2000; Thomson et al., 1999). Other authors dealt specifically with crash testing of transitions from roadway guardrail to concrete bridge barrier (among them: Buth, et al., 2000; Faller, et al., 1998; Faller, et al., 1996). Few authors dealt with the finite-element simulation of collisions of heavy trucks with portable concrete barriers (Marzougui et al., 2008; Itoh et al., 2007; Atahan, 2006).

Few authors dealt with the use of fiber reinforced polymers, FRP's, in concrete barriers. Maheu and Bakht (1994) developed a new barrier wall using FRP (GFRP) NEFMAC grids, with connection to the deck slab by means of double-headed tension bars of steel spaced at 300 mm. This new barrier wall system was adopted in the CHBDC of 2000. El-Salakaway et al. (2003) tested to-collapse PL-2 and PL-3 barrier walls reinforced with GFRP bars. They conducted pendulum impact testing (Elsalakaway, 2005) on the developed barriers in lieu of vehicle crash testing.

2.3 Literature review of available connection details between the barrier wall and the deck slab

The Pre-stressed Group, (Aug-2008) developed connection details between the deck slab and precast barrier wall to withstand heavy impact by approached accidental vehicle as well as to give better stability and flexibility against negative moments and tension-compression variations. As shown in the schematic diagram in Fig. 2.1, a 100×50 mm slots were constructed at 600 mm c/c at the bottom front and rare side of the precast barrier wall in a staggered fashion. Before pouring concrete in the precast barrier wall, 100×100 steel plate inserts with 4 lugs (i.e. shear studs) are embedded in the formwork with the steel plate facing the vertical side of the slot. Similar arrangement is embedded in the deck slab with the steel plate facing the horizontal side of the slot. To connect the precast barrier wall with the deck slab, a 10 mm steel plate or rod is placed at the junction between the vertical and horizontal steel plate in each slot and then welded together. Concrete grout is then used to fill the slots. The drawback of this connection details is the possibility of the corrosion of the steel plates if they are not well protected against the environment. Also, the quality of welding on site is questionable.

Other precast barrier-to-deck slab connection detail was developed by Washington Department of Transportation as shown in its Standard Plan C-8e (Washington, Aug-2005). As shown in Fig. 2.2, a staggered/alternate hole of 35 mm at the bottom front and bottom rear faces of the precast barrier wall at such an angle that would cross the middle of the barrier wall when intrudes the deck slab. Similar holes are introduced in the deck slab at the same line of action. 30 mm dia. Bolts are used to establish full tight joint and the holes are then filled with grout to avoid weather effects. In case of replacement of deteriorated barrier walls, holes in the existing deck slab are drilled down. If slab reinforcement is in the way drilling, drilling is specified to be avoided. In return, anchor bolts with epoxy grout are used. All steel elements are either epoxy painted or hot-dip galvanized for environmental protection.

Construct a staggered/alternate hole of 35mm at bottom front and bottom rear face of precast barrier wall in such an angle that it will cross bottom middle of barrier wall and then intrudes in deck slab. As shown in (Fig. 2.2), Insert and apply full tight connection of nuts and threaded rod (30mm diameter); after it, grout bolt pockets to avoid any weather effects. During drilldown hole in deck slab if reinforcement coming as an obstruction or in path. We just stop drilling and provide anchor bolt with epoxy grout would work better. Trapezoidal pockets at the joint of each barrier connects are grouted later on to insure better water and salt resistance.

The AASHTO Guide for Selecting, Locating and Designing Traffic Barriers (AASHTO, 1977) developed precast barrier wall-to-deck slab joint as shown in Fig. 2.3. , Construct an additional 'L' shape at rear-bottom side of the precast barrier wall in such a way that depth equals to deck slab width. Also design Precast barrier wall in such a way

that centre of gravity of whole barrier lies on deck slab. On inner front bottom side of barrier wall fixed threaded sockets or ferrules @ 600mm c/c and 300mm from each ends to hold a bolt coming from the bottom of deck slab. Construct hole at bottom rear side of precast barrier walls in such a way that it will match central depth of deck slab @ 600mm c/c.

In its Guide for Selecting, Locating, and Designing Traffic Barriers (AASHTO, 1977), and precast concrete barrier details provided by Clampcrete Construction Systems, Hudson, New Hampshire (at <http://csigroup.biz/clampcrete/html/featuresindex.htm>) as shown in Fig. 2.4. Clampcrete has designed this barrier system to emulate cast-in-place types for strength, load transfer continuity and overall performance. Yet, the system exceeds C.I.P. by gaining the high quality of factory manufacture, inspection and approval. It was claimed that typically, 5-men crews install 120 linear meters a day. In this system, vertical holes are constructed through the lower slopped side of the barrier wall down to the bottom of the barrier at 600 mm c/c. Similar holes are introduced in the deck slab at the same line of action of the barrier holes. High tensile rods are then used to tighten the precast barrier-slab joint. Then, grouting materials are pressured into the holes. The disadvantage of this system is the possibility of local failure in the concrete surrounding the anchor rods due to high-localised stresses.

Similar connection detail was developed by State of California Department of Transportation (Oct-1999) but with double alternate side bolts as shown in Fig. 2.5. In this case, alternate vertical holes at front and rear side of the bottom portion of the precast barrier wall is introduced at 600 mm c/c in with 300 mm c/c offset between the front and rear holes. Threaded rods are used to connect precast barrier wall with deck slab.

Similar connection detail to that developed by Clampcrete Corporation and California Department of Transportation presented above was designed and tested by Maheu, J., and Sadler C. (1992). Figure 2.6 shows that 90 mm diameter circular recesses are incorporated in the front face of the precast barrier at 220 mm spacing. Those coincides with the locations of especially fabricated anchorage assemblies shown in Fig. 2.6, which are cast into the concrete to prevent local failure around the anchor rods to be used to connect the barrier wall with the deck slab. Barrier walls are then attached to the deck slab by means of coarsely threaded dowel bars, 19.6 mm in diameter and 390 mm long. The bars are epoxy into holes, which are drilled into the deck slab after the barrier wall is placed in position. The holes are 250 mm deep and 25 mm in diameter. The top portion of the bar extends into the recess in the barrier wall, through a hole in the steel plate of the anchorage assembly. After the epoxy reaches a specified strength of 30 MPa, the wall is then fastened to the dowels using plate washers and anchor nuts, hand tightened to a snug fit. After the nuts have been tightened, the anchorage recesses are filled with a non-shrink grout and finished flush with the face of the barrier wall. In 2008, Ngan and Stierner reported results from full-size static load testing to collapse of similar single side connection between a PL-2 precast barrier wall and slab (Ngan and Stierner, 2008).

NCHRP-119 (Transportation, 1978) presented the connection detail shown in Fig. 2.7. To connect the precast barrier and the deck slab, holes to accommodate 30 mm diameter dowel bars are allowed in the formwork of the precast barrier wall as well as drilled in the deck slab at the similar alignment. 80 mm diameter holes are constructed in the barrier wall in pairs every 1 m c/c and 200 mm c/c of the two holes in each pair. This

is in addition to one hole at 300 mm from the each end of the barrier wall as shown in Fig. 2.6. During drilling down holes in the deck slab, if slab reinforcement obstructs the path, drilling must be stopped and in return anchor bolts with epoxy grout are used. Length of dowels is varied according to variation in length of barrier wall. A knob is welded at the ends of dowel bar for better anchoring as shown in Fig. 2.6. After placing precast barrier walls in position, cement grout is pressured into the holes from the top.

William et al. (William et al., 2007) developed barrier anchorage system as shown in Fig. 2.8 for repair or retrofit. In this system, #6 bent bars anchored approximately 11 inches from the field side edge of the 8-inch thick deck slab. These bars were anchored approximately 5.25 inches into the concrete deck slab using HILTI RE 500 adhesive anchoring system. Several days, after these #6 anchoring bars were installed, the bridge barrier was reconstructed similar to previous installation. This replaced barrier wall was then tested under static and dynamic loading.

Mitchell et al (2006) developed barrier wall retrofit using post-installed mechanical anchors as shown in Fig. 2.8. This configuration consists of $\frac{3}{4}$ inch ASTM A307 threaded rods used as vertical reinforcement running through PVC ducts embedded into concrete and connected to flush-mount undercut anchors installed in the deck slab. Using the anchor connection design provisions of ACI 318-05 Appendix D, the capacity of the bridge deck overhang as governed by concrete breakout around the anchors was estimated in terms of the number of anchors used to connect the barrier wall with the deck slab.

Another unique type of barrier wall to deck slab connection was developed by Mohsen A. Shahawy NCHRP SYNTHESIS 324, (2003) and Tokerud, R (TRB,

September 1978). The main concept is easy and fast construction with lowest machinery and drilling process by fixing dowels in deck slab and align barrier wall on top of it and then fill the gap with grout. The standard construction procedure is: Drill holes on concrete deck slab and grout a 30mm dowel bars. During drilldown hole in deck slab if reinforcement coming as an abstraction or in path. We just stop drilling and provide anchor bolt with epoxy grout would work better. Length of dowel is very according to variation in length of barrier wall. Weld a knob at the end of dowel bar for better anchoring. Construct a 80mm hole in precast barrier wall in pair of 2 dowels (pair distance 200mm c/c) @ 1 meter c/c and 300mm from each ends. After placing barrier wall in a position, apply pressure grout from the top of 80mm hole.

2.4 Literature review of available connection details at barrier-to-barrier joints

State of New York Department of Transportation (New York, 2005) developed a rear side bolt connection between barrier walls shown in Fig. 2.10. The main concept behind this connection is to provide better tension resistance by tightly fixing bolts; to give better resistance power against rear side tension crack that occurs due to impact from the front side of barrier wall. Three slots of 100X100 mm are constructed at rear face of the end of each precast barrier wall. An insert plate of 12 mm thickness is attached to each side of the end of each barrier wall as shown in Fig. 2.10 by providing sufficient anchored hook embedded in to concrete. 22 mm diameter holes are drilled at the centre of insert plates to allow for bolting them from the rear side of precast barrier walls with 20 mm diameter bolts.

Texas Department of Transportation and Manitoba Floodway Authority had developed barrier-to-barrier joint details with longitudinal dowel connection as shown in

Fig. 2.11 (Texas, 1990) & Manitoba Floodway Authority, (Manitoba, Oct-2005). Four dowels of 20 mm are placed and anchored from inside of precast barrier wall. Each dowel has welded or applied threaded nut of at least 2D in diameter (i.e. 40 mm) at the end of each dowel for better anchorage against slippage from hole when vehicle impact at joint occurs. At other side of barrier wall four consecutive holes/sleeves of 40 mm diameter are constructed to insert and place dowel bars projecting from the adjacent barrier wall. Trapezoidal shear key of 75X100X100 mm is applied constructed vertically at each end of the barrier wall to provide better locking facility against impact force. At the top face of barrier wall, one hole is constructed vertically through the sleeves so that cement grout can be pressured into the sleeves after dowel bars are inserted into sleeves from adjoining barrier wall.

Province of British Columbia's Ministry of Transportation and Highways (British Columbia, 1996) and KON KAST Products Ltd., (Feb-2005) presented wedge and hook connection to connect bridge barriers as shown in Fig.2.12. A 562 mm deep wedge with the lower width as twice as the upper width is constructed on one side on barrier wall end to be inserted into a matching groove on the side of the adjacent precast barrier wall. L-shaped male hook is provided on the top of expanded wedge at a distance of 100 mm, while a C-shaped female ring is provided at the side of the adjacent barrier wall end at a similar height to anchor the two barrier ends together.

Texas Department of Transportation Design Division (Texas, 1990) produced structural drawings of a Zigzag and Horizontal Bolt Connection between barrier walls as shown in Fig. 2.13. At each barrier end, a L-shaped slot of 300X100 mm is constructed along the barrier depth, to face a similar slot in the adjacent barrier wall in such a way that

they form the barrier thickness when combined. Thus, it is called opposite slots in a zigzag shape. Three boltholes are introduced into each barrier end to allow for 22 mm diameter bolts to join the barriers together as shown in Fig. 2.13. Boltholes are spaced vertically at 200 mm c/c with 111 mm edge distance from the top of barrier wall.

Texas Department of Transportation (Texas, 1990) and State of Washington Department of Transportation (Washington, 2000) had developed structural drawings for a staggered dowel connection to join precast barrier walls as shown in Fig. 2.14. In this connection, four dowel bars are introduced at each barrier end in such a way part of each bar is embedded into concrete while the rest of the bar projects from the side of the barrier end. To facilitate the overlap or to avoid obstruction between projecting anchor bars from adjacent barrier walls, these bars were installed in a staggered manner. As shown in Fig. 2.14, a trapezoidal-shaped recess is constructed on each side of the barrier end. After placing the barriers over the deck slab, a slot of 300 mm is formed and then filled with concrete grout to act as a shear key.

Texas Department of Transportation developed structural drawings for cross-bolted connection for F-shape barrier-to-barrier joints as shown in Fig. 2.15 (Texas, 2005). On inner side of the barrier wall, 22 mm diameter hole is constructed at 20-degree angle from the centre of barrier wall. When the barriers are positioned over the deck slab, a 20 mm diameter bolt was inserted so that it appears at the rear end of the other barrier wall. A similar bolt configuration is introduced at the joint from the adjacent barrier in such a way the two holes are at 200 mm c/c vertically. In addition, trapezoidal shear key slot of 300 mm length is constructed within the ends of the adjacent barrier walls as

shown in Fig. 2.15 and then filled for cement grout for better anchoring and interlocking of barrier walls.

Few authorities (Washington, 2006; Midwest, 2006; Oregon, 2005; Joint, 2004 and Connecticut, 2003) presented structural details of tie, pin and loop connection for barrier-to-barrier joints as shown in Fig. 2.16. In this connection, mild steel flat strip of 50 mm wide, 1000 mm long and 10 mm thick is placed at the inner face of a vertical slot located at the end of the barrier wall. This steel strip has 50x8 mm steel plates welded to it at 200 mm c/c and oriented horizontally as shown in Fig. 2.16. Each steel plate has 22 mm diameter hole at its centre. At the end of the adjacent barrier wall, similar steel plate assembly is constructed but with horizontal plates in a sluggard fashion. After placing the adjacent barriers over the deck slab, a 20 m pin is inserted in the holes from the top of the trapezoidal slot to lock the joint. Then the trapezoidal shear key slot is filled with cement grout.

Texas Department Of Transportation (Texas, 1990) and Florida Department Of Transportation (Florida, 2003) Design Division introduced the 'J' connection for barrier-to-barrier joints as shown in Fig. 2.17. In this connection, a MS flat steel plate of 50 mm wide, 500 mm long and 8 mm thick are bent as a 'J' shape and then welded to the reinforcing steel bars in the barrier wall. This steel plate is located at 200 mm from the top or bottom face of the barrier wall at its ends. Trapezoidal slot is constructed on each side of the barrier wall ends so that after placing the barriers over the deck slab, they form a shear key slot of 300 mm length. This slot is then filled with cement group for better anchoring and interlocking at the barrier ends.

Hy-Grade Precast Concrete Products, Inc., of New York specifies the 'I' hook connection for barrier-to-barrier joints as shown in Fig. 2.18. In this connection, an 'I' shape made of 10 mm thick steel plates is fabricated and sticks to the vertical side of the barrier end in such a way that a T portion of it projects out of the barrier end. This 'I' shape assembly is anchored to concrete using two steel anchors as shown in Fig. 2.18. On the other side of the barrier wall, a trapezoidal 'C' shape assembly made of steel plates is embedded in the vertical side of the barrier end and anchored to the concrete using hooked bars as shown in Fig. 2.18. After placing one barrier wall over the deck slab, the adjacent barrier is placed vertically over the deck slab in such a way that the 'T' shape assembly is inserted into the trapezoidal "C" shape assembly. Finally, the trapezoidal shape assembly is filled with cement grout for anchoring and interlocking at the joint.

The Pre-stressed Group, Windsor, Ontario specifies a welded connection between barrier ends as shown in Fig. 2.19 (<http://www.psi-hci.com/documents.htm>, 2008). In this connection, three cubic slots are constructed on each side of the barrier wall ends, on which three-100X100X12 mm insert plates are provided within the slot to form a cubic shape. When the subsequent barrier is placed in position over the deck slab, these slots will face each other as shown in Fig. 2.18. To connect these slots together, 120x70x12 mm steel plates are welded to the middle of insert plates at the joint. The slots are then pressured with cement grout.

Mohsen A. Shahawy, Transportation Research Board, Washington D.C. 2003 presented structural details of post-tensioned connection for barrier-to-barrier joints as shown in Fig. 2.20 (NCHRP SYNTHESIS 324, 2003). In this connection, four through holes are constructed in the longitudinal direction of barrier walls. All holes are

constructed in such a manner that the axis of each hole is parallel to the front impacting face of barrier wall with eccentricity with respect to the vertical centre of gravity of barrier member for better tension resistance capacities. Once, all precast barrier walls are placed in proper alignment and connected to the deck slab, post-tensioning cables inserted in the holes and post-tensioned.

Chapter-3

REVIEW OF THEORITICAL DESIGN STUDY

3.1 Introduction

The design process of bridge barriers specified by the current Canadian Highway Bridge Design Code (2006), CHBDC, is based upon the AASHTO Guide Specifications for Bridge Railings (1989) and the AASHTO Guide for Selecting, Locating, and Designing Traffic Barriers (1977). The AASHTO Guide Specifications for Bridge Railings is a major departure from the static load design procedures traditionally used for the design of traffic barriers. It is based on two aspects. The first aspect is that traffic barrier can differ from site to site and that traffic barrier performance should match specific bridge site requirements. This is called the multiple performance level concepts. The second aspect is that traffic barriers should be crash tested to confirm that they will meet the requirements of a specific performance level. This aspect acknowledges that traditional static load design methods are inadequate for predicting traffic barrier performance. Traffic barrier requirements vary from bridge site to bridge site and are based on the expected frequency and consequences of vehicle accidents at a bridge site. CHBDC specifies these requirements in the form of barrier performance levels 1, 2 and 3. These performance levels depend on traffic volume, percentage of trucks in traffic mix, highway type, barrier clearance, highway curvature, highway design speed, superstructure height above the ground level, number of people at risk beneath the bridge and hazards existing beneath the bridge. A traffic barrier serves dual and often conflicting roles. It must be capable of redirecting and/or containing an errant vehicle without

imposing intolerable conditions on the occupants of the vehicle. It should be able to do this for a range of vehicle sizes and weights, impact speed and impact angles. The following section presents (i) description of proposed precast barrier wall system, (ii) description of bridge models to be tested to-collapse, and (iii) criteria for structural design of bridge barrier.

3.2 Description of the proposed precast barrier wall system

As Ontario transportation infrastructure enters the era of maintenance, rehabilitation and replacement, prefabricated elements and systems, which can be quickly assembled, will certainly have many advantages as compared to the conventional systems. This is because prefabricated systems can be installed and the traffic can be opened in a very short period of time, minimizing the lane closure time, reducing the cost and design time, and minimizing forming and labour work. Prefabricated elements and systems reduce on-site construction time and labour by concentrating construction efforts in a controlled environment at the fabrication facility rather than at the bridge site. These advantages result in considerable savings in tax payers' money. As such, the objective of this research study is to develop cost-effective, crashworthy, and durable precast concrete bridge barrier system to accelerate the construction of new bridges and the replacement of deteriorated bridge barrier walls. To develop this barrier system, various aspects, including design loads, analysis and design, sequence of construction, connection details between the barrier and the bridge, joints between barrier segments and water leakage control, were considered. This is in addition to the previous experience of others reported in the literature with respect to barrier connection details.

Different scenarios of structural details of the proposed precast concrete barrier-to-concrete bridge deck slab connection included changes to the anchorage system size, location and arrangements, thickening, or otherwise strengthening, the rigid barrier, changing material properties, presence of asphalt layer, distance from the barrier to the edge of the deck and shear locking provided by the interface between the top of the deck slab and the bottom surface of the barrier. Special attention was directed to examine the difference between using snug-tight bolted joint and pre-tensioned bolted joint on ultimate strength and water leakage control.

This investigation resulted in a barrier wall system shown in Figs. 3.1 and 3.2, on which barrier walls of 3-meter segments will be fabricated in the precast plant and to be shipped to the site. The proposed barrier wall was intended to meet the criteria for CHBDC performance level 3 (PL-3), representing the majority of bridge barriers in Ontario highways. As Fig. 3.1 shows, a steel corrugated sleeve is to be embedded in the precast concrete slab at an angle equal to the slope of the top inclined portion of the inner face of the barrier. Similar steel corrugated sleeve is to be embedded in concrete deck slab, aligned with those present in the barrier. A 600-mm spacing between sleeves is proposed based on the results from structural design reported at the end of this chapter. After hardening of the deck slab, an overlay of 35 mm concrete grout material is then introduced over the concrete deck slab edge with a width equal to barrier width. Then, barrier segments can be laid over the deck slab edges, with centrelines of sleeves aligned so that 25-mm diameter threaded bar made of high strength steel can be inserted through. The threaded rod can then be bolted from the topside of the barrier followed by the bottom side of the deck. A torque force can be applied using mechanical torque to the top

nuts to provide initial (pre-tensioned) force in the threaded rod. The intention of the pre-tensioned force is to produce permanent contact pressure (i.e. compressive stresses) at the contact surface between the bottom of the barrier wall and the top of the concrete deck slab grout. This would assist in preventing water leakage over through traffic under the bridge. To increase shear resistance, the top surface of the concrete deck slab was scratched in the direction of traffic. To produce similar surface at the bottom side of the precast barrier wall, timber strips were fixed on the topside of the bottom sheet of timber form to produce the corrugated concrete surface shown in Fig. 3.1. After pre-tensioning the bolt to the desired level, concrete grout is to be inserted from a hole in the top steel plate on the side of the top nut to fill the gap between the threaded rod and the sleeve.

Figure 3.2 shows the proposed vertical joint between the precast barrier wall segments. In this case, a HHS 102×102×9.5 mm tube is embedded on one end of the precast barrier wall segment, with 4 shear studs welded to it to provide anchorage resistance with concrete. On the other side of the joint, a S 75×11 steel beam is projecting from the adjacent barrier wall segment, with four shear studs welded to it to provide anchorage resistance with concrete. To activate the joint resistance, one barrier wall segment is to be laid over the deck slab. The, the adjacent barrier wall with the S steel beam to be laid over the deck slab vertically in a way that the S steel beam web slides through a vertical slot in the HSS skin in the other barrier wall edge. The HSS steel tube is then filled with concrete grout to keep the S steel beam in location and enhance the rigidity of the vertical joint.

3.3 Description of bridge models to be tested to-collapse

The proposed precast barrier wall system shown in Fig. 3.1 was designed based on structural design criteria shown in the following section. However, this section is intended to describe the rationale for the selection of bridge model configurations to be tested experimentally. The theoretical design review showed that the following structural quantities are essential in the barrier design process: (i) flexure strength; (ii) punching shear capacity; (iii) pre-tensioned rod capacity; and (vi) failure yield line capacity. As such, five bridge models were erected in the structural laboratory to serve this purpose.

Model M1 was intended to examine the flexural capacity of the connection between the precast barrier wall and the concrete deck slab as well as the punching shear capacity of the threaded rod-steel plate connection embedded in the concrete deck slab. Also, this model would examine the flexural capacity of the one-way action of the joint between the two inclined surfaces of the inner face of the barrier wall. Figure 3.3 presents cross-section of the proposed barrier wall system showing the reinforcing bar arrangement in the barrier wall. While Fig. 3.4 presents the cross-section and elevation of the barrier wall system showing the threaded rod arrangement. It can be observed that the barrier length was selected to be 1200 mm to accommodate two-threaded rod, 600 mm apart. Figure 3.5 shows details of the steel reinforcement in the deck slab.

Model M2 was intended to be a control specimen representing the cast-in-place concrete barrier wall currently used in Ontario bridges for performance level 3 (PL-3). It should be noted that the amount of wall reinforcement, both vertically and horizontally, was identical to that used in model M1 for the sake of comparison. However, vertical reinforcement in Model M2 was extended into the concrete deck slab to provide concrete

anchorage. Figures 7 and 8 present the details of reinforcing bars in the barrier wall and deck slab, respectively. One may notice that both models M1 and M2 were loaded laterally at 990 mm depth from the top surface of the concrete deck slab using a line load of 1200 mm.

Model M3 was intended to examine the overall performance of the barrier wall segment when connected to the cantilever part of the concrete deck slab. Figure 3.9 shows Model M3 layout with 900 mm deck slab cantilever length. While Fig. 3.10 shows layout of the reinforcing bars and the threaded rods for the 3000 mm length barrier segment. Figures 3.11 and 3.12 show reinforcing bar details of the barrier wall and the deck slab, respectively. It should be noted that this barrier model was loaded over a 200×200 mm square area centred at the mid-length of the barrier and at 990 mm depth from the top surface of the deck slab.

Model M4 was identical to Model M3 but without the cantilever slab. Model M4 was intended to examine the overall performance of the barrier wall segment with respect to yield-line pattern when the barrier wall was loaded concentrically. In this case, the concrete slab was laid over the structural lab floor in a way to prevent its vertical deflection. However, Model M3 was intended to examine the effect of the slab cantilever on the structural performance of the barrier wall as well as the mode of failure. Figure 3.13 shows Model M4 layout with 225 mm thick deck slab. While Fig. 3.14 shows layout of the reinforcing bars and the threaded rods for the 3000 mm length barrier segment which was identical to those of Model M3. Also, it should be noted that this barrier model was loaded concentrically similar to Model M3.

Model M5 was identical to Model M4 but the applied lateral load was eccentric. Figure 3.15 shows Model M5 layout in addition to the barrier wall reinforcing steel detailing. While, Fig. 3.16 shows layout of the reinforcing bars and the threaded rods for the 3000 mm length barrier segment which were identical to those of Model M4. Figure 3.17 show reinforcing bar detailing of the deck slab. It should be noted that this barrier model was loaded over a 200×200 mm square area centred at the end of barrier and at 990 mm depth from the top surface of the deck slab as shown in Fig. 3.16. The intent of this barrier model was to examine the flexural capacity and the yield-line pattern under edge impact.

3.4 Criteria for structural design of bridge barrier

Initial design of the proposed precast bridge barrier wall was conducted to meet the design criteria specified in the CHBDC for static loading. Special attention is being given to barrier requirements that vary from bridge site to bridge site and are based on the expected frequency and consequences of vehicle accidents at a bridge site. CHBDC specifies these requirements in the form of barrier performance levels 1, 2 and 3. The lowest performance level 1 (PL-1) is intended for use on low traffic volume bridges that have unusual safety hazards. Performance level 2 (PL-2) is intended to produce traffic barriers whose performance is equivalent to that of the better traffic barriers designed according to CHBDC. These barriers are expected to use on highways with moderate to heavy traffic. The highest performance level 3 (PL-3) is intended for use in situations where there is a need for a superior traffic barrier. The requirements of this performance level are expected to be adequate for all but the most extraordinary bridge sites. Traffic barrier height requirements for each of these performance levels are intended to prevent

impacting vehicles from vaulting or rolling over a barrier; the higher the centre of gravity of the impacting vehicle, the greater the required traffic barrier height needed to contain it. CHBDC specifies minimum barrier heights and corresponding lateral (transverse), longitudinal and vertical design loads for each performance levels 1, 2 and 3. CHBDC specifies this height for lateral loading as 900 mm over the asphalt later for performance level 3.

CHBDC specifies loads on traffic barriers as stated in Table 3.1. The shown transverse, longitudinal and vertical loads shall be applied simultaneously over a certain barrier length. CHBDC specifies that transverse load shall be applied over a barrier length of 1200 mm for PL-1 barriers, 1050 for PL-2 barriers, and 2400 mm for PL-3 barriers. Since transverse loading creates the critical load carrying capacity, it was decided to ignore both the longitudinal and vertical loads in the design of barrier wall reinforcement and anchorages between the deck slab and the barrier wall. It should be noted that CHBDC specifies that vertical load shall be applied over a barrier length of 5500 mm for PL-1 and PL-2 barriers and 12,000 mm for PL-3 barrier. This makes the contribution of vertical load on the factored applied moment on the deck slab design small enough to be ignored. The following subsection summarize design procedure used to determine the moment of resistance of the section, the punching shear capacity and the capacity of the pre-tensioned rods and the LRFD yield-line analysis, respectively.

3.4.1 Bending Resistance

Following are the equations used to calculate moment of resistance of reinforced concrete section. It should be noted that this resisting moment was used to design barrier wall reinforcement, design of the size and number of pre-tensioned rods and to provide

means to calculate the ultimate load carrying capacity of the barrier wall based on the yield-line theory.

$$a = \frac{\phi_s A_s f_y}{\phi_c f_c' \alpha_l b} \dots\dots\dots(3.1)$$

$$\alpha_l = 0.85 - 0.0015 f_c' \geq 0.67 \dots\dots\dots(3.2)$$

$$M_r = \phi_s A_s f_y (d - a/2) \dots\dots\dots(3.3)$$

Where:

- a = Depth of equivalent rectangular stress block, mm
- A_s = Area of tensile reinforcement per unit width, mm²/m
- f_y = specified yield strength of reinforcement
- α_l = Constant
- f_c' = Average compressive strength of concrete
- b = Unit width of section, mm
- M_r = Nominal moment of resistance of the section, KN-m/m
- d = Effective depth of the section, mm
- ϕ_s = 0.90 (Ref. CHBDC CL. 8.4.6)
- ϕ_c = 0.75 (Ref. CHBDC CL. 8.4.6)

3.4.2 Punching Shear Capacity

Punching shear capacity of model M1 to M5 were calculated using different North American codes [AASHTO-LRFD (1998); ACI (2002); CHBDC (2000); CSA-A23.3 (2004)] and the general punching equation given by Tusi et al. (1986), whit et al. (1993) and Kim et al. (1994). It should be noted that the AASHTO-LRFD and ACI

punching-shear equations are similar. In general punching equation, a 37° value is used for angle of failure plane with horizontal surface, which was proposed by Tusi et al. (1986), in the calculation of punching shear.

The punching shear equations as described above and used in the current study are presented in SI units as follows:

1. Check of The Three Possible Failure Mode:

The following three checks are specified in Appendix D of CSA 23.3-04, Design of Concrete Structures (2004).

a. Tensile Capacity OF Steel Bolt:

$$N_{sr} = \phi_s n A_{se} f_{ut} R \dots\dots\dots (3.4)$$

Where:

N_{sr} = Factored resistance of an anchor in tension

ϕ_s = 0.9

f_{ut} = Ultimate tensile capacity

n = Number of bolt

A_{se} = Cross sectional area of bolt, mm²

R = Constant for ductile member, 0.8

b. Concrete Breakout of the Embedment:

$$N_{br} = \frac{A_N}{A_{NO}} (\psi_{ed, N} \psi_{c, N} \psi_{cp, N}) N_{br} \dots\dots\dots (3.5.a)$$

$$N_{br} = K \phi_c \sqrt{f_c} h_{ef}^{1.5} R \dots\dots\dots (3.5.b)$$

Where:

N_{br} = Factored concrete breakout of a single anchor in tension in cracked concrete

N_{cbr} = Factored concrete breakout of an anchor or anchor group in tension

$\psi_{ed, N}$ = Modification factor for edge effect

$\psi_{c, N}$ = Modification factor (CSA23.3-04 D.6.2.6)

$\psi_{cp, N}$ = Modification factor for post-installed anchors designed for uncracked concrete (CSA23.3-04 D.6.2.7)

A_N = Projected area of the failure surface for the anchor, mm^2

A_{NO} = Projected area of the failure surface of a single anchor remote from edges

ϕ_c = 0.75

K = 10, constant (CSA23.3-04)

f_c' = Ultimate Compressive Strength, MPa

h_{ef} = Effective Height

R = Constant for ductile member, 1.15

c. *Concrete Pry-out Capacity:*

$$N_{pr} = 8 A_{bh} \phi_c f_c' R \dots\dots\dots (3.6)$$

Where:

N_{pr} = Pullout capacity in tension of a single headed stud or headed bolt

ϕ_c = The resistance factor of concrete, 0.75

A_{bh} = Area of plate excluding diameter of hole inside

f_c' = Ultimate Compressive Strength

R = Constant for ductile member, 1.15

2. *AASHTO and ACI Punching Shear Equation*

$$V_c = \left(0.17 + \frac{0.33}{\beta_c} \right) \sqrt{f_c'} b_o d \dots\dots\dots(3.7)$$

Where:

V_c = Punching Shear Capacity, N

β_c = Ratio of short side to long side of concentrated load

f_c' = Ultimate Compressive Strength, MPa

b_o = Perimeter of critical section for slab and footing, mm

d = Average effective depth of section, mm.

3. *CHBDC Shear Resistance Equation*

$$V_r = \phi_c f_{cr} b_o d \dots\dots\dots(3.8)$$

$$f_{cr} = 0.4 \sqrt{f_c'} \dots\dots\dots(3.9)$$

Where:

V_r = Factored shear resistance, N

β_c = Ratio of short side to long side of concentrated load

f_c' = Ultimate Compressive Strength, MPa

f_{cr} = The cracking strength of concrete, MPa

ϕ_c = The resistance factor of concrete, 0.75

d = Average effective depth of section, mm

4. General Punching Shear Equation

$$V_c = 2 \left(b_1 + b_2 + \frac{2d}{\tan \theta} \right) \frac{d}{\tan \theta} f_t \dots\dots\dots(3.10)$$

$$f_t = \left(0.17 + \frac{0.33}{\beta_c} \right) \sqrt{f'_c} \dots\dots\dots(3.11)$$

Where:

V_c = Punching Shear Capacity, N

b_1 = Short side of concentrated load or reaction area, mm

b_2 = Long side of concentrated load or reaction area, mm

d = Average effective depth of section, mm

β_c = Ratio of short side to long side of concentrated load

f_t = Diagonal tensile strength of concrete, MPa

θ = actual angle between the horizontal and the assumed failure plane

3.4.3 Design for Pre-tensioning

Design of pre-tension required for model M1 to M5 were conducted using procedure presented elsewhere (Collins and Mitchell, 1994). The rationale for the pretensioning the threaded rods was to provide sufficient pre-compressive strength at interface between the bottom of the precast barrier wall and the top surface of the concrete deck slab. This has been achieved by maintaining the actual tensile stress at the serviceability limit state, at the inner face of the barrier at this contact location, to the allowable tensile stress limit of $0.5 \lambda f'_c$ (CHBDC, 2000). As such, CHBDC live load

factor or 1.7 was not included in this elastic analysis of bending stresses. The following equations were used to develop design force:

$$P_f \geq \frac{M_{max} - S_b f_b}{E + k_t} \dots\dots\dots(3.12)$$

$$K_t = S_b / A \dots\dots\dots(3.13)$$

$$\text{Compressive Stress limit} = 0.45 \sqrt{f_c'} \dots\dots\dots(3.14)$$

$$\text{Tensile Stress limit} = 0.5 \lambda \sqrt{f_c'} \dots\dots\dots(3.15)$$

Where:

P_f = Pre-stressing Force in the threaded rod, KN

M_{max} = Maximum bending moment at particular section

S_b = Section modulus, mm³

f_b = Concrete stress at bottom fibre (i.e. tension side of section)

E = Modulus of elasticity

k_t = Tension stiffness of member

A = Cross sectional area of concrete structure, mm²

f_c' = Ultimate compressive strength of concrete

3.4.4 LRFD Yield Line Analysis

The Yield Line Analysis conducted herein was based on the AASHTO-LRFD Bridge Design Specifications (2004) and included only the ultimate flexural capacity of the concrete components. In the analysis, it was assumed that the yield line failure pattern occurs within the barrier wall only and does not extend into the deck slab. This means

that the deck slab must have sufficient resistance to force the yield like failure pattern to remain within the barrier wall. If the failure pattern extends into the deck, the equations for resistance of the barrier wall are not valid. The LRFD yield line analysis is also based on the assumption that sufficient longitudinal length of barrier wall exists to result in the yield line failure pattern shown. For short lengths of barrier walls, a single yield line may form along the juncture of the barrier wall and the deck slab. Such a failure pattern is permissible, and barrier wall resistance should be computed using appropriate analysis. Moreover, the LRFD yield line analysis is based on the assumption that the negative and positive wall resisting moments are equal and that the negative and positive beam resisting moments are equal.

AASHTO-LRFD Design Specifications assume two yield line failure patterns based on the location of the truck collision with the barrier wall. A force F_t distributed over a length L_t as shown in Fig. 3.19 produces the first yield line failure pattern caused by a truck collision within a wall segment. This interior yield line pattern is assumed to have three yield lines. Two of the yield lines have tension on the inside face of the barrier wall and one yield line has tension on the outside face of the barrier wall. The latter is vertical crack along the height of the barrier wall at the location of vehicle impact. The second yield line failure pattern occurs at the end of the barrier wall as produced by a force F_t distributed over a length L_t as shown in Fig. 3.20. In this case, there is only one diagonal yield line that produces tension on the inside face of the barrier. This type of yield line pattern is assumed to occur at bridge barrier ends and at locations of deflection joints and expansion joints. Figure 3.21 shows schematic diagram of the barrier wall with possible locations of yield line failure patterns at the interior and end regions.

A solution is obtained for the barrier wall load carrying capacity (i.e. nominal barrier wall resistance) is obtained by equating the external work due to the applied loads to the internal work delivered by the resisting plastic moments along the yield lines. The resisting moment along the yield line is a resultant of the moment about the vertical axis from the longitudinal reinforcement and (M_w) and the moment about the horizontal axis from the transverse reinforcement (M_c). The angle of the inclined yield lines can be expressed in terms of the critical length L_c as shown in Figs. 3.19 and 3.20. The applied force F_t is then minimized with respect to the L_c to get the least value of this upper bound solution. With some algebraic manipulation based on the principle of virtual work, the critical wall length over which the yield line mechanism occurs, L_c , can be determined. As such, the following equations are introduced.

For impacts within the barrier segment:

The critical barrier wall length over which the yield line mechanism occurs shall be taken as:

$$L_{ci} = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \left[\frac{8 \cdot H \cdot (M_{bint} + M_{wint} \cdot H)}{M_{cint}}\right]} \dots\dots\dots(3.16)$$

The nominal barrier wall resistance to transverse load may be determined using a yield line approach as:

$$R_{wi} = \left(\frac{2}{2 \cdot L_{ci} - L_t}\right) \left(8 \cdot M_{bint} + 8 \cdot M_{wint} \cdot H + \frac{M_{cint} \cdot L_{ci}^2}{H}\right) \dots\dots\dots(3.17)$$

For impacts at end of wall or at joint:

The critical barrier wall length over which the yield line mechanism occurs shall be taken as:

$$L_{ce} = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + H \cdot \left(\frac{M_{bend} + M_{wend} \cdot H}{M_{cend}}\right)} \quad (3.18)$$

The nominal barrier wall resistance to transverse load may be determined using a yield line approach as:

$$R_{we} = \left(\frac{2}{2 \cdot L_{ce} - L_t}\right) \left(M_{bend} + M_{wend} \cdot H + \frac{M_{cend} \cdot L_{ce}^2}{H}\right) \quad (3.19)$$

Where:

L_{ci} = Critical length of interior yield line failure pattern

L_{ce} = Critical length of exterior yield line failure pattern

L_t = Longitudinal length of distribution of impact force F_t

H = height of barrier wall

M_{bint} = Flexural resistance of the cap beam

M_{wint} = Flexural resistance of the wall about its vertical axis

M_{cint} = Flexural resistance of the wall about a horizontal axis

R_{wi} = Total transverse resistance of the barrier wall

For use of the above equations, M_c and M_w should not vary significantly over the height of the wall. Where the width of the concrete wall varies along the height, M_c

should be calculated as the average of its value along the height. The nominal resistance equations include flexural resistance, M_b , when a top beam or beam cap is present at the top of wall. Barrier wall has no additional beam section at its top. M_b term is set equal to zero in the barrier wall resistance computation since this beam cap does not exist in the proposed barrier wall system in the current study. The measurement of system resistance of a concrete barrier wall is R_w , which is compared to the loads in Table 3.1 to determine structural adequacy. In the terminology of AASHTO-LRFD specifications, R_w is the nominal resistance and it is compared to nominal load given in Table 3.2. Bridge engineers can then apply resistance factors to both concrete and reinforcing steel to obtain the factored transverse resistance of the barrier wall. Appendix A presents manual calculation procedure for the proposed barrier wall system based on the design equations presented earlier in this chapter.

Chapter-4

REVIEW OF EXPERIMENTAL STUDY

4.1 Introduction

Based on the information obtained from literature review as well as CHBDC code and AASHTO-LRFD Specifications, precast concrete bridge barrier wall system with pre-tensioned rods was developed as presented in Chapter 3. To verify and substantiate the design procedure and the theoretical model, five full-scale barrier wall models were fabricated in the structures laboratory of Ryerson University and then tested to complete collapse. The experimental program was undertaken to investigate the structural behaviour and ultimate load carrying capacity of the developed barrier wall and its connection with the bridge deck slab. Water leakage test was also performed on one of the tested models to examine the durability level of the barrier wall-to-deck slab joint. In this chapter, description of the barrier wall models is presented including the geometry, material properties, and construction of the models, instrumentation, test equipment, experimental set-up and test procedure.

4.2 Description of Barrier Wall Models

The experimental program was carried out on five actual-scale barrier wall connected to the bridge deck. The choice of model configurations was determined in such a way that experimental failure load of each mode can possibly be related to one of the possible failure modes (i.e. one-way slab flexural failure of the barrier wall, punching shear failure of the threaded rod embedded in the concrete deck slab, tensile fracture of the threaded rod, two-way failure mode based on AASHTO-LRFD yield line patterns).

It should be noted that the current study focus only of PL-3 barrier walls for bridge decks. Geometry of the PL-3 precast barrier wall was taken from Fig. 10.1.3.3 of MTO Structural Manual (MTO, 2003). The depth of the barrier wall measured from top of the wall to top of the concrete deck slab was 1140 mm, including 1050 mm net depth over the 90-mm thick asphalt layer. The bottom and top widths of the barrier wall were taken 475 and 225 mm, respectively. Model M1 represents the proposed precast barrier wall system attached to the concrete deck slab to examine the possibility for one-way slab flexure failure mode of the barrier wall, the punching shear capacity of the threaded rod anchorage in the concrete deck slab and the tensile fracture of the threaded rod. Full description of Model M1 is presented in Figs. 3.3 to 3.5. Its barrier wall was connected to 225 mm thick concrete deck slab of 1200 mm length or width. Per structural design of the barrier wall presented in Appendix A, 15M reinforcing rebar were used as main vertical reinforcement @ 165 mm c-c and as distribution (longitudinal) reinforcement @ 240 mm c-c in a staggered manner. However, the deck slab was reinforced with top main rebar of M20 @ 100 mm c-c., while the bottom main rebar and all distribution (transverse) rebar was taken as M15 @ 250 mm c-c. It should be noted that the 165 mm spacing between vertical rebar could not be maintained with the presence of the 75 mm diameter sleeves every 600 mm spacing. As such, a 135 mm spacing between rebar was used for rebar located between sleeves to maintain the same amount of reinforcement required for wall strength.

Model M2 represented a monolithic cast-in-place connection between the cast-in-place barrier wall and the deck slab. Full description of the model is presented in Figs.

3.6 to 3.8. It should be noted that reinforcing bar arrangements in both the deck slab and the barrier wall was similar to those in model M1.

Models M3 and M4 are intended to examine the load carrying capacity of the 3-m length precast barrier segment when loaded concentrically. Figure 3.9 to 3.12 show views Model M3, while Figs. 3.13 and 3.14 show views of model M4. It can be observed that Model M3 introduced the cantilever part of the deck slab to examine its effect of the structural response and the load carrying capacity of the developed precast barrier wall segment. While the precast barrier wall segment of Model M3 was connected to the concrete deck slab that was tied-down to the laboratory floor to prevent any possible vertical movement, thus forcing the failure to be in the barrier wall or its connection to the deck slab. It should be noted that the cantilever portion of the deck slab of Model M3 had 225 mm thickness and 900 mm net length, while the slab portion that was tied down to the laboratory floor had 375 mm thickness and 600 mm length. However, the deck slab of Model M4 had constant thickness of 225 mm and length of 1200 mm. The deck slab of both models had a width of 3000 mm equal to the length of the precast wall segment under testing. Model M5 was identical to model M4 as detailed in Figs. 3.15 to 3.17. However, model M5 was loaded eccentrically as shown in Fig. 3.16.

4.3 Material Properties

The following subsection summarizes mechanical properties of the materials used to fabricate the barrier models.

4.3.1 Ready Mixed Concrete

A ready-mix concrete having a 3-day specified high early strength of 35 MPa was used for all the barrier models. The cement used in this mix was normal-strength Portland

cement, CSA type 10, manufactured by Canadian Cement Company. The aggregate used in this concrete mix had a maximum nominal size of 20 mm. Three concrete cylinders were filled during pouring concrete for each barrier model. The cylinders were kept beside the models to ensure the same curing conditions after casting. The concrete cylinders were tested under compression on the same day each barrier model was tested to determine the concrete compressive strength. An average of three cylinders was taken for each test of the barrier wall models. Also, 75 mm slump test was marked during concreting of each model. Results of cylinder test results values are summarized in Table 4.1. While Fig. 4.1 shows views of concrete cylinders during casting, capping and testing.

4.3.2 Cement Grout

A ready-mix cement grout having a 3-day specified high early strength of 35 MPa, supplied by Williams forms Inc. of London, Ontario, Canada, were used to fill the sleeves after pre-tensioning the threaded rods. Three cubes of 50-mm width were cast during each grouting stage and then tested after hardening. Results of average cube test values are summarized in Table 4.2. Fig. 4.2 shows views of cement grout cubes during casting, capping and testing, while the top-left part of Fig. 4.3 shows view of the cement grout bag. This type of cement grout was chosen because after grouting, it starts bulging, which provide better grip with pre-tensioned threaded rod. It should be mentioned herein that before laying down the precast barrier wall on the top of the deck slab, a 25 mm thick layer of cement grout was laid over the deck slab, with a width and length equal to the same as bottom face of barrier wall for better privilege of barrier alignment and water leakage resistance

4.3.3 Reinforcing steel

The concrete barrier wall and deck slab models were reinforced using reinforcing steel rebar. The specified yield strength of rebar was taken as 400 MPa. No coupon testing was conducted to stand on the mechanical properties of the used rebar.

4.3.4 Galvanized iron corrugated sleeves

The galvanized metal corrugated duct or sleeve supplied by Williams Forms Inc. of London, Ontario, Canada, was embedded in the concrete deck slab as well as the barrier wall segments at 600 mm spacing to provide holes for the threaded rods to go through the entire depth of the barrier wall system. It should be noted that the used sleeves was of 75 mm internal diameter and it was specified by Williams Forms Inc. as matching the ASTM A653 Standard and meeting all physical and structural recommendations for pre-tensioning duct as laid down by post tensioning institute. The top right portion of Figure 4.3 shows view of the galvanized sleeve used in the current study.

4.3.5 Pre-tensioned full threaded rods

The Pre-tensioned all-threaded-rods of 25 mm diameter, supplied by Williams Forms Inc., were used in the current study. Figure 4.3 shows views of the threaded rod as well as the high strength nut and washer, while Table 4.3 shows the mechanical properties of the threaded rods as supplied by Williams Forms Inc. As supplied by Williams Forms Inc., the specified yield strength and the ultimate strength of threaded rods are 827 and 1034 MPa, respectively. The bars are cold stressed and stress relieved to produce such strengths. All threaded rods were produced to ASTM A-722-98 and AASHTO M275 physical standards. The pre-stressing steel was high in strength yet

ductile enough to exceed the specified elongation of 4% and 20% reduction of area. It also had pass heat of 135° supplemental bend test and also met post tensioned bar criteria as set by the Post-Tensioning Institute including dynamic test requirements beyond 500,000 cycles of load. Also, it had ideal relative rib area configuration to provide excellent bond strength capability to grout far better than traditional reinforcing deformation pattern. The High Strength Nuts are machine-threaded (not cast-threaded) to specific tolerances for precision adjustments and also designed to develop 100% of the all-threaded bar ultimate strength criteria. The high strength nut was produced per ASTM A29, Grade C1045 criteria. It had 25 mm internal diameter and 50 mm in thickness to hold all-threaded-rods. The Hardened Washer was manufactured per ASTM F436 requirements. It was of 25 mm internal diameter and 4 mm in thickness.

4.4 Construction of the models

A 19 mm thick plywood sheets were used to support the concrete for both the barrier wall and deck slabs. For better tie-up supporting system and bracing-up, 35x50 mm size wood studs were used. An electrical saw available at the structural laboratory of Ryerson University was used to cut the wood studs and the plywood sheets. At the top of barrier wall form, trapezoidal cut outs were made to provide means for a recess in the top concrete part of the wall every 600 mm of the length of the barrier wall. Through each recess, the threaded rod will be inserted in the sleeve all the wall to the bottom of the deck slab and then pre-tensioned to the desired initial force. An arrangement was made on the two opposite sides of the recess to have a shear lock system for grouted trapezoidal pocket against pry-out when applying load. Sixty four mm vibration resistance nails and 64 mm screws were used to make a formwork for one-piece barrier wall casting. Also,

15x15 mm galvanize iron small angle clips were screw down to hold-corrugated duct, which was imbedded into barrier wall and deck slab. After completion of formwork, all the joints were sealed with duct tap. Before placing of the reinforcement mesh, all the plywood surfaces, which were in contact with concrete, were covered with biodegradable concrete form releasing agent. A paintbrush applied the concrete releasing agent and excessive amount of agent was wiped out immediately. Figures 4.4 to 4.9 show selected views of the formwork for all models in addition to the formwork for the recesses at the top of the sleeves. While

Galvanized iron corrugated ducts of 75 mm were installed at 600 mm intervals in the formworks of both the barrier wall and the deck slab to allow for passing through the pre-tensioned threaded rods. Ducts were cut in size and aligned at each location as per design of pre-tensioning system. Small steel angle clips of 15x15 mm were used to hold ducts from top and bottom side of the barrier wall. Figure 4.7 show how the ducts were supported at the top and bottom of the barrier wall, while Fig. 4.10 show view of how the ducts were fixed to the bottom of the formwork of the concrete deck slab. All reinforcing steel rebar were cut and bent off campus facility. Then, all rebar were tie-up inside the formwork. Plastic cover blocks were used to provide cover to the reinforcement. Plastic straps were used to tie-up each longitudinal and transverse rebar. It should be noted that before putting rebar into the forms, steel strain gauges were attached to specified rebar at specified locations. Figures 4.11 to 4.21 show selected views of the formwork with final set of the reinforcing steel mesh and sleeves for all models.

High early strength concrete of a specified compressive strength 35 MPa was used for all models. Ready mix concrete was order and poured in the barrier wall using conical

shaped bucket having capacity of 0.5 m^3 , driven by the overhead crane available at Ryerson structures laboratory. Each barrier wall was poured in two steps. In the first step, a layer of 600 mm depth of concrete was poured and then, concrete was consolidated using 20 mm diameter electric vibrator. In the second step, the rest of the barrier wall was poured and then vibrated. Finally, by using different levelling equipments, top exposed surface of model was finished. After 24 hours, formwork from each side of the barrier wall and the deck slab was removed. Then, Models were covered with jute and water was sprinkled to keep it wet during the curing time.

To connect the barrier wall and the deck slab together at the test spot in the structures lab, the deck slab was laid over the laboratory floor. Then a frame of 25x25 mm-wood studs was formed over the top of the deck slab at the footprint of the barrier wall. Then, ready-mix cement grout of 35 MPa was poured inside the frame. During pouring the cement grout, all the corrugated ducts imbedded into deck slab were plugged. As soon as the initial setting time started for cement grout, barrier wall segment was placed over the cement grout. Then, wooden frame was removed. Figure 4.22 shows view of this framing after pouring the cement grout for model M1. While Fig. 4.23 shows view of the barrier wall of model M1 after being laid over the cement group. Figure 4.25 shows side view of the cement grout at the barrier-slab joint. After placing the barrier wall on the top of deck slab, 3 days time was given to cement grout to rich its full strength of 35 MPa. Each threaded rod was then placed inside each duct and kept in location using steel bearing plate of 150x150x16 mm and a nut and a washer at the top face of barrier wall inside of the pocket, followed by similar setup at the bottom end of the threaded rod. It should be noted that at least 15 mm of threaded portion of rod was

kept projecting outside of nut at the bottom of the rod to get a full grip. At the top end of the rod. A design torque of 525 ft-lb was applied, which is equivalent to transfer 79 KN of pre-tensioned force to the threaded rod. This pre-tension force is equivalent to 17% of the nominal ultimate tensile strength of the threaded rod. Equivalency in terms of lb-ft from KN was evaluated experimentally by conducting four consecutive torque tests on a load cell available in structural laboratory. Results of each test average out are summarized in Table 4.4. Views of this test are shown in Figs. 4.66 and 4.67.

After pre-tensioning all rods, cement grout with one part of cement grout mix with four parts of water was properly mixed and dripped inside the duct through a hole drilled in the top steel bearing plate. After all ducts were filled with grout, the grout was left for 24 hours to get settled and then filled again with grout if the duct was not completely filled during that time. Finally, each recess or trapezoidal pocket was clamped by laminated plywood from the inner and outer side of the barrier wall and then filled with concrete grout. Figures 4.23 and 4.24 show views of the recesses in model M1 before and after filled it with concrete grout. Figures 4.26 and 4.27 show views of models M4 and M3, respectively, in their final position for testing. While Figs. 4.68 and 4.69 show views of trapezoidal bracket after filling with cement grout.

4.5 Instrumentation

To measure the longitudinal strain in the steel rebar at critical locations in the barrier wall during testing, electrical strain gauges, type KGF-10-120-c1-11, were used. The length of steel strain gauge was 10 mm with a resistance of $120 \pm 2\%$ ohms, and a gauge factor The strain gauge were mounted on the rebar and protected using M-PREP kit, strain gauge protective coating/ Environmental protection kit, which contain M-PREP

Neutralizer, M-PERT Conditioner A, 200 Catalyst-C, M-Bond 200 Adhesive, Silicon sealant, aluminium tape. View of the steel strain gauge kit and its accessories is shown in Fig. 4.54. Figure 4.28 shows the arrangement of steel strain gauges for model M1 where these gauges were located in the vertical rebar on the tension side of the barrier wall. Similar arrangement was considered for Model M2 as shown in Fig. 4.32. Models M3 had two sets of steel strain gauges, horizontally and vertically. Figures 4.35 and 4.36 show arrangement of strain gauges instrumented on horizontal rebar at the inner face and outer face of the barrier wall, respectively. While Fig. 4.37 shows arrangement of steel strain gauges on the vertical rebar at the inner side of the barrier wall. It should be noted that steel strain gauges were also installed on the threaded rods at the interface between the deck slab and the barrier wall as shown in Fig. 4.39. It should be noted that model M4 has identical arrangement of steel strain gauges at the mid-length of the barrier wall as that for model M3 as depicted from Figs. 4.42, 4.43, 4.44 and 4.45. Model 5 had similar arrangement of steel strain gauges but located near the end of the barrier wall where the impact load strikes the barrier wall. This can be depicted in Figs. 4.48, 4.49 and 4.45 for model M5.

In order to measure vertical strain on the compressive side of barrier wall during static load testing, electric strain gauges, type N11-FA-60-120-11, were used. The concrete strain gauges had a length of 60 mm, a resistance of $120 \pm 0.3\%$ ohms, and a gauge factor of $2.12 \pm 1\%$. The concrete surface was prepared to mount the strain gauges using M-Bond AE-10 adhesive kit, which contained M-Bond adhesive resin type AE and M-Bond type-10 curing agent. Figure 4.54 shows views of the strain gauge packages and accessories used to stick them to the concrete surface. These gauges were attached to

specified concrete surface locations. Small area of each location, about 80 mm in length and 30 mm in width, was scrubbed and made smooth using grit sand paper. Before attaching the strain gauge to the scrubbed area, the surface made neutral by using a neutralizer. Strain gauges were then attached to the concrete surface using M-Bond 200 epoxy after cleaning the surface with acetone and conditioner and making the surface neutral using neutralizer. Each strain gauge was protected by m-Coat F kit and finally covered with thick aluminium tape. Figures 4.30 and 4.32 present a schematic diagram of the barrier wall with the concrete strain gauges located at the lowest part of the outer face of the precast barrier wall for Models M1 and M2, respectively. Figures 4.40, 4.46, and 4.52 shows identical arrangement of concrete strain gauges for models M3, M4 and M5 respectively.

Linear Variable Displacement transducers (LVDTs), of 150 mm electrical stock, were used to measure movements of the barrier wall system during testing. Figure 4.55 shows view of the LVDT used in testing. Figures 4.31, 4.34, 4.41, 4.47 and 4.53 present schematic diagrams for the barrier wall system with the arrangement of the LVDTs for models, M1, M2, M3, M4 and M5, respectively. Lateral deflection of the barrier wall was recorded using LVDTs oriented horizontally against the outer face of the wall and opposite to the direction of the applied load. Since each model was tied-down to the laboratory floor and the static load was applied horizontally, there was an issue of possible global horizontal movement of the barrier wall in the direction of the applied load. As such, steel rings, shown in Fig. 4.64, were fabricated and then inserted on the top portion of the holes of the laboratory floor to have no gap between the tied-down threaded rods and the floor. Other lateral resisting system made of HSS steel beam and

solid round steel rods, shown in Fig. 4.65, was fabricated and then installed on the outer side of the deck slab to assist on preventing its lateral movement during testing. To observe possible lateral movement of the deck slab, an LVDT was installed on the outer face of the deck slab and oriented horizontally against the direction of the applied load. As it is shown in Fig. 4.58, the deck slab was tied down near its back end to prevent uplift under applied horizontal load. To observe possible up lift, an LVDT was installed vertically at the back edge of the deck slab as shown in Fig. 4.58.

4.6 Test Setup for test procedure for ultimate load test

Figure 4.58 shows a schematic diagram of the test setup. Each barrier wall model was supported over the laboratory floor of structural laboratory. Then, it was tied down to the floor using 50 mm diameter threaded rods. Each rod was placed @ 600 mm c-c and tightened by applying a torque of 525 lb-ft, which is equivalent to 79 kN, to control vertical movement of the deck slab during application of load. As stated earlier, two different systems, Figs. 4.64 and 4.65, were developed to hold the entire barrier wall model against horizontal movement during load application. In first system, each tie-down threaded rod passed through a 20 mm thick steel plate of 50 mm internal diameter and 90 mm outer diameter. While the second system consisted of an HSS box member assembly that was installed against the outer face of the deck slab. LVDTs were then installed to measure deflection of barrier wall and vertical and horizontal movements of deck slab during load application. Concrete strain gauges were then installed as required. All strain gauges were checked for resistance before connecting them to the system using electric wire. A hydraulic jack having a capacity of 900 kN was used for the application of the load. The jack was mounted and strapped down with two heavy metal pedestal

supported over the deck slab. The vertical concrete wall took the reaction from the jack available in the structures laboratory. A universal flat load cell of 900 kN capacity was used to measure the applied loads on barrier models. Figure 4.57 shows view of this loading system. Before testing, LVDTs, strain gauges and load cells were connected to SYSTEM 6000 data acquisition unit to record readings. Figure 4.56 shows view of the data acquisition unit used for testing.

Each model was tested under increasing monotonic load up-to-collapse. For models M3 and M4, a concentric transverse load was applied over 150x150 mm surface area using HSS box section at the mid-length of the barrier wall. Similar arrangement was done for model M5 but with eccentric transverse load located the end of the barrier wall. For models, M1 and M2, Uniformly distributed lateral load was applied to the barrier wall using an HSS box member over an area of 1200 mm length and 150 mm width. Trapezoidal gap between vertical face of box member and the inclined front face of barrier wall was filled by a wooden plank, which was profiled in a trapezoid shape using electric saw machine available at Ryerson structural laboratory. All transverse loads were applied to the inner face of the barrier wall at a 990 mm height measured from the top of the concrete deck slab. During the test, was applied in increments of 10 kN. After each increment, the load was maintained for few minutes to observe crack initiation and propagation as well as change in bridge geometry as depicted from LVDT readings. It should be noted that the data acquisition unit was adjusted to record a reading per second from all sensors. Failure of the model was attained when the reading from sensors were increasing while the model did not take any further increase in load. After the failure of each model, the applied load was released slowly. Figures 4.59, 4.60, 4.61,

4.62, and 4.63 show views of barrier models M1, M2, M3, M4 and M5, respectively, during testing.

Chapter- 5

REVIEW OF EXPERIMENTAL RESULTS

5.1 Introduction

The experimental program reported herein was intended to study the structural behaviour and ultimate load carrying capacity of the developed precast barrier wall system. This includes load-deflection and load-strain histories, crack patterns and failure modes. In order to achieve the above objectives, experimental tests were conducted on five full-scale barrier models. The first two connections were of 1.2-m length. One of which represented the currently used cast-in-place PL-3 barrier wall system, while other one represented the developed precast barrier wall system. The other three models represented the developed precast barrier segment with 3 m length. One of which was connected to the cantilever portion of the deck slab, while the second one was connected to a non-deformable deck slab. Both models were loaded concentrically. The last model was similar to the fourth model but it was loaded at its wall edge. The following section presents a summary of the experimental results on the five models as well as the water-leakage test. Then, correlation between the experimental findings and theoretical ultimate load capacities of the tested models is presented. It should be noted that experimental results of ultimate load tests are presented in Appendix B.

5.2 Water-leakage test

The developed precast barrier wall system is examined for durability by conducting water-leakage test. The design requirements of the precast barrier include preventing water leakage between the barrier wall and the deck slab over thru traffic

under the bridge in the winter times. As such, threaded rods connecting the precast barrier to the deck slab were pre-tensioned to 17% of the tensile capacity to ensure no tension at the interface at the barrier-slab joint. Water-leakage test was conducted on model M5. One layer of 50 mm thick brick wall was constructed on the top of deck slab as shown in Fig. 5.1 to form a shallow water tank with the precast barrier wall as one side of this tank. After constructing the brick wall, all cement joins between adjacent bricks were filled by Polysulfide sealant, to resist water leakage. After sealant treatment, Synthetic (Oil Based) paint was applied three times to the bottom and outer sides of the tank, leaving the side with barrier wall cement grout unpainted. This multi-layer paint was required to make sure that water would not leak from the tank or would not be absorbed by the top surface of the deck slab or the inner sides of the brick wall. As such, water can leak only through barrier wall-to-deck slab junction. To start the test, the tank was filled with water in such a way to submerge the 25 mm cement grout at the barrier-wall-to-deck slab junction. Water level in the tank as well as outer sides of barrier wall-to-deck slab junction was periodically observed for 7 days. After seven days, it was observed that no water leakage was found as shown in Figs. 5.1 and 5.2.

5.3 Structural behaviour and failure mode of Model M1

As mentioned before barrier model M1 emulated the precast barrier wall with its pre-tensioned connection with the deck slab. The length of the barrier wall was taken as 1200 mm to study the resistance of the joint between the barrier and the deck slab. Testing to-collapse of barrier model M1 began by applying the lateral load in increments of 10 kN. At each load increment, possible cracks were observed visually. At load level of 60 kN, the first concrete crack was observed at the front side of the barrier wall

between the bottom surface of the barrier and top surface of the cement grout as depicted in Figs. 5.3 and 5.4. With increase in the applied load, a major horizontal crack was developed approximately at the mid-depth of the deck slab on the left side of the barrier wall as depicted in Fig. 5.3, leading to an anchorage failure under the barrier wall. This major anchorage failure was also exhibited on the right side of the barrier wall as depicted in Fig. 5.4, but associated with combined flexure-shear-bond crack pattern. Other view of this combined crack pattern is shown in Fig. 5.5. Concrete major crack associate with threaded rod anchorage into concrete slab was formed as shown in Fig. 5.5 in the deck region below the barrier wall. The anchorage crack extended into the slab away from the front side of the barrier wall in a combined tension-shear-bond crack pattern. Nevertheless, the specimen failed mainly due anchorage major crack in the deck slab at 174.61 KN. The test was terminated when the specimen could not absorb any further load. It should be noted that no cracks were recorded in the precast barrier wall.

During testing model M1, readings from sensors were recorded at a rate of 1 reading per second. However, selecting few points on the recorded data to reduce graph capacity herein produced the presented graphs. Figure 5.23 shows the relationship between the applied load and the strain in the vertical rebar at the junction of the inclined surfaces of the front face of the barrier wall. One may notice that the maximum strain reached at peak load was about 86.21 micro-strains which are considered very low. This may be attributed to the contribution of the threaded rods at 600 mm spacing in resisting the applied bending moment on the barrier wall. Figure 5.24 depicts the load-strain relationship in the pre-tensioned threaded rod. It can be observed that the threaded rod started to catch axial tensile forces at a load level of about 30 kN. The strain at peak load

was recorded as 2625 micro-strain, which is far below the yield strain of 4135 micro-strain. Figure 5.39 depicts the load-concrete strain relationship at the bottom rear side of the barrier wall for model M1. It can be observed that the maximum strain recorded at peak load was about 1174.65 micro-strains. No sign of concrete crushing was observed. Figure 5.44 shows the load-lateral deflection relationship of the barrier wall at load location for model M1. It can be observed that the maximum-recorded deflection was 43.39 mm at peak load.

5.4 Structural behaviour and failure mode of Model M2

Barrier model M2 was considered as a control specimen since it represented the cast-in-place barrier wall system. Testing to-collapse of barrier model M2 began by applying the lateral load in increments of 10 kN. At each load increment, possible crack were observed visually. At load level of 70 kN, the first concrete crack was observed at the front side of the barrier wall between the bottom surface of the barrier and the deck slab as depicted in Fig. 5.6. With increase in the applied load, a combined flexure-shear-bond crack appeared at 170 kN as depicted in Figs. 5.6 and 5.7 for the left and right side of the barrier wall, respectively. Fig. 5.6 shows also a major crack in the slab under the barrier wall running parallel to and at the location of the horizontal development length of the barrier wall tension reinforcement embedded into concrete slab. However, this type of crack pattern did not appear on the right side of the specimen. The specimen failed at a load level of 182 kN when the specimen could not absorb any further load. It should be noted that no cracks were recorded in the cast-in-place barrier wall.

During testing Model M2, readings from sensors were recorded at a rate of 1 reading per second. However, selecting few points on the recorded data to reduce graph

capacity herein produced the presented graphs. Figure 5.25 shows the relationship between the applied load and the strain in the vertical rebar at the junction of the inclined surfaces of the front face of the barrier wall. One may notice that the maximum strain reached at peak load was about 823.06 micro-strains, which is considered low compared to the yield strain of 2000 micro-strain. Figure 5.40 depicts the load-concrete strain relationship at the bottom rear side of the barrier wall for model M2. It can be observed that the maximum strain recorded at peak load was about 305.33 micro-strains. No sign of concrete crushing was observed. Figure 5.45 shows the load-lateral deflection relationship of the barrier wall at load location for model M2. It can be observed that the maximum-recorded deflection was 14.63 mm at peak load.

5.5 Structural behaviour and failure mode of Model M3

Model M3 represented a 3-m length precast barrier wall connected to the cantilever portion of the deck slab using pre-tensioned rods. Testing to-collapse was performed in this barrier model by applying a horizontal load over a 150×150 mm patch area located at the mid-length of the barrier wall and at 990 mm from top of the deck slab. The load was applied in 10 kN increments to allow for observing crack initiation and propagation. When the applied load reached 50 kN, a flexural crack appeared at the top surface of concrete at the fixed end of the cantilever portion of the deck slab as shown in Figs. 5.10 and 5.11 for the right and left sides of the deck slab, respectively. Also, at load level of 150 kN, a horizontal concrete crack was observed between the bottom of the precast barrier wall and the top surface of the cement grout. With increase in applied load, a major anchorage crack appeared on the deck slab under the barrier wall on the right and left sides of the specimen as depicted in Figs. 5.9 and 5.11, respectively. Figure

5.8 shows view of these combined crack pattern on the right side of the specimen. It was observed that this major anchorage crack failed the specimen at a load level of 313 KN. When the crane from the test site lifted the specimen, it was observed that the bottom surface of the cantilever portion of the deck slab experienced big lump of concrete pry-out between threaded rod anchorage locations. This concrete spalling was observed at two locations as shown in Fig. 5.12. A close-up view of one these spalled areas is shown in Fig. 5.13. While Fig. 5.14 shows views of the spalled concrete found on top of the lab floor. One may observe that the spalled concrete area extended from the fixed end of the cantilever slab to the nearest edge of the anchorage steel plates as shown in Fig. 5.13.

During testing model M3, readings from sensors were recorded at a rate of 1 reading per second. However, selecting few points on the recorded data to reduce graph capacity herein produced the presented graphs. Figure 5.26 shows the relationship between the applied load and the strain in the vertical rebar at the junction of the inclined surfaces of the front face of the barrier wall. One may notice that the maximum strain reached at peak load was about 129.23 micro-strains, which are considered very low compared to the yield strain of reinforcing steel of 2400 micro-strains. This may be attributed to the contribution of the threaded rods at 600 mm spacing in resisting the applied bending moment on the barrier wall. One may notice that the axial strain in the rebar is getting smaller towards the ends of the barrier wall as shown in Fig. 5.26. Figures 5.27 and 5.28 depict the stain history on the horizontal rebar along the depth of the barrier wall at the applied load location. It can be observed that rebar horizontal strains were getting larger towards the top of the barrier wall. Also, it was observed that the strains on the front face of the barrier wall were in compression while those on the rear face were in

tension, as expected. Figure 5.29 depicts the load-strain relationship in the pre-tensioned threaded rod. It can be observed that the threaded rod started to catch axial tensile forces at a load level of about 100 kN. The maximum strain at peak load was recorded as 2626.64 micro-strains, which are below the yield strain of 4135 micro-strain. Figure 5.30 shows the load-strain relationship in the top rebar at the fixed end of the cantilever portion of the deck slab. One may observe that the maximum strain at peak load was about 2285.40 micro-strains, which is slightly less than the yield strength of 2400 micro-strain. Figure 5.41 depicts the load-concrete strain relationship at the bottom rear side of the barrier wall for model M3. It can be observed that the maximum strain recorded at peak load was about 627.76 micro-strains. No sign of concrete crushing was observed. Figure 5.46 shows the load-lateral deflection relationship of the barrier wall at load location for model M3. It can be observed that the maximum-recorded deflection was 41.67 mm at peak load.

5.6 Structural behaviour and failure mode of Model M4

Model M4 was identical to Model M3 but without the cantilever portion of the deck slab. Testing to-collapse of this model began by applying the load in 10 kN increments. When the load reached 130 kN, flexural crack line was observed in the cement grout at the barrier-to-slab joint as shown in Fig. 5.15. With increase in applied load, vertical flexural crack appeared on the rear side of the precast barrier wall just at the location of the applied load as shown in Fig. 5.17. With further increase in applied load, major anchorage crack was observed in the deck slab under the barrier wall as shown in Fig. 5.16, leading to specimen failure at a 279 kN horizontal load. After failure, it was observed that the flexural crack opening occurred in the cement grout between the barrier

wall and deck slab, it was wider at the mid-length of the barrier wall and getting thinner towards barrier wall ends.

During testing model M4, readings from sensors were recorded at a rate of 1 reading per second. However, selecting few points on the recorded data to reduce graph capacity herein produced the presented graphs. Figure 5.31 shows the relationship between the applied load and the strain in the vertical rebar at the junction of the inclined surfaces of the front face of the barrier wall. One may notice that the maximum strain reached at peak load was about 136.99 micro-strains, which is considered very low compared to the yield strain of reinforcing steel of 2400 micro-strain. This may be attributed to the contribution of the threaded rods at 600 mm spacing in resisting the applied bending moment on the barrier wall. One may notice that the axial strain in the rebar is getting smaller towards the ends of the barrier wall as shown in Fig. 5.31. Figures 5.32 and 5.33 depict the strain history on the horizontal rebar along the depth of the barrier wall at the applied load location. It can be observed that rebar horizontal strains were getting larger towards the top of the barrier wall. Also, it was observed that the strains on the front face of the barrier wall were in compression while those on the rear face were in tension, as expected. Figure 5.34 depicts the load-strain relationship in the pre-tensioned threaded rod. It can be observed that the threaded rod started to catch axial tensile forces at a load level of about 30 kN. The maximum strain at peak load was recorded as 2635.75 micro-strains, which are below the yield strain of 4135 micro-strain. Figure 5.42 depicts the load-concrete strain relationship at the bottom rear side of the barrier wall for model M4. It can be observed that the maximum strain recorded at peak load was about 976.22 micro-strains. No sign of concrete crushing was observed. Figure 5.47 shows the load-

lateral deflection relationship of the barrier wall at load location for model M4. It can be observed that the maximum-recorded deflection was 44.28 mm at peak load.

5.7 Structural behaviour and failure mode of Model M5

Model M5 was identical to model M4 except that the applied load was located at edge of the barrier wall simulating vehicle impact at the end of the bridge barrier wall. Testing to-collapse of the specimen started by applying the load in 10 kN increments. At a load approaching 150 kN, few diagonal cracks were observed in the front face of the barrier wall as depicted in Fig. 5.18. The farthest crack was at distance 1500 and 800 mm from barrier end at top and bottom of wall, respectively. With increase in applied load, signs of local punching shear cracks appeared around the loaded area at a load of about 190 kN. This punching shear crack patterns was observed on the facing and side wall of the barrier wall as shown in Fig. 5.19 and on the top surface of the barrier wall as depicted in Fig. 5.20, extending vertically on the rear face of the barrier wall at a distance of about 600 mm from barrier end as shown in Fig. 5.21. The specimen then absorbed further load up to 210 kN. In spite the fact that diagonal cracks appeared on the front face of the barrier wall that would have led to diagonal yield line failure similar to AASHTO-LRFD crack pattern, premature local failure due to punching shear at load location occurred.

During testing model M5, readings from sensors were recorded at a rate of 1 reading per second. However, selecting few points on the recorded data to reduce graph capacity herein produced the presented graphs. Figure 5.35 shows the relationship between the applied load and the strain in the vertical rebar at the junction of the inclined surfaces of the front face of the barrier wall. One may notice that the maximum strain

reached at peak load was about 621.08 micro-strains, which are considered low compared to the yield strain of reinforcing steel of 2400 micro-strain. This may be attributed to the contribution of the threaded rods at 600 mm spacing in resisting the applied bending moment on the barrier wall. One may notice that the axial strain in the rebar is getting smaller towards the other end of the barrier wall as shown in Fig. 5.35. Figures 5.36 and 5.37 depict the stain history on the horizontal rebar along the depth of the barrier wall at the applied load location. It can be observed that rebar horizontal strains were getting larger towards the top of the barrier wall. Also, it was observed that the strains on the front and rear faces of the barrier wall were fluctuating between compressive and tensile strain in a very small magnitude (i.e. ± 50 micro-strain) for applied load up to about 100 kN for the rear face and 150 kN for the front face. Then, with increase in load beyond 150 kN, steel strains increasing considerably till reaching about 1400 and 800 micro-strain in tension for the rear and front faces, respectively, at the peak load, as a result of the local punching shear cracks at around the load location. Figure 5.38 depicts the load-strain relationship in the pre-tensioned threaded rod. It can be observed that the threaded rod started to catch axial tensile forces at a load level of about 70 kN. The maximum strain at peak load was recorded as 2067.91 micro-strains, which are below the yield strain of 4135 micro-strain. Figure 5.43 depicts the load-concrete strain relationship at the bottom rear side of the barrier wall for model M5. It can be observed that the maximum strain recorded at peak load was about 356.75 micro-strains. No sign of concrete crushing was observed. Figure 5.48 shows the load-lateral deflection relationship of the barrier wall at load location for model M5. It can be observed that the maximum-recorded deflection was 21.53 mm at peak load.

5.8 Comparisons between tested models

When preparing the setup, effects caused by any slippages between anchor rods at the locations of tie-down of the deck slab and associated anchor holes in the laboratory strong floor have been eliminated as presented in Chapter 4. Also, the deck slab was prevented for possible uplift at its end away from the barrier wall using tie-down rod system as presented in Chapter 4. Few LVDTs were oriented vertically and horizontally at these two locations to record possible slab movement during testing. Figure 5.49 depicts the slab lateral movement in the direction of the applied load for all models. It can be observed that the maximum lateral movement was very small in the order of less than 0.1 mm for models M3, M4 and M5. The maximum lateral movement was recorded as 2.1 and 0.8 mm for models M2 and M1, respectively, at peak load. These values are considered very small and would not affect the structural behaviour of the barrier wall.

5.8.1 Models M1 and M2

Model M1 and M2 were constructed of 1.2-meter barrier length and load was applied over entire length in a uniformly distributed manner. Model M1 had a precast barrier wall connected to the deck slab with pre-tensioned threaded rods. While, Model M2 replicates the cast-in-place barrier wall currently used in Ontario highway bridges. At load level of 60 kN, the first concrete crack in model M1 was observed at the front side of the barrier wall between the bottom surface of the barrier and top surface of the cement grout. This crack appeared at 70 kN in case of model M2. Model M1 failed due to major anchorage failure in the deck slab under the barrier wall associated with threaded rod anchorage into concrete slab but associated with combined flexure-shear-bond crack pattern in the deck slab near the front face of the barrier wall at an ultimate load of

174.61 kN. While model M2 failed at an ultimate load of 182.00 kN with a combined flexure-shear-bond crack pattern in the deck slab, associated with other local crack in the slab under the barrier wall running parallel to and at the location of the horizontal development length of the barrier wall tension reinforcement embedded into concrete slab. In both models M1 and M2, no cracks were recorded in the barrier wall. Figure 5.51 shows the relationship between the applied load and the maximum strain in the vertical rebar at the junction of the inclined surfaces of the front face of the barrier wall for all models. One may notice that the maximum strain reached at peak load was about 86.21 micro-strains, which are considered very low as compared to 823.06 micro-strains for model M2. This may be attributed to the contribution of the threaded rods at 600 mm spacing in resisting the applied bending moment on the barrier wall. These pre-tensioned threaded rods stated to catch axial tensile forces at a load level of about 30 kN as depicted in Fig. 5.52. Figure 5.53 depicts the load-concrete strain relationship at the bottom rear side of the barrier wall for all models. It can be observed that the maximum strain recorded at peak load was about 1174.65 micro-strain for model M1 and 305.33 micro-strains for model M2. No sign of concrete crushing was observed in both models. Figure 5.54 shows the load-lateral deflection relationship of the barrier wall at load location for all models. It can be observed that the maximum-recorded deflection was 43.39 and 14.63 mm at peak load for models M1 and M2, respectively. To qualify the developed precast barrier wall system, it can be evaluated to be "as good as" the cast-in-place barrier wall that is currently used in Ontario highway. In this sense, it can be observed that the ultimate load carrying capacity of model M2 was 4 % more than model M1. While the deflection of model M1 was almost 34 % more than that of model M2. As such, one may

qualify the developed barrier wall for strength or ultimate limit state design as per CHBDC design provisions. However, since the maximum deflection of model M1 was significantly more than that for model M2, one may assume that the developed precast barrier wall system is more ductile than the cast-in-place barrier wall. This may be advantageous for vehicle impact to absorb more energy resulting from vehicle impact. Nevertheless, this comparison for serviceability limit state may be waived since CHBDC does not specify design provisions for serviceability limit state design of barrier wall other than conducting crash testing.

5.8.2 Models M3 and M4

As stated earlier, Model M3 and M4 were constructed of 3-m length precast barrier wall which was connected to concrete deck slab using pre-tensioned threaded rods at 600 spacing. The only difference between these two models was that model M3 has an 800 mm cantilever portion projecting from the deck slab while model M4 has a non-deformable deck slab. This would assist in studying the effect of deformation of the deck slab cantilever on barrier wall behaviour. It should be noted that researchers used to connect the barrier wall to non-deformable deck slab so that the failure is forced to happen in the barrier wall. Also, the yield-line failure patterns specified in AASHTO-LRFD for ultimate resistance of barrier walls assumed rigid connection between the barrier wall and a non-deformable deck slab. It should be noted that the applied load on models M3 and M4 was identical, concentric patch load over 250x250 mm-surface area located at the mid-length of the barrier wall and at 990 mm over the deck slab.

In model M3, when the applied load reached 50 kN, a flexural crack appeared at the top surface of concrete at the fixed end of the cantilever portion of the deck slab. At load level of 150 kN, a horizontal concrete crack was observed between the bottom of the precast barrier wall and the top surface of the cement grout, while this flexural crack was observed at 130 kN for model M4. Model M3 failed due to major anchorage crack on the deck slab under the barrier wall at a load level of 313 kN. Model M4 failed due to similar anchorage failure after forming major vertical crack on the rear side of the precast barrier wall just at the location of the applied load at load level of 279 kN. This means that the deformation in the cantilever slab increase the ultimate load carrying capacity of the barrier fixed to non-deformable slab by 12%. The favourable presence of the cantilever slab in model M4 is that it delayed the occurrence of the major vertical yield line at the location of applied load and on the rear side of the barrier wall of model M4. Figure 5.51 shows the relationship between the applied load and the maximum strain in the vertical rebar at the junction of the inclined surfaces of the front face of the barrier wall for all models. One may notice that the maximum strain reached at peak load was 129.23 and 136.99 micro-strain for models M3 and M4, respectively, which is considered very low compared to the yield strain of reinforcing steel of 2400 micro-strain. Figure 5.52 depicts the load-strain relationship in the pre-tensioned threaded rod for all models. It can be observed that the threaded rod started to catch axial tensile forces at a load level of about 100 and 30 kN, for models M3 and M4, respectively. The maximum strain at peak load was recorded as 2626.64 and 2635.75 micro-strain, for models M3 and M4, respectively, which is below the yield strain of 4135 micro-strain. Figure 5.53 depicts the load-maximum concrete strain relationship at the bottom rear side of the barrier wall for

all models. It can be observed that the maximum strain recorded at peak load was about 627.76 and 976.22 micro-strain for models M3 and M4, respectively. No sign of concrete crushing was observed. Figure 5.54 shows the load-lateral deflection relationship of the barrier wall at load location for all models. It can be observed that the maximum-recorded deflection was 41.67 and 44.28 mm at peak load for models M3 and M4, respectively. It can also be observed that M3 exhibited less lateral deflection at top of the barrier wall than that for model M4 at all load levels. However, the change of deflection values of about 6% at peak load does not give an indication of the significance of the presence of the cantilever slab to support the barrier wall. Figures 5.55 and 5.56 depict the maximum strain history on the horizontal rebar along the depth of the barrier wall at the applied load location for models M3 and M4. It can be observed that the presence of the cantilever portion in the deck slab did not significantly affect the maximum horizontal strain in the top rebar of the barrier wall.

In general, a rigid conclusion cannot be reached with respect to the effect of the presence of the cantilever portion on the crack pattern at failure since both models failed primarily due to anchorage failure in the deck slab for the pretension rods. However, one may conclude that to force the barrier wall to have more cracks or yield-line failure patterns, it should be connected to a rigid, non-deformable, slab base. In any case, the presence of the cantilever portion of the deck slab to support the precast barrier wall provides more load carrying capacity unless the failure occurs in the deck slab itself due to combined tension and flexure loading.

5.8.3 Models M4 and M5

As indicated earlier, models M4 and M5 were identical except on location of the applied load. Model M4 was loaded using concentric patch load over 250x250 mm-surface area located at the mid-length of the barrier wall and at 990 mm over the deck slab. Model M5 was loaded similarly but with the patch load located at the end of the barrier wall. An experimental study indicates that the ultimate load carrying capacity of model M4 was 24 % more than model M5, as expected. While the deflection of model M4 was almost 51 % more than that of model M5. Model M4 failed due to threaded rod anchorage failure in the deck slab under the barrier wall after forming major vertical crack on the rear side of the precast barrier wall just at the location of the applied load at load level of 279 kN. However model M5 failed in a different way due to the change in the location of the applied load. In this model few diagonal cracks were observed in the front face of the barrier wall at a load level less than the ultimate load. The appearance of this diagonal crack pattern on the front face of the barrier wall gave the impression that the AASHTO-LRFD diagonal crack pattern would be achieved experimentally. However, premature local failure due to punching shear at load location occurred at a load level of 210 kN.

5.9 Correlation between experimental findings and design loads

CHBDC specifies a specified horizontal load of 210 kN to be applied over a length of 2400 mm with 42° dispersal angle of the load from 990 mm height to the top level of the deck slab. Considering a live load factor of 1.7, the factored ultimate horizontal load would be 85.35 kN per meter length of the barrier at the barrier-deck slab junction. As such for models M1 and M2 of 1200 mm barrier length, the design factored

ultimate horizontal load is 102.42 kN. It can be observed the experimental ultimate load of models M1 and M2 are 70 % and 78% more than the CHBDC design factored ultimate horizontal load. In a similar sense, the CHBDC factored ultimate horizontal load at the barrier-deck slab junction for models M3 and M4 would be 173.5 kN, considering the loaded barrier length of 250 mm as conducted experimentally. As such, it can be observed that the experimental ultimate load for models M3 and M4 are 80% and 61% more than the CHBDC factored ultimate horizontal load. In case of vehicle crash to end portion of the barrier wall as the case of model M5, CHBDC specifies a specified horizontal load of 210 kN to be applied over a length of 2400 mm with 48° dispersal angle of the load from 990 mm height to the top level of the deck slab. Considering a live load factor of 1.7, the factored ultimate horizontal load per meter length of the barrier at the barrier-deck slab junction would be 102.00 kN. Since the applied horizontal load in model M5 was over a 250 mm barrier length, the corresponding length of the barrier wall affected by the 48° dispersal angle would be 1349.50 mm. Thus, the corresponding CHBDC factored ultimate horizontal load would be 137.65 kN. It can be observed the experimental ultimate load of model M5 is 53% more than the CHBDC design factored ultimate horizontal load. Table 5.1 summarizes the comparison discussed above.

5.10 Design consideration for the developed precast barrier wall system

The tested barrier models represent a full-scale segment of barrier wall in a typical bridge. The results from experimental testing of these bridge barrier models can be applied with confidence in bridge design, given the following considerations:

- 1- In spite of the fact that the main failure of models M1, M3 and M4 were due to threaded rod anchorage problem in the deck slab, the anchorage system is sufficient since the

experimental ultimate load carrying capacities of the developed barrier wall system comply with CHBDC design provisions for strength.

2- To increase the load carrying capacity of the developed barrier wall system, the detail of the anchorage of the threaded rods in the deck slab needs careful attention not to initiate premature crack formation as occurred in the tested barrier models.

3- The length of the precast barrier wall in practice is important in resisting vehicle crash. As expected, the longer the longitudinal length of the barrier wall, the closer is the structural behaviour to that of the longitudinally continuous cast-in-place barrier. In continuous barrier wall, the barrier resistance can be fully exerted by the AASHTO-LRFD yield line failure pattern, incorporating biaxial bending, i.e. flexural strength of barrier wall about its longitudinal and vertical axes. In the tested barrier models, it was assumed that precast barriers would be installed in segments of 3 m length with no lateral connection between them. As, such the desirable AASHTO-LRFD yield line pattern was replaced by (i) the one-way-slab (i.e. cantilever) flexural failure mode incorporating the contribution of vertical reinforcement only, or (ii) anchorage failure of the pretension rods in the concrete deck slab under the barrier wall. However, the ultimate load carrying capacity of the developed precast barrier segment is proved adequate per CHBDC strength provisions. To enhance the load carrying capacity of the developed precast barrier wall and to utilize the contribution of horizontal reinforcement available in the barrier wall, male-female shear key can be established at the vertical joint between the precast barrier walls to enhance the longitudinal continuity of a series of precast barriers segments. In this case, the

barrier resistance about its vertical axis would shift the failure mode to be by biaxial bending about the vertical and horizontal axis of the barrier wall.

- 4- Assuming that the anchorage detail of the threaded rod in the deck slab will be enhanced, it is expected that the cantilevered failure will occur through a horizontal yield line between the developed precast barrier wall and the deck slab. To attain the diagonal yield line failure specified by AASHTO-LRFD, the length of the barrier should be more than twice the barrier height for a vehicle crash at the end of the barrier. This conclusion was reached by letting $M_c = M_w/H$ in the AASHTO-LRFD equations for yield-line analysis of the barrier loaded at its end. In a similar way, the AASHTO-LRFD diagonal yield-line failure pattern occurring within the barrier length can be achieved if the length of the barrier wall is more than 5.65 the height of the barrier wall. In case of the developed barrier wall system, three precast barrier walls need to be connected at their vertical joints to achieve this diagonal yield line pattern that would enhance the load carrying capacity of the barrier system.
- 5- One may observe that barrier wall designed based on AASHTO-LRFD yield-line pattern is considered over-designed with respect to wall reinforcement. AASHTO-LRFD yield line failure equations provide high load carrying capacity due the assumption used to develop these equations. In the derivation of these equations, a general assumption was made that flexural resistance of the yield lines should not significantly vary along the height of the barrier wall. However, the tapered shape of the developed precast barrier violates this assumption. As such, in this current study, the barrier wall was divided into slices and the resisting moments were calculated for each slice about the horizontal and vertical axes and the averaged. The second

assumption in the yield-line equation was the separation between the flexural resistances along a diagonal yield-line into the resistances about the two orthogonal axes. This assumption was mainly used to simplify deriving the equations. Moreover, even if flexural cracks forming the yield-line pattern were developed, concrete crushing along the yield-line may not be full achieved experimentally. As such, it can be concluded that AASHTO-LRFD yield-line equations provide higher ultimate load carrying capacity since they were derived on the basis of concrete crushing in compression, concrete cracking in tension and yielding of the reinforcing steel crossing the yield-line vertically and horizontally.

Chapter-6

SUMMARY AND CONCLUSION

6.1 Summary

In this study, experimental and theoretical studies were conducted to investigate the structural behaviour of a developed precast barrier wall connected with the deck slab using pre-tensioned threaded rods. A literature review was conducted in order to establish the foundation for the study. It was observed that there is lack of information regarding precast barrier wall-to-deck slab connection detailing. In this study, five actual-scale barrier wall models of the developed precast barrier system were constructed and tested up-to-collapse under different load conditions to examine its structural performance and static load carrying capacity. Conclusions and recommendations for further research are presented in the following sections.

6.2 Conclusions

Based on the data generated from the experimental testing, the following conclusions are drawn.

1. The failure mode of barrier wall Models M1, M3 and M4 of the developed barrier wall system failed due to major anchorage failure in the deck slab under the barrier wall associated with threaded rod anchorage into concrete slab, with a secondary combined flexure-shear-bond crack patterns appeared in the deck slab in some of these models.
2. Model M2 that represents the cast-in-place barrier wall failed due to a combined flexure-shear-bond crack pattern in the deck slab, associated with other local

crack in the slab under the barrier wall running parallel to and at the location of the horizontal development length of the barrier wall tension reinforcement embedded into concrete slab.

3. Model M5 of the developed barrier wall system that was loaded at the end of the barrier failed primarily due to local punching shear around the loaded area, associated with diagonal torsional-flexural crack appeared in the barrier wall.
4. Model M1 of the developed barrier wall system proved “as good as” model M2 of the cast-in-place barrier system with respect to strength.
5. Models M3, M4 and M5 of the developed barrier wall system proved adequate with strength based on CHDBC design loads.
6. Comparing results of models M3 and M4 of the developed barrier wall system, it can be concluded that the presence of a cantilever portion of the deck slab to support the precast barrier wall increased the load carrying capacity of the system for the given cantilever length.

6.3 Recommendations For Future Research

It is recommended that further research efforts to be directed towards the following:

1. The experimental study of barrier wall by applying dynamic load in form of pendulum impact tests.
2. To increase the load carrying capacity of the developed barrier wall system, the detail of the anchorage of the threaded rods in the deck slab needs careful

attention not to initiate premature crack formation as happened in the tested barrier models.

3. Theoretical study of these connections using Finite Element Modeling can be done.

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Performance level	Transverse load, kN	Longitudinal load, kN	Vertical load, kN
PL-1	50	20	10
PL-2	100	30	30
PL-3	210	70	90

Table 4.1 Properties of concrete mix

Model	Average Compressive Strength (MPa)	Slump Value (mm)
M1	35	75
M2	32	75
M3	32	75
M4	32	75
M5	28	75

Table 4.2 Properties of cement grout

Model	Average Compressive Strength (MPa)
M1	50
M2	55
M3	58
M4	65
M5	62

Table 4.3 Mechanical properties of the threaded rods

Properties	All-Threaded-Bar	Nut	Washer
Bar Diameter	26 mm	26 mm	26 mm
Yield Stress	827 MPa	827 MPa	-
Ultimate Stress	1034 MPa	1034 MPa	-

Table 4.4 Laboratory torque test results

Test : 1			Test : 2			Test : 3			Test : 4		
FT-LB	KN	avg./ 25 FT-L	FT-LB	KN	avg./ 25 FT-L	FT-LB	KN	avg./ 25 FT-L	FT-LB	KN	avg./ 25 FT-L
100	14.98	3.75	100	11.50	2.88	100	13.50	3.38	100	10.84	2.71
125	19.70	4.72	125	17.50	6.00	125	15.50	2.00	125	14.61	3.77
150	25.00	5.30	150	20.50	3.00	150	19.42	3.92	150	18.44	3.83
175	30.50	5.50	175	25.71	5.21	175	24.00	4.58	175	22.61	4.17
200	35.00	4.50	200	30.00	4.29	200	28.40	4.40	200	26.19	3.58
225	40.50	5.50	225	34.29	4.29	225	33.40	5.00	225	31.25	5.06
250	46.00	5.50	250	39.16	4.87	250	38.68	5.28	250	34.36	3.11
4.60			3.92			3.87			3.44		
			3.74								
			For 79 KN Required Torque in FT-LB							528.07	

Table 5.1 Comparison between the factored ultimate horizontal load and experimental findings

Model	Load type	Exp. ultimate load (kN)	Max. Deflection at peak load (mm)	CHBDC factored ultimate horizontal load (kN)	Exp./theoretical ultimate load ratio
M1	Line load	174.61	42.83	102.42	1.70
M2	Line load	182.00	14.03	102.42	1.78
M3	Patch load	313.00	41.67	173.50	1.80
M4	Patch load	279.00	44.28	173.50	1.61
M5	Patch load	210.00	21.53	137.65	1.53

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(a)

(b)

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(a)

(b)

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Fig. 5.60 Comparison of applied torque-strain relationship in the pre-tensioned rod for model M5

Model M1

Length: 1.2 m

Load: Line

Connection: Pretension

Deck Slab: Flat

Model M2

Length: 1.2 m

Load: Line

Model M3 (Centre Loading)

Length: 3 m

Load: Patch

Connection: Pretension

Deck Slab: Cantilever

Model M4 (Centre Loading)

Length: 3 m

Load: Patch

Connection: Pretension

Deck Slab: Flat

Model M5 (End Loading)

Length: 3 m

Load: Patch

Connection: Pretension

Deck Slab: Flat

Fig. 5.61 Views of all experimental models (M1 to M5)

Sequence:

1. Design of “NJ” Type Pre-Cast Traffic Barrier Wall
2. Design Of Bridge Deck Slab, Both Flat And Cantilever
3. Design for Numbers Of Post-tensioned Bolt Required Per Barrier Wall
4. Design For Punching Shear Capacity
5. Design For Tensile Capacity Of Post-tensioned Threaded Rod
6. Design For Post-tension Force And Alignment Angle
7. Load And Resistance Factor Design

1. Design of “NJ” Type Pre-Cast Traffic Barrier Wall

- Data Considered:

1. Performance Level : (P.L) – 3
2. Height of barrier wall : 1140 mm (Ref. CHBDC Table 12.5.2.2)
3. Transverse Load Pt : 210 KN (Ref. CHBDC Table 3.8.8.1)
4. Load Factor : 1.7 (Ref. Commentary CAN/CSA-S6-06, C3.8.8.1)
5. Load applied over the length : $L = 2.4$ meter (Ref. CHBDC Table 12.5.2.4)
6. Load acting height : 990 mm (Ref. CHBDC Table 12.5.2.4)

- Assumption made:

- f_c = 35 MPa
 f_y = 400 MPa
 ϕ_c = 0.75 (Ref. CHBDC CL. 8.4.6)
 ϕ_s = 0.9 (Ref. CHBDC CL. 8.4.6)

2. Shape Of The Barrier: (Ref. MTO Structural Manual-Dec, 2003 fig. 10.1.3.3)

Top Thickness = 225 mm

3. Determine the factored bending moment (M_f): (Ref. CSA Table 9-2)

Consider barrier wall as a cantilever

$$M_f = 353.43 / 2.4 = 147.26 \text{ kN-m/m (Load Is Apply over the Length of 2.4 meter)}$$

$$V_f = 357 / 2.4 = 148.75 \text{ kN/m}$$

4. Determine the area of main flexural reinforcement:

- Cover to the reinforcement: (CHBDC Table 8.11.2.2)

Consider 50 mm cover for Pre-cast concrete exposed to de-icing chemicals, spray or surface runoff containing de-icing chemicals, mineral spray with or without freezing and thawing conditions. (Ref. CSA Table 1 C-1 condition)

- No's of reinforcement layers (curtains): (CSA 14.1.8.3)
- If the thickness is either equal greater than or equal to 210 mm, than provide two curtains. 220mm is grater than this value and for safer design provide 2 layers of reinforcements.
- Find the average A_{st} & spacing from two critical sections:

A) For section at 800mm from top:

- Effective depth (d_e):

Consider M15 bar designation as per CSA = 16mm diameter reinforcement

$$\begin{aligned}d_e &= t - \text{cover} - d_b/2 \\&= (225+80) - 50 - 16/2 \\&= 247 \text{ mm}\end{aligned}$$

- Factored Moment (M_f):

$$\begin{aligned}M_f &= 147.26 * 0.65 / 0.99 \\&= 96.68 \text{ KN-m/m}\end{aligned}$$

- Determine the value of resistance factor (K_r):

$$\begin{aligned}K_r &= (M_f) (10^6) / bd^2 \\&= (96.68)(10^6) / 1000 * (247)^2 \\&= 1.58 \text{ MPa}\end{aligned}$$

- Determine the % of reinforcement (ρ) : Ref.: CSA Table 2.1)

$$\rho = 0.48 \%$$

- Determine area of tension steel (A_{st}) :

$$\begin{aligned}A_{st} &= \rho bd \\&= (0.0048) (1000) (247) \\&= 1185.6 \text{ say } 1190 \text{ mm}^2/\text{m}\end{aligned}$$

- Determine required spacing (S):

$$\begin{aligned}S &= A_b * 1000 / A_{st} \\&= 168 \text{ mm c/c}\end{aligned}$$

B) For section at 990mm from top:

- Effective depth (d_e):

Consider M15 bar designation as per CSA = 16mm diameter reinforcement

$$\begin{aligned}d_e &= t - \text{cover} - d_b/2 \\&= (475) - 50 - 16/2 \\&= 417 \text{ mm}\end{aligned}$$

- Factored Moment (M_f):

$$M_f = 147.26 \text{ KN-m/m}$$

- Determine the value of resistance factor (K_r):

$$\begin{aligned}K_r &= (M_f) (10^6) / b d^2 \\&= (147.26)(10^6) / 1000 * (417)^2 \\&= 0.85 \text{ MPa}\end{aligned}$$

- Determine the % of reinforcement (ρ): Ref.: CSA Table 2.1)

$$\rho = 0.255 \%$$

- Determine area of tension steel (A_{st}):

$$\begin{aligned}A_{st} &= \rho b d \\&= (0.00255) (1000) (417) \\&= 1063.35 \text{ say } 1070 \text{ mm}^2/\text{m}\end{aligned}$$

- Determine required spacing (S):

$$\begin{aligned}S &= A_b * 1000 / A_{st} \\&= 187 \text{ mm c/c}\end{aligned}$$

❖ Consider minimum spacing i.e. 168 mm say at 165 mm c/c

▪ **Provided (A_{st}):**

$$A_{st} = A_b * 1000 / S$$

$$= 1212 \text{ mm}^2/\text{m}$$

▪ **Confirm minimum steel required for slab is satisfied ($A_{st(\min)}$):**

$$A_{st(\min)} = (0.2 \sqrt{f_c} / f_y) b * h$$

$$= (0.2 \sqrt{35} / 400) 1000 * 305$$

$$= 902 \text{ mm}^2/\text{m} < 1212 \text{ mm}^2/\text{m}$$

Okay.....

At End put one extra bar at s/2 distance

▪ **Check the required spacing ($S_{\min}$):** (Ref. CHBDC Cl 8.18.4.3.1)

$$S_{\min} = \text{lesser of } 3t \text{ or } 300 \text{ mm}$$

$$= 3t = (3) (305)$$

$$= 915 \text{ mm or}$$

$$\text{Consider } = 300 > 165 \text{ mm}$$

Okay.....

▪ **Check the size of bar:** (Ref. CSA Cl. 14.1.8.2)

$$\text{Should not exceed } t/10 = 35.5 \text{ mm} > 16 \text{ mm}$$

Okay.....

▪ **Check the reinforcement ratio (ρ):**

$$\rho = A_s / bd$$

$$= (1212) / (1000) (247)$$

$$= 0.0049 < 0.03 = \rho_{\text{bal}} \text{ (Ref. CSA Table 2-1 for } f_c = 35)$$

Okay.....

- **Check the for Flexure moment of Resistance (Mr) at (A):**

Depth Of Compression Block On Each Side =

$$a = \phi_s A_s f_y / \phi_c f_c \alpha_l b$$

$$= (0.9)(1212)(400)/(0.75)(0.8)(35)(1000)$$

$$= 20.78 \text{ mm}$$

$$M_r = \phi_s A_s f_y (d - a/2)$$

$$= (0.9) (1212) (400) (247 - (20.78/2))$$

$$= 103.23 \text{ KN-m/m} > 96.68 \text{ KN-m/m}$$

Okay.....

- **Check the for Flexure moment of Resistance (Mr) at (B):**

Depth Of Compression Block On Each Side =

$$a = \phi_s A_s f_y / \phi_c f_c \alpha_l b$$

$$= (0.9)(1212)(400)/(0.75)(0.8)(35)(1000)$$

$$= 20.78 \text{ mm}$$

$$M_r = \phi_s A_s f_y (d - a/2)$$

$$= (0.9) (1212) (400) (417 - (20.78/2))$$

$$= 177.41 \text{ KN-m/m} > 147.26 \text{ KN-m/m}$$

Okay.....

Summary of longitudinal main reinforcement:

- Diameter : M15 (16mm diameter)
- Spacing : 165 mm
- Layers : 2
- Cover : 50 mm
- No's of bar in each layers : 7 bars

5. Determine the area of shrinkage & temperature reinforcement:

- Determine the minimum area of steel ($A_{st(min)}$): (Ref. CHBDC Cl 8.18.4.3.1)

$$A_{st(min)} = 0.003 A_g$$

$$= (0.003) (352250)$$

$$= 1056 \text{ mm}^2 \text{ (Divide Equally on Each Face)}$$

- Determine required spacing (S):

$$A_b = 200 \text{ mm}^2 \text{ for 16 mm bar}$$

$$S = (A_b) (1000) / A_s$$

$$= (200) (1000) / (1056/2)$$

$$= \text{Provide at } 240 \text{ mm} < 3t \text{ \& } 300 \text{ mm} \quad (\text{Ref. CHBDC Cl 8.18.4.3.1}) \text{ Okay...}$$

- Check the shear resistance:

1. Effective shear depth:

$$d_v \text{ is a larger of } 0.9 d = (0.9)(297) = 267.3 \text{ mm Or}$$

$$0.72 h = (0.72) (355) = 255.6 \text{ mm}$$

$$\text{Consider grater } d_v = 267.3 \text{ mm}$$

2. Shear Resistance (V_r) at (A):

$$V_c = \beta \lambda \phi_c \sqrt{f'_c} b d_v$$

$$= (0.18) (1.0) (0.7) (\sqrt{35}) (1000) (247)$$

$$= 184.12 \text{ KN} > V_f = 148.75 \text{ KN}$$

Okay.....

3. Shear Resistance (V_r) at (B) Support:

$$V_c = \beta \lambda \phi_c \sqrt{f'_c} b d_v$$

$$= (0.18) (1.0) (0.7) (\sqrt{35}) (1000) (417)$$

$$= 444 \text{ KN} > V_f = 148.75 \text{ KN}$$

Okay.....

Since $V_c > V_f$ no shear reinforce required, flexure reinforcement are enough to control crack.

Failure Mode is in such a way that both tension and compression sides are acts as a tension side. As shown in figure the failure cracks are 2 in front and one at bottom; that mean consideration must make to design both side as a tension side and calculate moment of resistance for both sides accordingly

2. Design Of Bridge Deck Slab, Both Flat And Cantilever

- Data Considered:

1. Moment Transferred : 147.26 KN-m/m
2. Shear Transferred : 577.83 KN

- Assumption made:

$$\begin{aligned}
 f_c &= 35 \text{ MPa} \\
 f_y &= 400 \text{ MPa} \\
 \phi_c &= 0.75 \text{ (Ref. CHBDC CL. 8.4.6)} \\
 \phi_s &= 0.9 \text{ (Ref. CHBDC CL. 8.4.6)}
 \end{aligned}$$

$$a = \rho_s A_s f_y / \rho_c f_c \alpha_1 b$$

$$a = 32.30 \text{ mm}$$

$$T_r = 577.831 \text{ KN/Meter}$$

$$T_r = 348.091 \text{ KN/Bolt}$$

Where:

α_1	0.8
b	1000
ϕ_c	0.75
f_c	35
f_y	827
ϕ_s	0.9
A_s	911.34
d	271
M_f	1.47E+08

Determine the area of main flexural reinforcement:

▪ Cover to the reinforcement: (CHBDC Table 8.11.2.2)

Consider 40 mm cover for Pre-cast concrete exposed to de-icing chemicals, spray or surface runoff containing de-icing chemicals, mineral spray with or without freezing and thawing conditions. (Ref. CSA Table 1 C-1 condition)

▪ No's of reinforcement layers (curtains): (CSA 14.1.8.3)

If the thickness is either greater than or equal to of 210 mm, than provide 2 curtains. 225mm is grater than this value and for safer design provide 2 layers of reinforcements.

- Find the average A_{st} & spacing from two critical sections:

A) For section at 800mm from top:

- Effective depth (d_e):

Consider M15 bar designation as per CSA = 16mm diameter reinforcement

$$\begin{aligned} d_e &= t - \text{cover} - d_b/2 \\ &= (225) - 40 - 19.5/2 \\ &= 175 \text{ mm} \end{aligned}$$

- Determine the value of resistance factor (K_r):

$$\begin{aligned} K_r &= (M_f) (10^6) / b d^2 \\ &= (147.26)(10^6) / 1000 * (175)^2 \\ &= 4.8 \text{ MPa} \end{aligned}$$

- Determine the % of reinforcement (ρ) : Ref.: CSA Table 2.1)

$$\rho = 1.67 \%$$

- Determine area of tension steel (A_{st}) :

$$\begin{aligned} A_{st} &= \rho b d \\ &= (0.0167) (1000) (175) \\ &= 2922 \text{ mm}^2/\text{m} \end{aligned}$$

- Depth Of Compression Block On Each Side:

$$\begin{aligned} a &= \phi_s A_s f_y / \phi_c f_c \alpha_1 b \\ &= (0.9)(2922)(400) / (0.75)(0.8)(35)(720) \\ &= 69.57 \text{ mm} \end{aligned}$$

- Steel Occupied By Area 1 :

$$C = \phi_c f_c a_l b h_f$$

$$= (0.75)(0.8)(35)(720)(33)$$

$$= 499 \text{ KN}$$

$$C = T = \phi_s A_s f_y$$

$$A_{st} = (499)(10^3)/(0.9) * (400)$$

$$= 1386 \text{ mm}^2/\text{m}$$

$$M_r = \phi_s A_s f_y (d - h_f/2)$$

$$= (0.9) (1386) (400) (175 - (33/2))$$

$$= 79.09 \text{ KN-m/m}$$

- Steel Occupied By Area 2 :

$$A_{st} = (2922) - (1386)$$

$$= 1536 \text{ mm}^2/\text{m}$$

$$a = \phi_s A_s f_y / \phi_c f_c a_l b$$

$$= (0.9)(1536)(400)/(0.75)(0.8)(35)(1000)$$

$$= 26.33 \text{ mm}$$

$$M_r = \phi_s A_s f_y (d - a/2)$$

$$= (0.9) (1536) (400) (175 - (26.33/2))$$

$$= 89.48 \text{ KN-m/m}$$

- Total Resistance Value (M_r) :

$$= 79.09 + 89.48 \text{ KN-m/m}$$

$$= 168.57 \text{ KN-m/m} > 147.26 \text{ KN-m/m}$$

Okay.....

- Determine required spacing (S):

$$S = A_b * 1000 / A_{st}$$

$$= 300 * 1000 / 2922 = 102.66 \text{ Say } 100 \text{ mm c/c}$$

- Confirm minimum steel required for slab is satisfied ($A_{st(min)}$):

$$A_{st(min)} = (0.2 \sqrt{f_c / f_y}) b * h$$

$$= (0.2 \sqrt{35 / 400}) 1000 * 225$$

$$= 665 \text{ mm}^2/\text{mt} < 2922 \text{ mm}^2/\text{m}$$

Okay.....

At End put one extra bar at s/2 distance

- Check the required spacing (S_{min}): (Ref. CHBDC Cl 8.18.4.3.1)

$$S_{min} = \text{lesser of } 3t \text{ or } 300 \text{ mm}$$

$$= 3t$$

$$= (3) (225)$$

$$= 675 \text{ mm or}$$

$$\text{Consider } = 300 > 100 \text{ mm}$$

Okay.....

- Check the size of bar: (Ref. CSA Cl. 14.1.8.2)

$$\text{Should not exceed } t/10 = 35.5 \text{ mm} > 16 \text{ mm}$$

Okay.....

- Check the reinforcement ratio (ρ):

$$\rho = A_s / bd$$

$$= (2922) / (1000) (175)$$

$$= 0.016 < 0.03 = \rho_{bal} \text{ (Ref. CSA Table 2-1 for } f_c = 35)$$

Okay.....

Determine the area of shrinkage & temperature reinforcement:

- Determine the minimum area of steel ($A_{st(min)}$): (Ref. CHBDC Cl 8.18.4.3.1)

$$\begin{aligned}A_{st(min)} &= 0.003 A_g \\&= (0.003) (225 * 1000) \\&= 675 \text{ mm}^2\end{aligned}$$

- Determine required spacing (S):

$$A_b = 200 \text{ mm}^2 \text{ for 16 mm bar}$$

$$\begin{aligned}S &= (A_b) (1000) / A_s \\&= (200) (1000) / (675/2) \\&= \text{Provide at 296 Say } 250 \text{ mm} < 3t \text{ \& } 300 \text{ mm (Ref. CHBDC Cl 8.18.4.3.1)}\end{aligned}$$

Okay...

5) Summary of longitudinal main reinforcement:

- Diameter : M20 (19.5mm diameter)
- Spacing : 100 mm
- Layers : 2
- Cover : 40 mm
- Bottom and Transverse Rebar : M15 @ 290 mm c/c

Check the shear resistance:

1. Effective shear depth:

$$d_v \text{ is a larger of } 0.9 d = (0.9)(175) = 157.5 \text{ mm Or}$$

$$0.72 h = (0.72) (225) = 162 \text{ mm}$$

Consider greater $d_v = 162 \text{ mm}$

2. Shear Resistance ($V_r = V_c + V_s$) :

$$V_c = \beta \lambda \phi_c \sqrt{f_c'} b d_v$$

$$= (0.18) (1.0) (0.7) (\sqrt{35}) (1000) (162)$$

$$= 120.75 \text{ KN} < V_f = 577.83 \text{ KN}$$

$$V_s = \phi_s A_v f_y d_v \cot \theta / s \text{ (Tension Reinforcement)}$$

$$= (0.9) (600) (400) (162) (1.43) / (100)$$

$$= 500 \text{ KN}$$

$$V_s = \phi_s A_v f_y d_v \cot \theta / s \text{ (Compressive Reinforcement)}$$

$$= (0.9) (200) (400) (162) (1.43) / (290)$$

$$= 57.51 \text{ KN}$$

$$\text{Total } V_r = 678.26 \text{ KN} > V_f = 577.83 \text{ KN}$$

Okay.....

3. Design for Numbers Of Post-tensioned Bolt Required Per Barrier Wall

Design Of Bolt Connection							
Find No's Of bolt requires Per Meter Length Of Barier Wall :							
Summary Sheet							
Strength Require Per Meter Length :							
Case	dv	c	Ast	Spacin	Number	Mr	Mf
I	425	100	1098	500	6	229.99	147.26
II	375	150	1281.366	428	7	234.17	147.26
III	325	200	1465.83	375	8	228.71	147.26
IV	225	300	2013.183	273	11	204.12	147.26
V	235	290	911.34	602	5	148.45	147.26

Assumption Made :

Performance Level : PL 3

Diameter Of Bolt : 31.75 mm

Matrrial Of Bolt : S.S

fy : 600 MPa

fc : 35 MPa

Effective Temsile Stress Area Of Bolt = 79% of Gross Area (Ref. CAN/CSA- A_b = 549 S16.1-94 - Dimension OF Bolt Page 6-144 Ref. Tensile stress Area)(As per suppliers test certificate)

Transverse Load (Pt) : 210 KN (Ref. CHBDC 3.8.8.1)

Impact Load Distance : 900 mm from the bottom of barrier wall (Ref. CHBDC 12.5.2.4)

Applied Over A Barrier Length Of : 2.4 Meter (Ref.: CHBDC 12.5.2.4(a))

Load Factor 1.7

W 210 KN 357

Mf W X L 0.99 353.43 KN-Mt

Applied Over 2.4 Meter Length

Mf 189 X 1/2.4 147.26 KN-Mt

CASE - V

ϕ_c	0.75 (Ref. CHBDC CL. 8.4.6)
ϕ_s	0.9 (Ref. CHBDC CL. 8.4.6)
α_1	0.8
β_1	0.88
f_c	35
f_y	827
d_v	235

$$K_r = \frac{(M_f)(10^6)/bd^2}{2.67} \quad \text{MPa}$$

$$\rho = \frac{\phi_s f_y \pm \sqrt{(\phi_s f_y)^2 - (4) ((\phi_s f_y)^2 / 2\alpha_1 \phi_c f_c)(K_r)}}{(2)(\phi_s f_y)^2 / 2\alpha_1 \phi_c f_c}$$

$$\rho = 0.00385$$

Area Of Steel (A_{st}) :

$$A_{st} = \rho b d = 904.75 \quad \text{mm}^2$$

$$\text{Actual } A_{st} = 911.34 \quad \text{mm}^2 \quad \# \text{ Of Bolt Required / Meter}$$

Spacing (S) : $(A_b)(1000)/A_{st}$

$$S = 602 \quad \text{mm}$$

Find Out The Depth Of Equivalent Rectangular Stress Block :

$$a = \frac{\phi_s f_y A_{st}}{\phi_c \alpha_1 f_c b} = 32.3 \quad \text{mm}$$

Cohesion Stress (C):

$$c = a/\beta_1 = 36.70 \quad \text{mm}$$

Moment Of Resistance (M_r):

$$M_r = \phi_s f_y A_{st} (d - a/2) = 148.45 \quad \text{KN-Mt} > 147.26 \text{ KN-Mt} \quad \text{Okay}$$

Flexure Check :

$$c/d = 0.156$$

$$c/d \leq \frac{700}{700 + f_y} = 0.458 \quad \text{Under Reinforced Section} \quad \text{Okay}$$

4. Design For Punching Shear Capacity

Capacity Of Bolt & Concrete in Tension & Shear

Tension Force Per meter Length $T_r = M_f / (d-a/2)$

$$a = \phi_s A_s f_y / \phi_c f_c \alpha_1 b$$

$$a = 32.30 \text{ mm}$$

$$T_r = 577.831 \text{ KN/Met}$$

$$T_r = 348.091 \text{ KN/Bolt}$$

Where :

α_1	0.8
b	1000
ϕ_c	0.75
f_c	35
f_y	827
ϕ_s	0.9
A_s	911.34
d	271
M_f	1.47E+08

Check For Three Possible Failure Mode :

- Tensile Capacity Of Steel Bolt
- concrete Breakout Of The Embedment
- Concrete Pryout capacity

Member with least value will fail first

a) Tensile Capacity Of Steel Bolt

$$N_{sr} = \phi_s n A_{se} f_{ut} R$$

$$N_{sr} = 393.83 \text{ KN}$$

Where :

ϕ_s	0.9
f_{ut}	1034 MPa *
A_{se}	529
R	0.8 **
n	1 Nos of bolt

* (Ref.: CSA23.3-04 D.6.1.2) F_u is lesser of 1.9 F_y or 860 MPa)

** (Ref.: CSA23.3-04 D.5.4) R for ductile tesion member = 0.8

b) concrete Breakout Of The Embedment

Considered Only Concrete

$$N_{br} = K \phi_c \sqrt{f_c} h_{ef}^{1.5} R$$

$$N_{br} = 194.48 \text{ KN}$$

Where :

K	10 *
f_c	35 MPa
ϕ_c	0.75
h_{ef}	244 mm
R	1.15 **

* (Ref.: CSA23.3-04 D.6.2.2)) Headed Bolt

** (Ref.: CSA23.3-04 D.5.4 (c))

$$N_{cbr} = \frac{A_N}{A_{NO}} (\psi_{ed,N} \times \psi_{c,N} \times \psi_{cp,N}) \times N$$

$$A_{NO} = 9 h_{ef}^2 = 535824 \text{ mm}^2$$

$$A_N = (278 + 344) \times (344 + 344) = 427936 \text{ mm}^2$$

$$\psi_{ed,N} = 0.7 + 0.3 (C_{min} / 1.5 h_{ef})$$

Ref. : CSA D.6.2.5

$$\psi_{ed,N} = 0.94$$

$$\psi_{c,N} = 1.25 \text{ Assume Uncracked Con.}$$

$$\psi_{cp,N} = 1$$

$$N_{cbr} = 182.98 \text{ KN}$$

c) Concrete Pryout capacity

$$N_{pr} = A_{bh} 8 \phi_c R f_c$$

Ref.: CSA D.6.3.4

$$A_{bh} = (150 \times 150) - \pi/4 (25.4)^2$$

Note : 150 X 150 mm bottom plate of 19 mm thick

$$21993.55 \text{ mm}^2$$

$$R = 1.15 \text{ Ref.: CSA D.5.4(c)}$$

$$N_{pr} = 5311.44 \text{ KN}$$

Member with least value will fail first $N_{cbr} = 182.98 \text{ KN} < T_r = 348.1 \text{ KN/Bolt}$

Sequence Failure :

1st : Concrete Breakout

2nd : Crack Occure at bottom support of barrier wall

3rd : Bolts start yielding

4th : Crushing Of Concrete at rear suporting face will start

5th : Either complete failure of bolt or Deck slab will break completely

4) AASHTO And ACI Punching Shear Equation :

$$V_c = (0.17 + 0.33/\beta_c) \sqrt{f_c} b_o d \quad \text{N}$$

Where :

β_c	Ratio Of Short Side To Long Side Of Concentrated Load 145/145	1
f_c		35 MPa
b_o	Perimeter Of Critical Section = 150 mm Plate +200/2 Slab thickness ,	1400 mm
d	Average Effective Depth OF Section	200 mm

$$V_c = 839.847 \text{ KN} > 348.09 \text{ KN}$$

Okay

5) CHBDC Shear Resistance Equation :

$$V_c = \phi_c f_{cr} b_o d \quad \text{N}$$

Where :

f_{cr}	Cracking Strength Of Concrete $0.4\sqrt{f_c}$ Normal desity Concrete	2.37 MPa
f_c		35 MPa
b_o	Perimeter Of Critical Section = 150 mm Plate +200/2 Slab thickness ,	1400 mm
d	Average Effective Depth OF Section	200 mm

$$V_c = 496.951 \text{ KN} > 348.09 \text{ KN}$$

Okay

6) General Punching Shear Equation : (Ref.: Tusi,White & Kim)

$$f_t = (0.17 + 0.33/\beta_c) \sqrt{f_c} \quad \text{MPa}$$

$$V_c = (b_1 + b_2 + 2d/\tan\theta) d/\tan\theta f_t \quad \text{N}$$

Where :

β_c	Ratio Of Short Side To Long Side Of Concentrated Load 140/140	1
f_c		35 MPa
b_1	Short Side OF Concentrated Load	150 mm
b_2	Long Side OF Concentrated Load	150 mm
d	Average Effective Depth OF Section	200 mm
θ	Acute Angle between horizontal & the assumed failure plane	37°
	$\tan\theta$	0.75

$$f_t = 2.96 \text{ MPa}$$

$$V_c = 657.342 \text{ KN} > 348.09 \text{ KN}$$

Okay

5. Design For Tensile Capacity Of Post-tensioned Threaded Rod

Tensile Capacity Of Bolt And Corresponding Moment OF Resistance

Case 1

Bolt Resistance In Tension (Ref.: CAN/CSA-S16.1-94 - Page 1- 88) At Ultimate Capacity

$$T_r = \phi_b A_n F_u$$

Where :

ϕ_b	0.8	#
A_n	549	mm ² *
F_u	1034	MPa **

* (Ref. : CAN/CSA-S16.1-94 Page 6-144) A_n = Tensile Stress Area Of Bolt

** (Ref.: CSA23.3-04 D.6.1.2) F_u is lesser of 1.9 F_y or 860 MPa) **Suppliers Test Report**

(Ref.: CHBDC 10.5.7)

$$T_r = 454.13 \text{ KN} \quad \text{For one Bolt}$$

$$T_r = 753.86 \text{ KN} \quad \text{For 1.66 Bolts / Meter Length}$$

$$a = \phi_s A_s f_y / \phi_c f_c a_1 b$$

$$a = 32.30 \text{ mm}$$

$$M_r = T (d-a/2)$$

Where :

a_1	0.8
b	1000
ϕ_c	0.75
f_c	35
f_y	827
ϕ_s	0.9
A_s	911.34
d	235

$$M_r = 164.982 \text{ KN-Mt/Mt} > M_f = 147.26 \text{ KN-Mt/Mt}$$

Okay

Bolt Resistance In Shear (Ref.: CAN/CSA-S16.1-94 - Page 1- 88)

$$V_r = 0.6 \phi_b n A_n F_u$$

Where :

ϕ_b	0.8	#
A_n	549	mm ² *
F_u	1034	MPa **
n	7	Nos ***

* (Ref. : CAN/CSA-S16.1-94 Page 6-144) A_n = Tensile Stress Area Of Bolt

** (Ref.: CSA23.3-04 D.6.1.2) F_u is lesser of 1.9 F_y or 860 MPa) **Suppliers Test Report**

*** (Ref. : CAN/CSA-S16.1-94 Page 6-144) n = # of threads/inch

(Ref.: CHBDC 10.5.7)

$$V_r = 1907.36 \text{ KN/Bolt} \quad \text{For one bolt}$$

$$V_r = 3166.214 \text{ KN/Mt} \quad \text{For 1.66 bolts per meter l****}$$

**** But not greater than lateral bearing resistance (Ref.: CAN/CSA-S16.1-94 --- 26.2.2)

$$B_r = 1.4 \phi_c n A f_c$$

(Ref.: CAN/CSA-S16.1-94 --- 25.2.3.3)

$$B_r = 1462.75 \text{ KN/Mt} > V_r$$

$$V_r = 1462.75 \text{ KN/Mt} \quad \text{Considered}$$

n = number or bolt per meter length

$$V_r = 1462.75 \text{ KN/Mt} > V_f = 148.8 \text{ KN/Mt}$$

Okay

Case 2

Bolt Resistance In Tension (Ref.: CAN/CSA-S16.1-94 - Page 1- 88) At Yield Capacity

$$T_r = \phi_s A_s F_y$$

Where :

$$\phi_s = 0.9$$

$$A_s = 549 \text{ mm}^2 \quad *$$

$$F_y = 827 \text{ MPa}$$

* (Ref. : CAN/CSA-S16.1-94 Page 6-144) A_s = Tensile Stress Area Of Bolt

$$T_r = 408.62 \text{ KN}$$

For one Bolt

$$911.34$$

$$T_r = 678.31 \text{ KN}$$

For 1.66 Bolts / Meter Length

$$a = \phi_s A_s f_y / \phi_c f_c a_1 b$$

$$a = 32.30 \text{ mm}$$

$$M_r = T (d - a/2)$$

Where :

$$a_1 = 0.8$$

$$b = 1000$$

$$\phi_c = 0.75$$

$$f_c = 35$$

$$f_y = 827$$

$$\phi_s = 0.9$$

$$A_s = 911.34$$

$$d = 235$$

$$M_r = 148.448 \text{ KN-Mt/Mt} > M_f = 147.26 \text{ KN-Mt/Mt}$$

Okay

6. Design For Post-tension Force And Alignment Angle

Design For Post-tensioning (Bonded) - Using Deformed Stainles Steel Bars

Mf 8.66E+07

At Section 2 : L = 0.65

At Service Load

Find The minimum force require (p_f) :

Eccentricity at Front : 33 mm

Eccentricity at Rear : 34 mm

Total Eccentricity (e) = 67 mm

Where :

$$P_f \geq \frac{M_{\max} - S_b f_b}{e + k_t}$$

$$P_f \geq 131.05 \text{ KN}$$

$$M_{\max} = 5.69E+07$$

$$f_b = 2.96$$

$$S_b = 1.55E+07$$

$$e = 33$$

$$A = 305000$$

$$k_t = S_b / A = 50.83$$

$$\text{Force required per bolt} = 78.94 \text{ KN} \quad \# \text{ of bolt per meter length} = 1.66$$

Torque Force / bolt :

Where :

$$\text{Torque} = K D P / 12 \text{ ft/lbs}$$

$$K = 0.2 \text{ constant}$$

$$D = 1 \text{ in}$$

$$P = 17746.35 \text{ lbs}$$

$$\text{Torque} = 295.77 \text{ ft/lbs}$$

% of Initial Force Applied :

$$f_{pi} = P / A_{st}$$

Where :

$$f_{pi} = 143.79 \text{ MPa}$$

$$A_{st} = 549 \text{ mm}^2$$

$$P = 78.94 \text{ in}$$

$$\% \text{ Of Force} = 17\%$$

$$F_y = 827.00 \text{ MPa}$$

Stress At The End Of Barrier Wall (M = 0) At Ultimate Load

(+) Compressive

Compressive Stress limit : $0.45 \sqrt{f_c} = +15.75 \text{ MPa}$

(-) Tensile

Tensile Stress limit : $0.5 \lambda \sqrt{f_c} = -2.96 \text{ MPa}$ $P/A \pm Pe/S$

Where :

$$+ P/A \quad 0.48$$

$$+ Pe/S \quad 0.00$$

$$e = 0$$

$$- Pe/S \quad 0.35$$

$$e = 34$$

$$- M/S \quad 2.40$$

Front : $-2.27 \leq 2.96 \text{ MPa}$

Tensile

Okay

Rear : $3.23 < 15.75 \text{ MPa}$

Compressive

Okay

At Section 3 :

$$L = 0.965$$

At Service Load**Find The minimum force require (P_f) :**

Eccentricity at Front : 65 mm

Eccentricity at Rear : 34 mm

Total Eccentricity (e) = 99 mm

Where :

$$P_f \geq \frac{M_{\max} - S_b f_b}{e + k_t}$$

$$M_{\max} = 8.44 \text{E}+07$$

$$f_b = 2.96$$

$$S_b = 3.76 \text{E}+07$$

$$P_f \geq -186.35 \text{ KN}$$

$$e = 65$$

$$\text{Provide } P_f \geq 131.05 \text{ KN}$$

$$A = 475000$$

$$k_t = S_b / L = 79.17$$

$$\text{Force required per bolt} = 78.94 \text{ KN} \quad \# \text{ of bolt per meter length} = 1.66$$

Torque Force / bolt :

Where :

$$\text{Torque} = \frac{K D P}{12} \text{ ft/lbs} \quad K = 0.2 \text{ constant}$$

$$D = 1 \text{ in}$$

$$P = 17746.35 \text{ lbs}$$

$$\text{Torque} = 295.77 \text{ ft/lbs}$$

% of Initial Force Applied :

$$f_{pi} = P / A_{st}$$

Where :

$$f_{pi} = 143.79 \text{ MPa}$$

$$A_{st} = 549 \text{ mm}^2$$

$$P = 78.94 \text{ in}$$

$$\% \text{ Of Force} = 17\% \quad F_y = 827.00 \text{ MPa}$$

Stress At The End Of Barrier Wall (M = 0) At Ultimate Load

(+) Compressive

Compressive Stress limit : $0.45 \sqrt{f_c} = +15.75 \text{ MPa}$

(-) Tensile

Tensile Stress limit : $0.5 \lambda \sqrt{f_c} = - 2.96 \text{ MPa}$

$P/A \pm Pe/S$

Where :

+ P/A 0.28

+ Pe/S 0.23

- Pe/S 0.12

- M/S 2.25

e = 65

e = 34

Front :

-1.86 $\leq$ 2.96 MPa

Tensile

Okay

Rear :

2.41 $<$ 15.75 MPa

Compressive

Okay

Front

0.28

0.23

0.12

2.25

1.86

+

+

+

=

Rear

0.28

0.23

0.12

2.25

2.41

P/A

Pe/A

Pe/A

M/S

Front

Rear

7. Load And Resistance Factor Design

Load & Resistance Factor Design

Design By Considering Following Cases :

TL 4 : Test Level 4

Test Level 4 runs with a small car and a pickup truck at 60 mph and a single unit truck impacting at 50 mph.

	LRFD	CHBDC
Design Forces and Designations	TL - 4	PL - 3
F_t transverse (kip)	54	357 KN
F_L Longitudinal (kip)	18	
F_v vertical/down (kip)	18	
L_t and L_L (ft)	3.5	1.2 mt
L_v (ft)	18	
H_e minimum height of horizontal load (in)	32	
H Minimum Height of rail (in)	32	

Find Center of gravity On X - X Axis :

Region	a	y	ay
1	30875	32.5	1003437.5
2	236250	590	139387500
3	20000	190	3800000
4	32000	581.6666667	18613333.33
5	21250	148.3333333	3152083.333
	340375	1542.50	1.66E+08

$$y = \frac{\sum ay}{\sum a} = 488 \text{ mm}$$

$$y_b = 488 \text{ mm}$$

$$y_t = 627 \text{ mm}$$

Transverse Load (Pt): 210 KN (Ref. CHBDC 3.8.8.1)

Impact Load Distance : 900mm from the bottom of barreir wall (ref. CHBDC 12.5.2.4)

Applied Over A Barrier length Of : 2.4 Meter (Ref. CHBDC 12.5.2.4 (a))

$$W = 210.00 \text{ KN} \quad 357.00 \text{ KN} \quad \text{Load Factor } 1.7$$

$$M_f = W \times L = 357.00 \times 0.99 = 353.43 \text{ KN-mt}$$

Applied Over 2.4 mt Length

$$M_f = 353.43 \times 1/2.4 = 147.26 \text{ KN-mt}$$

Development Length of M15: 390 mm

Development Length Equation Ref. : CHBDC Table 8.15.2.3

Development Length of 25.4 mm bolt: 1600 mm

Find Flexure moment of Resistance About Vertical Axis (Mw):

Mw for Interior Region :

Ref. : Figure 1

$$Mw_{(int)} = A_s f_y (d - a/2)$$

where $\phi_s = 1.0$ for extreme event limit state

$$a = (6 \text{ nos of bar}) A_b f_y / 0.85 f_c b$$

Where :

$$A_b = 200 \text{ mm}$$

$$f_y = 400 \text{ Mpa}$$

$$\phi_s = 0.9$$

$$f_c = 35 \text{ Mpa}$$

$$b = 1115 \text{ mm (height Of barrier wall)}$$

$$a = 12.06 \text{ mm}$$

$$a/2 = 6.03 \text{ mm}$$

Bar	d (mm)	Lever Arm d - a / 2	ϕM_n For inside Region	ϕM_n For Outside Region
1	75	68.97	5.52	
2	174	167.97		13.44
3	74	67.97	5.44	
4	195	188.97		15.12
5	75	68.97	5.52	
6	219	212.97		17.04
7	90	83.97	6.72	
8	309	302.97		24.24
9	112	105.97	8.48	
			31.67 KN - m	69.83 KN - m

$$\phi M_n_{(int)} = \phi M_n / b \text{ (In- side)}$$

$$= (31.67) / 1.115$$

$$28.40 \text{ KN} \cdot \text{m/m}$$

$$\phi M_n_{(out)} = \phi M_n / b \text{ (Out Side)}$$

$$= (69.83) / 1.115$$

$$62.63 \text{ KN} \cdot \text{m/m}$$

Since when accident occurs will create 2 tension line at inside face and 1 tension line at outside face of barrier wall

$$Mw_{(int)} = (2) \phi M_n_{(int)} + (1) \phi M_n_{(out)} / 3$$

$$Mw_{(int)} = 39.81 \text{ KN} \cdot \text{m/m}$$

Mw for End Region ($M_{w(end)}$):**Ref. : Figure 2**

Since when accident occurs will create 1 tension line at inside face of barrier wall

Ref. : Figure : 2

Bar	Embedded Length (mm)	Bar fraction	Developed As (mm ²)	d (mm)	Lever Arm d - a / 2	ΣM_n
1	1004.00	1.00	200.00	75.00	70.46	5.64
3	809.00	1.00	200.00	74.00	69.46	5.56
5	550.00	1.00	200.00	75.00	70.46	5.64
7	254.00	0.65	130.26	90.00	85.46	4.45
9	43.00	0.11	22.05	112.00	107.46	0.95
			752.31			22.23

Development Length : 390 mm

$$a = \frac{A_s f_y}{0.85 f_c b}$$

Where :

$$A_s = 752.31 \text{ mm}^2$$

$$f_y = 400 \text{ Mpa}$$

$$\phi_s = 0.90$$

$$f_c = 35 \text{ Mpa}$$

$$b = 1115 \text{ mm (height Of barrier wall)}$$

$$a = 9.07 \text{ mm}$$

$$a/2 = 4.54 \text{ mm}$$

$$M_{w(out)} = \phi M_n / b$$

$$M_{w(end)} = 19.94 \text{ KN - m/m}$$

Find Flexure moment of Resistance About Horizontal Axis (Mc):

Mc for Interior Region :

Ref. : Figure 1 & 3

Use average dimension Refere fig : 1

Location	d (mm)	Average (mm)
Top	184	211 Top
Middle	238	
Bottom	385	311.5 Bottom

$$a = \frac{A_s f_y}{0.85 f_c b}$$

Where :

$$A_s = 1949 \text{ mm}^2$$

$$A_s \text{ total} = 7 \times 200 = 1400 \text{ mm}^2/\text{meter length (Ref. figure \# 3)}$$

$$f_y = 519.56 \text{ MPa}$$

$$+ 1 \times 549 = 549 \text{ mm}^2/\text{meter}, \text{ Total } A_s = 1949 \text{ mm}^2/\text{meter}$$

$$\phi_s = 0.90$$

$$f_y = 400 \quad 72\%$$

$$f_c = 35 \text{ Mpa}$$

$$f_y = 827 \quad 28\%$$

$$f_y \text{ average } 519.56$$

$$b = 1000 \text{ mm (1 meter strap is considered for design)}$$

$$a = 34.04 \text{ mm}$$

$$a/2 = 17.02 \text{ mm}$$

$$M_{ci(\text{TOP})} = \phi A_s f_y (d - a/2)$$

$$(1.0) (1949) (519.56) (211 - 17.02)$$

$$M_{ci(\text{TOP})} = 196.43 \text{ KN} \cdot \text{m/m}$$

$$M_{ci(\text{bottom})} = \phi A_s f_y (d - a/2)$$

$$(1.0) (1949) (519.56) (311.5 - 17.02)$$

$$M_{ci(\text{bottom})} = 298.20 \text{ KN} \cdot \text{m}$$

$$M_{ci} = (y_t) \phi M_{ci(\text{TOP})} + (y_b) \phi M_{ci(\text{BOTTOM})} / 1.115$$

$$M_{ci} = 240.93 \text{ KN} \cdot \text{m/m}$$

Mc for End Region :**Ref. : Figure 2**

Reference figure : 2

Bar	d (mm)	fraction	developed As
1	1015.00	1.00	200.00
2	886.00	1.00	200.00
3	792.00	0.55	288.94
4	701.00	1.00	200.00
5	577.00	1.00	200.00
6	453.00	1.00	200.00
7	328.00	0.84	168.21
8	235.00	0.16	85.73
9	42.00	0.11	21.54
10	19.00	0.05	9.74
			1574.17

Development Length : 390 mm

Development Length Equation Ref. : CSA 12.2.3

$$a = \frac{A_s f_y}{0.85 f_c b}$$

Where :**As = Average Area Of Steel/meter length**

$$A_s = 1312 \text{ mm}^2 \quad 1312$$

$$f_y = 501.63 \text{ Mpa}$$

$$f_c = 35 \text{ Mpa}$$

$$b = 1000 \text{ mm (Converted Ast for 1000 mm span)}$$

$$f_y = 400 \quad 76\%$$

$$f_y = 827 \quad 24\%$$

$$f_y \text{ average} = 501.63$$

$$a = 22.12 \text{ mm}$$

$$a/2 = 11.06 \text{ mm}$$

$$M_{ce(TOP)} = \phi A_s f_y (d - a/2) \\ (1.0) (1312) (501.63) (211 - 11.06)$$

$$M_{ce(TOP)} = \mathbf{131.57 \text{ KN} \cdot \text{m}}$$

$$M_{ce(TOP)} = \phi A_s f_y (d - a/2) \\ (1.0) (1312) (501.63) (311.5 - 11.06)$$

$$M_{ce(bottom)} = \mathbf{197.70 \text{ KN} \cdot \text{m}}$$

$$M_{ce} = (y_t) \phi M_{ce(TOP)} + (y_b) \phi M_{ce(BOTTOM)} / 1.115$$

$$M_{ce} = \mathbf{160.49 \text{ KN} \cdot \text{m/m}}$$

Check The capacity For Interior Region :

$$L_{ci} = (L_t/2) + \sqrt{(L_t/2)^2 + [(8 \times H (M_{b(int)} + M_{w(int)} \times H))/M_{c(int)}]}$$

Where :

L_t = 2.4 meterM_{b(int)} = 0 Since No Beams Are providedM_{w(int)} = 39.81 KN - m/mM_{c(int)} = 240.93 KN - m/m

H = 1.14 Mt

L_{ci} =	2.98 Mt	9.76 ft
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$$R_{wi} = (2/2L_{ci} - L_t) (8 \times M_{w(int)} \times H) + (M_{c(int)} \times L_{ci}^2 / H)$$

R_{wi} =	940.74 KN	> 357 KN	
	211.40 KIP	> 54 KIP	

Okay

Okay

Check The capacity For End Region :

$$L_{ce} = (L_t/2) + \sqrt{(L_t/2)^2 + [H (M_{b(end)} + M_{w(end)} \times H))/M_{c(end)}]}$$

Where :

L_t = 1.2 meterM_{b(end)} = 0 Since No Beams Are providedM_{w(end)} = 19.94 KN - mM_{c(end)} = 160.49 KN - m

H = 1.14 Mt

L_{ce} =	1.32 Mt	4.34 ft
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$$R_{we} = (2/2L_{ce} - L_t) (M_{b(end)} + (M_{w(end)} \times H) + (M_{c(end)} \times L_{ci}^2 / H))$$

R_{we} =	372.26 KN	> 357 KN	
	83.65 KIP	> 54 KIP	

Okay

Okay

Reference Figure # 3

**STEEL/CONCRETE STRAIN GAUGE AND LVDT'S READING
FOR MODEL M1 TO M5****Sequence:**

1. Model M1
2. Model M2
3. Model M3
4. Model M4
5. Model M5
6. Applied Load v/s. Equivalent Torque For Each Models
7. Pre-tensioned Rod Test Certificate

Table B.1 Model M1: Deflection

Applied Static Load (KN)	LVDT 1	LVDT 2
0	0.00	0.00
10	0.25	0.29
20	0.62	0.68
30	1.15	1.25
40	1.83	1.94
50	2.57	2.71
60	3.67	3.86
70	5.38	5.53
80	7.51	7.52
90	9.58	9.53
100	12.19	12.03
110	15.35	15.23
120	18.54	18.32
130	20.99	20.55
140	22.93	22.49
150	25.40	24.98
160	28.98	28.53
170.17	33.98	33.05
174.61	43.39	42.26

Table B.2 Model M1: Post-tensioned Rod Strain

Applied Static Load (KN)	Strain Bolt, SB1	Strain Bolt, SB2
0	816	816
10	817	807
20	826	814
30	865	857
40	939	937
50	1032	1042
60	1160	1180
70	1302	1282
80	1409	1392
90	1585	1603
100	1754	1779
110	1813	1788
120	1915	1925
130	2028	2014
140	2147	2137
150	2278	2288
160	2423	2443
170	2576	2561
174.61	2625	2607

Table B.3 Model M1: Concrete Strain At Compression Side Of Barrier Wall

Applied Static Load (KN)	Concrete Strain , C1	Concrete Strain , C2	Concrete Strain , C3
0	0.00	0.00	0.00
10	-5.20	-3.31	-4.25
20	-9.45	-9.45	-11.32
30	-23.62	-22.20	-20.75
40	-55.74	-52.90	-38.21
50	-99.66	-99.65	-64.15
60	-147.83	-162.93	-105.18
70	-208.74	-235.64	-180.63
80	-253.12	-309.28	-271.62
90	-308.36	-384.33	-353.17
100	-356.03	-455.12	-439.42
110	-411.73	-539.11	-545.45
120	-463.92	-616.01	-628.37
130	-477.89	-697.14	-678.46
140	-500.72	-777.27	-702.43
150	-534.21	-856.96	-737.36
160	-581.84	-947.01	-759.03
170	-630.89	-1067.67	-781.64
174.61	-639.38	-1174.65	-780.23

Table B.4 Model M1: Steel Strain At Load Facing Side Of Vertical Rebar

Applied Static Load (KN)	Steel Strain , S1	Steel Strain , S2	Steel Strain , S3
0	0.00	0.00	0.00
10	4.35	8.33	7.26
20	12.10	24.10	15.00
30	15.97	39.20	21.29
40	20.81	45.08	25.65
50	25.65	51.20	30.97
60	29.52	54.78	38.72
70	34.36	62.08	46.46
80	36.56	65.85	52.75
90	37.74	68.43	55.66
100	41.13	71.20	61.47
110	45.00	73.85	63.40
120	48.39	75.29	68.24
130	52.56	77.89	69.21
140	58.12	80.31	71.63
150	63.67	82.12	73.88
160	68.45	83.98	75.78
170	72.09	85.44	77.75
174.61	73.45	86.21	78.60

Table B.5 Model M2: Deflection

Applied Static Load (KN)	LVDT 1	LVDT 2
0.00	0.00	0.00
10.00	0.08	0.06
20.00	0.50	0.53
30.00	0.93	1.00
40.00	1.54	1.67
50.00	2.31	2.54
60.00	3.07	3.34
70.00	3.97	4.29
80.00	5.00	5.26
90.00	5.83	6.10
100.00	6.73	7.02
110.00	7.33	7.65
120.00	8.23	8.60
130.00	8.89	9.36
140.00	9.64	10.20
150.00	10.46	11.17
160.00	11.21	12.07
170.00	13.33	13.00
180.00	13.42	14.55
182.00	13.43	14.63

Table B.6 MODEL M2: Concrete Strain At Compression Side Of Barrier Wall

Applied Static Load (KN)	Concrete Strain, C1	Concrete Strain, C2	Concrete Strain, C3
0.00	0.00	0.00	0.00
10.00	-4.25	-6.16	-9.45
20.00	-7.08	-13.26	-20.79
30.00	-12.28	-18.47	-30.24
40.00	-16.53	-26.52	-40.63
50.00	-22.67	-40.25	-55.27
60.00	-31.64	-57.29	-68.03
70.00	-43.45	-69.60	-83.62
80.00	-54.78	-78.12	-106.29
90.00	-69.42	-97.53	-126.60
100.00	-79.80	-115.00	-148.33
110.00	-101.52	-127.00	-162.97
120.00	-131.74	-151.23	-184.22
130.00	-159.12	-170.89	-204.06
140.00	-187.45	-197.45	-223.42
150.00	-218.13	-229.12	-241.84
160.00	-247.87	-250.47	-260.72
170.00	-276.19	-277.76	-281.03
180.00	-303.09	-302.88	-302.75
182.00	-306.87	-305.33	-304.17

Table B.7 Model M2: Steel Strain At Load Facing Side Of Vertical Rebar

Applied Static Load (KN)	Steel Strain, S1	Steel Strain, S2	Steel Strain, S3
0.00	0.00	0.00	0.00
10.00	6.98	10.12	7.26
20.00	24.12	26.51	25.66
30.00	36.57	41.46	39.70
40.00	58.78	67.97	61.48
50.00	81.70	96.90	85.21
60.00	98.50	123.42	101.67
70.00	118.60	155.73	121.04
80.00	132.40	195.75	138.47
90.00	156.70	238.67	161.72
100.00	195.12	300.41	201.91
110.00	229.34	352.02	233.88
120.00	265.60	419.08	278.92
130.00	290.67	474.09	308.95
140.00	328.96	542.61	341.41
150.00	367.30	621.27	375.32
160.00	397.30	682.57	408.75
170.00	435.78	741.95	440.24
180.00	467.50	816.31	472.22
182.00	471.30	823.06	478.04

Table B.8 Model M3: Deflection

Applied Static Load (KN)	LVDT 1	LVDT 2	LVDT 3	LVDT 4	LVDT 5
0	0.00	0.00	0.00	0.00	0.00
10	0.00	0.15	0.25	0.21	0.00
20	0.00	0.33	0.55	0.49	0.00
30	0.00	0.58	0.98	0.85	0.03
40	0.02	0.94	1.53	1.33	0.07
50	0.33	1.64	2.47	2.07	0.11
60	1.32	3.24	3.96	3.27	0.15
70	2.67	4.83	5.50	4.47	0.20
80	3.60	5.97	6.70	5.41	0.29
90	4.82	7.32	8.01	6.38	0.54
100	5.78	8.82	9.53	7.57	0.83
110	7.65	10.72	11.39	8.99	1.12
120	8.36	11.93	12.64	9.86	1.30
130	8.63	12.95	13.79	10.72	1.45
140	8.65	14.26	15.07	11.76	1.60
150	9.01	14.75	16.39	12.68	1.75
160	9.97	15.56	18.31	13.94	1.89
170	10.48	15.75	19.73	14.84	2.08
180	10.49	17.19	21.07	15.97	2.23
190	10.48	18.34	22.58	17.10	2.43
200	10.85	18.88	23.68	18.00	2.58
210	10.86	20.25	26.19	19.95	2.79
220	10.87	21.95	29.35	22.39	3.15
230	10.88	23.92	33.45	25.56	3.66
240	10.99	25.12	35.37	26.81	3.86
250	10.98	25.44	35.84	26.96	3.88
260	11.00	25.71	36.19	27.36	3.97
270	11.10	25.91	36.37	27.60	4.06
280	11.14	26.43	37.47	27.80	4.14
290	11.20	27.12	38.60	27.88	4.28
300	11.33	28.47	39.78	28.78	4.49
310	11.42	29.56	40.83	29.87	4.65
313	11.47	30.14	41.67	30.56	4.85

Table B.9 Model M3: Post-tensioned Rod Strain

Applied Static Load (KN)	Rod Strain , SB1	Rod Strain , SB2	Rod Strain , SB3	Rod Strain , SB4	Rod Strain , SB5
0	805.00	805.00	805.00	805.00	805.00
10	805.00	805.00	805.00	805.00	805.00
20	805.00	805.00	805.00	806.12	805.00
30	805.00	805.00	805.00	821.10	805.00
40	805.00	805.00	805.00	821.10	809.34
50	805.00	805.00	805.00	821.10	811.54
60	805.00	805.00	805.00	821.10	788.90
70	806.47	805.00	805.00	821.10	790.34
80	808.42	805.00	805.00	821.10	792.25
90	811.84	806.95	809.37	823.09	795.60
100	818.20	814.24	820.52	830.52	801.84
110	831.39	836.13	874.86	852.85	814.76
120	837.75	855.09	914.65	872.19	821.00
130	863.17	878.44	963.66	896.01	845.91
140	906.19	930.49	1024.81	949.10	888.07
150	960.45	1023.40	1150.04	1043.87	941.24
160	1005.44	1078.38	1233.06	1099.95	985.33
170	1077.81	1139.70	1321.92	1162.49	1056.25
180	1134.05	1182.52	1397.20	1206.17	1111.37
190	1203.51	1238.98	1486.57	1263.76	1179.44
200	1256.34	1295.45	1561.39	1321.36	1231.21
210	1330.71	1382.59	1725.64	1410.24	1304.10
220	1472.13	1536.96	1935.64	1567.70	1442.69
230	1636.12	1745.45	2206.05	1780.36	1603.40
240	1706.13	1839.01	2342.77	1875.79	1672.01
250	1708.09	1852.43	2340.23	1889.48	1673.93
260	1740.90	1868.73	2352.02	1906.10	1706.08
270	1763.43	1905.29	2406.03	1943.40	1728.16
280	1807.01	1959.88	2495.59	1999.08	1770.87
290	1861.87	2013.98	2548.64	2054.26	1824.63
300	1907.34	2073.85	2604.34	2115.33	1869.19
313	1927.12	2095.13	2626.64	2137.03	1888.58

Table B.10 Model M3: Concrete Strain At Compression Side Of Barrier Wall

Applied Static Load (KN)	Concrete Strain , C1	Concrete Strain , C2	Concrete Strain , C3	Concrete Strain , C4	Concrete Strain , C5
0	0.00	0.00	0.00	0.00	0.00
10	0.00	0.00	0.00	0.00	0.00
20	0.00	-0.94	0.00	-0.67	-0.05
30	-0.08	-1.41	-0.47	-0.89	-0.09
40	-0.08	-1.41	-0.47	-1.12	-0.09
50	-1.12	-2.36	-2.35	-1.65	-1.03
60	-1.12	-2.83	-6.59	-2.13	-1.14
70	-1.22	-4.24	-16.47	-2.97	-1.28
80	-1.45	-5.65	-33.42	-3.56	-1.50
90	-1.62	-6.60	-53.65	-4.54	-1.69
100	-1.74	-8.01	-74.36	-6.78	-1.82
110	-2.12	-7.54	-95.54	-7.12	-2.45
120	-2.74	-8.48	-118.12	-7.58	-2.97
130	-3.22	-9.89	-140.71	-8.33	-3.44
140	-3.57	-10.36	-164.23	-9.07	-3.67
150	-4.02	-11.78	-185.41	-9.89	-4.44
160	-4.47	-12.72	-205.64	-10.67	-4.77
170	-6.32	-16.02	-227.75	-14.45	-6.54
180	-12.11	-19.32	-247.98	-16.23	-12.85
190	-17.40	-21.20	-268.68	-18.65	-18.20
200	-18.22	-25.44	-287.96	-22.58	-19.40
210	-24.20	-30.15	-308.19	-27.17	-25.80
220	-26.74	-34.86	-327.95	-29.45	-28.60
230	-32.45	-40.51	-354.28	-36.78	-35.32
240	-36.54	-46.17	-374.04	-41.45	-38.40
250	-41.40	-52.76	-397.08	-48.86	-45.75
260	-48.76	-60.77	-422.01	-56.78	-52.40
270	-50.88	-68.31	-452.10	-63.56	-55.63
280	-56.76	-77.73	-493.01	-71.98	-61.35
290	-61.34	-94.21	-538.62	-84.59	-68.90
300	-68.99	-110.23	-568.43	-98.54	-72.80
313	-76.40	-147.44	-627.76	-140.45	-85.4

Table B.11 Model M3: Steel Strain At Load Facing Side Of Vertical Rebar

Applied Static Load (KN)	Steel Strain , S1	Steel Strain , S2	Steel Strain , S3	Steel Strain , S4	Steel Strain , S5	Steel Strain , S6	Steel Strain , S7	Steel Strain , S8
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	2.42	4.85	2.43	4.44	4.36	2.43	3.40	1.85
20	5.33	10.18	5.34	8.12	7.74	5.82	5.34	3.55
30	8.12	14.55	10.19	13.15	12.45	7.28	8.26	7.23
40	11.56	16.97	12.62	19.88	18.50	9.70	11.17	8.12
50	13.45	17.45	15.04	26.45	25.76	13.59	13.12	10.34
60	15.03	19.88	19.90	32.98	31.94	16.50	15.54	12.54
70	19.88	23.76	22.32	38.34	37.27	18.44	18.94	15.18
80	21.82	27.64	24.27	43.23	41.14	20.86	21.37	17.86
90	26.19	29.58	24.75	48.12	44.53	22.32	21.37	21.44
100	28.61	32.49	25.72	51.40	48.88	24.75	24.29	24.54
110	31.04	34.43	26.21	53.78	51.79	28.14	26.72	27.45
120	33.46	37.33	27.66	57.43	55.17	31.54	28.66	29.54
130	36.85	42.67	30.09	58.76	56.63	34.45	30.60	32.86
140	39.28	44.12	33.00	60.10	59.05	36.39	35.46	35.36
150	41.22	47.03	32.52	64.78	62.92	37.36	37.89	37.77
160	45.10	50.91	33.97	72.40	68.73	40.76	40.32	39.65
170	46.55	55.28	36.40	75.80	71.15	43.19	43.72	42.54
180	48.98	58.67	37.85	79.54	76.47	45.61	48.09	44.75
190	53.34	61.10	40.28	84.40	81.80	50.46	51.49	46.77
200	55.28	64.00	42.71	87.12	85.67	52.89	53.43	49.86
210	59.16	66.43	45.13	94.40	90.99	56.29	57.32	53.33
220	65.47	69.82	49.50	105.80	100.67	64.05	64.12	57.23
230	75.17	74.19	62.61	113.80	115.20	74.24	71.41	63.44
240	87.29	76.13	68.43	116.78	116.65	76.67	75.78	71.34
250	93.60	80.01	69.89	118.60	116.65	77.15	77.73	79.96
260	96.51	83.40	70.37	121.56	117.62	79.09	79.18	83.11
270	100.39	87.28	70.86	123.40	120.04	81.04	81.61	89.23
280	102.33	89.22	71.88	124.54	121.49	82.49	83.07	96.33
290	104.75	89.71	72.34	126.43	122.46	82.49	84.53	98.12
300	105.25	93.10	74.32	127.88	123.34	83.23	85.12	99.05
313	106.88	96.12	76.45	129.23	125.12	84.21	86.88	100.79

Table B.12 Model M3: Steel Strain At Load Facing Side Of Horizontal Rebar

Applied Static Load (KN)	Steel Strain SF1	Steel Strain SF2	Steel Strain SF3	Steel Strain SF4	Steel Strain SF5
0	0.00	0.00	0.00	0.00	0.00
10	-1.66	-1.45	-1.45	-0.78	-0.48
20	-3.98	-3.39	-2.90	-0.89	-0.67
30	-6.02	-5.32	-4.36	-1.12	-0.97
40	-9.12	-8.23	-4.84	-1.86	-1.94
50	-20.12	-10.16	-7.26	-2.45	-1.94
60	-25.14	-12.58	-8.23	-3.23	-2.91
70	-28.33	-17.22	-9.19	-3.46	-1.94
80	-33.43	-22.65	-10.65	-4.56	-2.42
90	-36.50	-28.19	-12.10	-5.65	-3.87
100	-39.54	-32.15	-13.55	-8.54	-3.39
110	-44.67	-38.45	-15.97	-9.45	-6.78
120	-47.18	-40.16	-18.39	-10.67	-9.20
130	-50.81	-42.09	-20.32	-12.68	-10.65
140	-53.32	-45.00	-22.74	-18.88	-14.04
150	-58.90	-48.38	-26.61	-22.40	-19.85
160	-62.40	-51.28	-30.00	-24.50	-23.73
170	-66.34	-53.70	-33.39	-28.40	-26.15
180	-69.74	-56.61	-34.84	-31.20	-28.57
190	-72.33	-59.51	-38.23	-35.48	-31.96
200	-75.80	-62.41	-41.13	-39.12	-32.93
210	-78.12	-65.31	-44.03	-45.40	-36.80
220	-81.43	-69.18	-47.90	-46.20	-41.16
230	-85.76	-71.66	-51.88	-49.66	-51.81
240	-88.43	-74.23	-59.12	-53.71	-57.62
250	-93.23	-77.45	-63.78	-58.40	-60.04
260	-95.19	-79.65	-67.23	-63.51	-61.50
270	-96.95	-82.66	-70.40	-65.83	-64.40
280	-99.49	-85.97	-74.23	-69.94	-67.31
290	-102.42	-88.37	-78.83	-73.25	-72.63
300	-105.40	-91.73	-81.40	-75.78	-75.12
313	-110.15	-96.12	-86.32	-82.06	-79.86

Table B.13 Model M3: Steel Strain At Rear Side Of Horizontal Rebar

Applied Static Load (KN)	Steel Strain , SR1	Steel Strain , SR2	Steel Strain , SR3	Steel Strain , SR4
0	0.00	0.00	0.00	0.00
10	1.65	1.24	0.99	0.58
20	2.47	1.85	1.48	0.86
30	2.88	2.16	1.73	1.01
40	4.11	3.09	2.47	1.44
50	4.11	3.09	2.47	1.44
60	4.11	3.09	2.47	1.44
70	5.75	4.32	3.45	2.01
80	9.05	6.79	5.43	3.17
90	10.29	7.71	6.17	3.60
100	13.17	9.87	7.90	4.61
110	15.63	11.72	9.38	5.47
120	17.28	12.96	10.37	6.05
130	19.75	14.81	11.85	6.91
140	24.68	18.51	14.81	8.64
150	35.79	26.84	21.47	12.52
160	68.29	51.22	40.97	23.90
170	122.60	91.95	73.56	42.91
180	239.87	179.90	143.92	83.95
190	310.67	233.00	186.40	108.73
200	418.53	313.90	251.12	146.49
210	519.42	389.56	311.65	181.80
220	609.20	456.90	365.52	213.22
230	698.61	523.95	419.16	244.51
240	815.63	611.73	489.38	285.47
250	943.82	707.87	566.29	330.34
260	1018.85	764.14	611.31	356.60
270	1079.06	809.29	647.43	377.67
280	1119.88	839.91	671.93	391.96
290	1162.45	871.84	697.47	406.86
300	1208.67	906.50	725.20	423.03
313	1302.43	976.8225	781.458	455.8505

Table B.14 Model M3: Steel Strain On Top Of Deck Slab's Main Rebar

Applied Static Load (KN)	Steel Strain SL1	Steel Strain SL2	Steel Strain SL3	Steel Strain SL4	Steel Strain SL5	Steel Strain SL6
0	0.00	0.00	0.00	0.00	0.00	0.00
10	3.85	4.23	5.80	5.79	4.37	4.36
20	9.69	10.12	13.40	12.06	10.20	10.18
30	21.40	15.80	34.50	30.39	22.82	16.97
40	38.65	40.12	58.70	54.03	44.19	42.18
50	80.90	112.80	145.60	130.26	75.76	114.92
60	142.40	200.34	245.65	232.07	135.99	205.61
70	250.50	306.78	418.40	400.04	239.95	312.33
80	298.32	365.89	470.90	451.22	284.64	371.52
90	354.40	431.42	542.50	526.06	340.52	438.97
100	437.60	530.30	661.40	633.27	422.65	536.51
110	511.23	611.65	759.74	726.97	497.02	616.12
120	584.12	680.70	828.40	808.62	561.67	690.39
130	636.40	755.40	895.40	878.20	616.61	752.54
140	705.34	818.54	983.46	951.65	673.99	813.73
150	761.42	880.60	1046.92	1017.38	729.42	875.41
160	823.56	938.54	1118.43	1089.89	784.87	929.32
170	878.40	1005.75	1202.54	1176.91	852.96	992.96
180	967.43	1085.65	1318.40	1281.36	933.72	1074.09
190	1012.72	1145.12	1392.50	1347.62	979.45	1130.94
200	1044.69	1188.23	1438.50	1388.73	1004.26	1160.58
210	1083.57	1295.40	1496.43	1467.58	1060.22	1280.63
220	1175.12	1415.67	1603.40	1575.48	1154.64	1392.93
230	1319.40	1573.64	1768.32	1732.28	1281.21	1556.80
240	1396.40	1650.50	1868.40	1813.61	1342.07	1628.30
250	1423.40	1669.80	1904.23	1817.00	1355.21	1649.21
260	1453.20	1692.40	1988.32	1845.56	1379.07	1675.00
270	1475.60	1725.80	2032.54	1874.61	1402.93	1704.18
280	1508.37	1768.90	2104.23	1915.77	1437.51	1744.08
290	1539.43	1802.40	2178.32	1967.10	1476.96	1788.84
300	1567.34	1843.60	2218.32	1998.65	1507.80	1829.12
313	1612.32	1897.18	2285.4	2052.13	1560.45	1878.3

Table B.15 Model M4: Deflection

Applied Static Load (KN)	LVDT 1	LVDT 2	LVDT 3	LVDT 4	LVDT 5
0	0	0	0	0	0
10	0.42	0.44	0.54	0.44	0.33
20	0.91	0.96	1.2	0.97	0.83
30	2.02	2.05	3.4	2.02	1.77
40	3.8	4.1	4.56	3.85	3.12
50	5.67	5.79	6.72	5.37	4.37
60	7.02	7.77	10.23	7.23	5.97
70	9.12	9.67	11.23	9.04	7.44
80	10.54	11.64	12.65	10.91	8.87
90	12.42	13.1	14.53	12.38	10.02
100	13.55	14.64	16.75	13.9	11.18
110	15.11	16.47	18.54	15.58	12.47
120	17.45	18.21	20.44	17.46	13.94
130	18.76	19.8	23.64	19.35	15.41
140	19.65	20.62	25.76	20.57	16.35
150	20.71	21.51	26.43	22.16	17.58
160	21.37	22.43	27.56	24.16	19.29
170	22.93	23.29	28.97	24.79	20.4
180	23.4	24.38	29.76	25.34	21.88
190	24.53	24.88	30.43	26.23	22.61
200	25.12	25.87	31.33	27.43	23.78
210	25.9	26.45	32.12	28.26	24.24
220	25.91	27.23	33.67	29.76	24.97
230	25.96	28.65	35.65	30.02	25.34
240	26.06	29.58	38.55	30.87	25.87
250	26.12	30.33	40.02	31.33	26.56
260	28.35	31.56	41.44	32.45	27.12
270	29.18	32.99	43.56	33.61	27.68
279	30.44	33.76	44.28	34.72	28.43

Table B.16 Model M4: Post-tensioned Rod Strain

Applied Static (KN)	Load	Rod Strain , SB1	Rod Strain , SB2	Rod Strain , SB3	Rod Strain , SB4	Rod Strain , SB5
0		805.00	805.00	805.00	805.00	805.00
10		805.00	805.00	805.00	805.00	805.00
20		805.00	807.45	823.60	806.12	805.00
30		805.00	812.34	828.59	828.59	810.23
40		872.98	890.44	908.24	908.24	855.52
50		900.29	927.30	955.12	945.85	882.29
60		952.32	990.41	1030.03	1010.22	933.27
70		1003.30	1053.47	1106.14	1074.54	983.24
80		1017.45	1088.68	1164.88	1110.45	997.10
90		1093.96	1181.47	1275.99	1205.10	1072.08
100		1122.26	1223.26	1333.36	1247.73	1099.81
110		1173.30	1278.90	1394.00	1304.47	1149.83
120		1244.12	1356.09	1478.14	1383.21	1219.24
130		1252.59	1377.84	1515.63	1405.40	1227.53
140		1280.95	1434.66	1606.82	1463.35	1255.33
150		1323.40	1495.45	1689.85	1525.36	1296.94
160		1337.56	1524.81	1738.29	1555.31	1310.80
170		1341.21	1542.39	1773.75	1573.24	1314.39
180		1428.15	1656.66	1921.72	1689.79	1399.59
190		1484.82	1737.24	2032.57	1771.98	1455.12
200		1524.49	1798.90	2122.70	1834.88	1494.00
210		1532.96	1824.22	2170.82	1860.70	1502.30
220		1549.95	1859.94	2231.93	1897.14	1518.95
230		1626.45	1968.01	2381.29	2007.37	1593.92
240		1634.92	1994.60	2433.42	2034.50	1602.22
250		1643.45	2021.44	2486.37	2061.87	1610.58
260		1649.07	2044.85	2535.62	2085.75	1616.09
270		1652.50	2065.62	2582.03	2106.93	1619.45
279		1660.21	2091.86	2635.75	2133.70	1627.01

Table B.17 Model M4: Concrete Strain At Compression Side Of Barrier Wall

Applied Static Load (KN)	Concrete Strain ,C1	Concrete Strain ,C2	Concrete Strain ,C3	Concrete Strain ,C4	Concrete Strain ,C5
0	0.00	0.00	0.00	0.00	0.00
10	0.00	-5.65	-11.32	-4.28	-4.71
20	0.00	-10.84	-23.12	-9.04	-8.48
30	0.00	-16.96	-39.16	-15.69	-12.25
40	0.00	-22.62	-55.67	-21.40	-16.02
50	0.00	-31.10	-72.19	-25.68	-18.85
60	-4.76	-37.23	-84.45	-32.81	-21.67
70	-9.99	-46.18	-102.85	-36.14	-24.50
80	-16.65	-57.96	-126.44	-39.95	-26.85
90	-21.40	-67.85	-145.30	-46.60	-28.27
100	-25.68	-81.52	-165.59	-52.78	-26.38
110	-31.86	-94.24	-193.42	-60.39	-24.50
120	-36.62	-101.78	-217.94	-71.80	-29.21
130	-39.95	-112.61	-245.29	-84.64	-27.80
140	-44.23	-121.09	-270.76	-101.28	-30.62
150	-49.94	-135.70	-301.41	-120.78	-30.15
160	-56.12	-147.00	-363.18	-141.22	-31.57
170	-60.87	-148.12	-428.24	-147.54	-33.45
180	-68.01	-149.36	-522.05	-148.32	-36.75
190	-70.86	-155.43	-578.61	-152.34	-38.16
200	-77.52	-165.43	-642.23	-158.76	-37.69
210	-83.22	-174.40	-701.60	-165.43	-43.34
220	-85.60	-180.34	-748.24	-171.23	-51.82
230	-88.45	-190.40	-800.54	-179.56	-59.36
240	-86.55	-210.32	-849.53	-189.32	-65.96
250	-80.85	-230.40	-892.86	-211.87	-72.08
260	-77.04	-235.65	-931.48	-223.55	-77.73
270	-74.66	-240.54	-955.03	-235.76	-77.73
279	-75.62	-243.45	-976.22	-237.54	-77.26

Table B.18 Model M4: Steel Strain At Load Facing Side Of Vertical Rebar

Applied Static Load (KN)	Steel Strain S1	Steel Strain S2	Steel Strain S3	Steel Strain S4	Steel Strain S5	Steel Strain S6	Steel Strain S7	Steel Strain S8
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	2.66	5.34	2.67	4.88	4.80	2.67	3.74	2.04
20	5.86	11.20	5.87	8.93	8.51	6.40	5.87	3.91
30	8.93	16.01	11.21	14.47	13.70	8.01	9.09	7.95
40	12.72	18.67	13.88	21.87	20.35	10.67	12.29	8.93
50	14.80	19.20	16.54	29.10	28.34	14.95	14.43	11.37
60	16.53	21.87	21.89	36.28	35.13	18.15	17.09	13.79
70	21.87	26.14	24.55	42.17	41.00	20.28	20.83	16.70
80	24.00	30.40	26.70	47.55	45.25	22.95	23.51	19.65
90	28.81	32.54	27.23	52.93	48.98	24.55	23.51	23.58
100	31.47	35.74	28.29	56.54	53.77	27.23	26.72	26.99
110	34.14	37.87	28.83	59.16	56.97	30.95	29.39	30.20
120	36.81	41.06	30.43	63.17	60.69	34.69	31.53	32.49
130	40.54	46.94	33.10	64.64	62.29	37.90	33.66	36.15
140	43.21	48.53	36.30	66.11	64.96	40.03	39.01	38.90
150	45.34	51.73	35.77	71.26	69.21	41.10	41.68	41.55
160	49.61	56.00	37.37	79.64	75.60	44.84	44.35	43.62
170	51.21	60.81	40.04	83.38	78.27	47.51	48.09	46.79
180	53.88	64.54	41.64	87.49	84.12	50.17	52.90	49.23
190	58.67	67.21	44.31	92.84	89.98	55.51	56.64	51.45
200	60.81	70.40	46.98	95.83	94.24	58.18	58.77	54.85
210	65.08	73.07	49.64	103.84	100.09	61.92	63.05	58.66
220	72.02	76.80	54.45	116.38	110.74	70.46	70.53	62.95
230	82.69	81.61	68.87	125.18	126.72	81.66	78.55	69.78
240	96.02	83.74	75.27	128.46	128.32	84.34	83.36	78.47
250	102.96	88.01	76.88	130.46	128.32	84.87	85.50	87.96
260	106.16	91.74	77.41	133.72	129.38	87.00	87.10	91.42
270	110.43	96.01	77.95	135.74	132.04	89.14	89.77	98.15
279	112.56	98.14	79.07	136.99	133.64	90.74	91.38	105.96

Table B.19 Model M4: Steel Strain At Load Facing Side Of Horizontal Rebar

Applied Static Load (KN)	Steel Strain, SF1	Steel Strain, SF2	Steel Strain, SF3	Steel Strain, SF4	Steel Strain, SF5
0	0.00	0.00	0.00	0.00	0.00
10	-1.91	-1.65	-1.64	-0.87	-0.53
20	-4.58	-3.86	-3.28	-1.00	-0.74
30	-6.92	-6.06	-4.93	-1.25	-1.08
40	-10.49	-9.38	-5.47	-2.08	-2.15
50	-23.14	-11.58	-8.20	-2.74	-2.15
60	-28.91	-14.34	-9.30	-3.62	-3.23
70	-32.58	-19.63	-10.38	-3.88	-2.15
80	-38.44	-25.82	-12.03	-5.11	-2.69
90	-41.98	-32.14	-13.67	-6.33	-4.30
100	-45.47	-36.65	-15.31	-9.56	-3.76
110	-51.37	-43.83	-18.05	-10.58	-7.53
120	-54.26	-45.78	-20.78	-11.95	-10.21
130	-58.43	-47.98	-22.96	-14.20	-11.82
140	-61.32	-51.30	-25.70	-21.15	-15.58
150	-67.74	-55.15	-30.07	-25.09	-22.03
160	-71.76	-58.46	-33.90	-27.44	-26.34
170	-76.29	-61.22	-37.73	-31.81	-29.03
180	-80.20	-64.54	-39.37	-34.94	-31.71
190	-83.18	-67.84	-43.20	-39.74	-35.48
200	-87.17	-71.15	-46.48	-43.81	-36.55
210	-89.84	-74.45	-49.75	-50.85	-40.85
220	-93.64	-78.87	-54.13	-51.74	-45.69
230	-98.62	-81.69	-58.62	-55.62	-57.51
240	-101.69	-84.62	-66.81	-60.16	-63.96
250	-107.21	-88.29	-72.07	-65.41	-66.64
260	-109.47	-90.80	-75.97	-71.13	-68.27
270	-111.49	-94.23	-79.55	-73.73	-71.48
279	-114.41	-98.01	-83.88	-78.33	-74.71

Table B.20 Model M4: Steel Strain At Rear Side Of Horizontal Rebar

Applied Static Load (KN)	Steel Strain, SR1	Steel Strain, SR2	Steel Strain, SR3	Steel Strain, SR4
0	0.00	0.00	0.00	0.00
10	1.94	1.46	1.16	0.68
20	2.90	2.18	1.74	1.02
30	3.39	2.54	2.03	1.19
40	4.84	3.63	2.90	1.69
50	4.84	3.63	2.90	1.69
60	4.84	3.63	2.90	1.69
70	6.77	5.08	4.06	2.37
80	10.65	7.99	6.39	3.73
90	12.10	9.08	7.26	4.24
100	15.49	11.62	9.29	5.42
110	18.39	13.79	11.03	6.44
120	20.33	15.25	12.20	7.12
130	23.23	17.42	13.94	8.13
140	29.04	21.78	17.42	10.16
150	42.10	31.58	25.26	14.74
160	80.34	60.26	48.20	28.12
170	144.23	108.17	86.54	50.48
180	282.20	211.65	169.32	98.77
190	365.49	274.12	219.29	127.92
200	492.39	369.29	295.43	172.34
210	611.08	458.31	366.65	213.88
220	716.71	537.53	430.03	250.85
230	821.89	616.42	493.13	287.66
240	959.57	719.68	575.74	335.85
250	1110.38	832.79	666.23	388.63
260	1198.65	898.99	719.19	419.53
270	1269.48	952.11	761.69	444.32
279	1317.51	988.13	790.51	461.13

Table B.21 Model M5: Deflection

Applied Static Load (KN)	LVDT 1	LVDT 2	LVDT 3	LVDT 4	LVDT 5
0	0	0	0	0	0
10.00	0.02	0.03	0.03	0.02	0.00
20.00	0.15	0.12	0.10	0.05	0.01
30.00	0.88	0.73	0.57	0.35	0.17
40.00	2.15	1.72	1.32	1.08	0.96
50.00	3.49	2.80	2.15	1.76	1.56
60.00	4.66	3.73	2.87	2.35	2.08
70.00	6.23	4.98	3.83	3.14	2.78
80.00	7.48	5.99	4.60	3.77	3.34
90.00	8.60	6.88	5.29	4.34	3.84
100.00	9.57	7.65	5.89	4.83	4.27
110.00	10.87	8.69	6.69	5.48	4.85
120.00	11.90	9.52	7.32	6.00	5.31
130.00	13.19	10.56	8.12	6.66	5.89
140.00	13.96	11.17	8.59	7.04	6.23
150.00	14.96	11.97	9.21	7.55	6.68
160.00	15.61	12.49	9.61	7.88	6.97
170.00	16.05	12.84	10.27	8.47	7.05
180.00	16.63	13.42	10.54	8.68	7.18
190.00	17.89	14.05	10.95	8.90	7.27
200.00	19.21	14.70	11.27	9.08	7.37
210.00	21.53	15.66	11.67	9.29	7.47

Table B.22 Model M5: Post-tensioned Rod Strain

Applied Static Load (KN)	Rod SB1 Strain,	Rod SB2 Strain,	Rod SB3 Strain,	Rod SB4 Strain,	Rod SB5 Strain,
0	804.97	804.00	804.00	803.52	803.44
10	803.52	804.97	804.22	803.03	803.58
20	802.55	805.94	804.67	804.48	804.12
30	801.58	805.94	803.76	804.97	804.39
40	796.75	808.37	803.43	803.52	804.41
50	799.16	818.58	804.87	804.00	804.52
60	804.00	828.30	805.43	803.52	804.85
70	810.29	836.08	806.54	803.52	805.12
80	824.31	846.29	812.23	805.94	806.43
90	847.05	858.44	819.43	807.87	808.34
100	899.77	865.25	845.43	818.53	815.23
110	907.99	879.35	869.78	842.75	832.23
120	957.34	897.82	902.45	875.20	855.23
130	1054.11	929.43	951.92	948.83	893.12
140	1137.83	986.32	1024.54	1073.83	945.23
150	1224.95	1048.07	1113.23	1161.54	1032.63
160	1314.02	1134.16	1167.12	1236.67	1134.98
170	1343.55	1238.75	1234.76	1287.57	1187.77
180	1480.59	1327.30	1305.65	1342.35	1209.91
190	1609.43	1411.49	1412.54	1401.50	1252.84
200	1764.96	1482.06	1470.34	1457.75	1302.32
210	2067.91	1538.04	1515.43	1498.97	1345.82

Table B.23 Model M5: Concrete Strain At Compression Side Of Barrier Wall

Applied Static Load (KN)	Concrete Strain , C1	Concrete Strain , C2	Concrete Strain , C3	Concrete Strain , C4	Concrete Strain , C5
0	0.00	0.00	0.00	0.00	0.00
10	-3.12	-2.83	-2.35	0.00	0.00
20	-7.14	-6.13	-5.17	0.00	0.00
30	-12.23	-10.84	-7.05	0.00	0.00
40	-34.34	-24.50	-5.17	-0.86	-0.34
50	-49.23	-36.75	-7.99	-0.86	-0.45
60	-67.43	-49.47	-11.28	-0.57	-0.51
70	-82.50	-59.84	-13.63	-0.29	-0.64
80	-102.80	-72.56	-17.86	-0.57	-0.70
90	-112.24	-81.98	-21.62	-0.57	-0.85
100	-119.34	-91.40	-27.73	-1.43	-0.99
110	-128.87	-99.89	-36.19	-1.15	-1.03
120	-141.32	-109.31	-42.77	-2.29	-1.23
130	-159.54	-114.49	-50.76	-2.87	-1.55
140	-171.23	-127.68	-55.46	-5.45	-3.45
150	-186.87	-137.57	-60.16	-7.74	-4.23
160	-203.34	-154.06	-62.51	-9.46	-5.01
170	-223.43	-180.91	-59.69	-9.75	-5.56
180	-245.98	-188.45	-46.53	-11.19	-5.89
190	-283.54	-195.04	-43.71	-11.76	-6.87
200	-312.86	-214.82	-51.70	-12.91	-7.23
210	-356.75	-227.07	-72.38	-12.62	-8.12

Table B.24 Model M5: Steel Strain At Load Facing Side Of Vertical Rebar

Applied Static Load (KN)	Steel Strain , S1	Steel Strain , S2	Steel Strain , S3	Steel Strain , S4	Steel Strain , S5	Steel Strain , S6
0	0.00	0.00	0.00	0.00	0.00	0.00
10	4.84	5.80	4.83	0.48	5.79	3.37
20	10.64	11.60	9.66	3.87	11.58	9.16
30	14.99	16.44	15.45	6.30	17.85	13.49
40	22.73	23.20	23.66	10.17	25.09	21.69
50	29.50	29.01	28.97	14.05	29.43	26.51
60	38.20	38.67	36.22	15.98	35.70	34.22
70	47.39	46.89	38.63	19.86	41.97	39.52
80	57.55	57.53	42.49	23.25	49.69	46.75
90	70.61	68.65	48.29	25.67	56.44	50.61
100	85.60	85.09	56.98	31.97	58.86	55.43
110	98.66	96.69	63.26	35.84	64.65	61.69
120	118.97	115.55	70.99	41.66	76.23	66.99
130	148.96	138.76	81.61	47.95	88.77	71.81
140	186.21	186.63	96.58	69.75	137.02	159.55
150	232.65	237.89	109.14	91.07	165.49	167.26
160	309.58	339.46	144.88	137.09	234.99	171.60
170	344.42	390.74	181.59	162.77	263.46	180.28
180	423.79	472.51	227.97	183.60	297.73	177.38
190	465.90	517.51	264.69	202.50	423.73	203.90
200	502.20	562.52	319.77	233.02	428.56	352.42
210	548.67	621.08	384.52	265.49	460.91	405.47

Table B.25 Model M5: Steel Strain At Load Facing Side Of Horizontal Rebar

Applied Static Load (KN)	Steel Strain SF1	Steel Strain SF2	Steel Strain SF3	Steel Strain SF4	Steel Strain SF5
0	0.00	0.00	0.00	0.00	0.00
10	-1.93	-1.05	-0.48	-1.45	-0.97
20	-3.86	-2.65	-0.48	-1.45	-0.97
30	-5.79	-3.54	0.00	-0.48	-0.97
40	-2.90	-4.02	0.48	-1.93	-1.45
50	-4.83	-4.55	0.48	-1.93	-1.45
60	-6.27	-5.43	0.97	0.48	-1.45
70	-8.69	-6.74	1.94	0.97	-2.42
80	-11.10	-8.22	3.39	2.42	-1.94
90	-11.58	-9.34	3.87	3.38	-1.94
100	-14.00	-11.23	5.32	4.83	-0.97
110	-10.62	-8.32	7.26	9.18	0.00
120	-9.17	-2.34	8.71	12.08	0.48
130	-7.24	-1.23	14.03	17.88	4.36
140	3.38	1.23	15.24	27.55	3.39
150	10.14	3.54	16.82	50.26	3.87
160	41.51	32.45	17.86	99.56	7.75
170	101.37	88.93	21.29	104.88	10.65
180	172.33	134.54	113.21	135.81	18.89
190	222.55	188.12	191.61	201.07	26.15
200	284.36	235.65	257.92	264.41	34.38
210	775.25	537.65	380.87	307.93	44.55

Table B.26 Model M5: Steel Strain At Rear Side Of Horizontal Rebar

Applied Static Load (KN)	Steel Strain , SR1	Steel Strain , SR2	Steel Strain , SR3	Steel Strain , SR4
0	0.00	0.00	0.00	0.00
10	2.41	0.00	0.00	0.00
20	5.31	0.00	0.97	0.00
30	9.17	1.45	0.97	-0.48
40	12.07	1.45	0.48	-0.97
50	14.97	1.94	0.97	-1.93
60	20.28	3.87	1.45	-2.42
70	25.11	5.32	1.93	-2.42
80	28.00	6.29	1.93	-3.87
90	33.32	6.77	2.41	-4.84
100	53.60	12.09	2.90	-5.32
110	115.89	23.22	5.31	-6.29
120	159.84	34.83	7.73	-7.74
130	209.11	54.67	9.66	-8.70
140	317.32	106.45	19.80	-8.22
150	380.13	129.19	28.01	-6.77
160	525.59	208.07	44.91	3.87
170	701.07	375.08	53.61	17.41
180	903.21	532.46	70.51	30.47
190	1052.70	622.06	95.14	42.07
200	1191.58	699.09	127.51	48.36
210	1386.18	763.44	206.25	58.52

Table B.27 Model M1: Applied pre-tensioned force

Equivalent Torque (ft-lb)	Equivalent Static Load (KN)	Rod Strain, SB1
0	0	0
50	7.49	72.65
100	14.97	145.30
150	22.46	217.94
200	29.94	290.59
250	37.43	363.24
300	44.91	435.89
350	52.40	508.54
400	59.88	581.19
450	67.37	653.83
500	74.85	726.48
550	82.34	799.13
562	84.13	816.57

Table B.28 Model M3: Applied pre-tensioned force

Equivalent Torque (ft-lb)	Equivalent Static Load (KN)	Rod Strain, SB1
0	0	0
50	7.49	72.65
100	14.97	145.30
150	22.46	217.94
200	29.94	290.59
250	37.43	363.24
300	44.91	435.89
350	52.40	508.54
400	59.88	581.19
450	67.37	653.83
500	74.85	726.48
550	82.34	799.13
554.04	82.94	805.00

Table B.29 Model M4: Applied pre-tensioned force

Equivalent Torque (ft-lb)	Equivalent Static Load (KN)	Rod Strain, SB1
0	0	0
50	7.49	72.65
100	14.97	145.30
150	22.46	217.94
200	29.94	290.59
250	37.43	363.24
300	44.91	435.89
350	52.40	508.54
400	59.88	581.19
450	67.37	653.83
500	74.85	726.48
550	82.34	799.13
554.04	82.94	805.00

Table B.30 Model M5: Applied pre-tensioned force

Equivalent Torque (ft-lb)	Equivalent Static Load (KN)	Rod Strain, SB1
0	0	0
50	7.49	72.65
100	14.97	145.30
150	22.46	217.94
200	29.94	290.59
250	37.43	363.24
300	44.91	435.89
350	52.40	508.54
400	59.88	581.19
450	67.37	653.83
500	74.85	726.48
550	82.34	799.13
554.02	82.94	804.97

CERTIFIED TEST REPORT

CHEMICAL ANALYSIS (WT %)

C	Mn	P	S	Si	Sn	Cu	Ni	Cr	Mo	Cb	V	Co	Al	Ti	Cu _{total}	N _{total}
0.52	1.48	0.008	0.025	0.87	0.022	0.10	0.44	0.23	0.03	0.001	0.031	0.009	0.003	0.0035	23	137

MATERIAL 100% MILLED AND ROLLED IN THE USA. MANUFACTURING PROCESSES FOR THIS STEEL, WHICH MAY INCLUDE SCRAP MELTED IN AN ELECTRIC ARC FURNACE AND HOT ROLLING, HAVE BEEN PERFORMED AT GERDAU AMERISTEEL MINNESOTA, 1678 RED ROCK ROAD, SAINT PAUL MINNESOTA, USA. ALL PRODUCT PRODUCED FROM STRAND CAST BILLETS. NO WELD REPAIRMENT PERFORMED. STEEL NOT EXPOSED TO MERCURY OR ANY LIQUID ALLOY WHICH IS LIQUID AT AMBIENT TEMPERATURES DURING PROCESSING OR WHILE IN GERDAU AMERISTEEL MINNESOTA POSSESSION.

JOMINY END QUENCH HARDENABILITY RESULTS (HRC)

* J1	J2	J3	J4	J5	J6	J7	J8	J9	J10	J11	J12
J13	J14	J15	J16	J18	J20	J22	J24	J26	J28	J30	J32

MECHANICAL TEST REPORT

SPECIMEN AREA (in ²)	YIELD (ksi)	TENSILE (ksi)	ELONG. (%)	RED. (%)
0.920	113.8	145.6	21.60	8.10
0.917	125.8	149.2	21.60	9.28

Additional Specifications/Comments:

Meets Material requirements of A722-88

A722/A722M-05

Grain Size:	Reduction Ratio:	C.E. Per:	As Rolled surface Hardness
Coding:	D.I. 5.38 in. Max 488.6 lbs F.	C.E.:	HBW HRC
			Test 1:
			Test 2:

CHARPY IMPACT TEST

* Test 1	Test 2
Temp (F)	
ft-lb 1	
ft-lb 2	
ft-lb 3	

ASTM E111 and a rigorous controlled test.

Micro Clean Average									
A:	B:	C:	D:	E:	F:	G:	H:	I:	J:
Macro Etch:									

The above results relate only to the items tested.

Chemical tests performed in accordance with ASTM E115 and E1018. Mechanical tests performed in accordance with ASTM E8, E10, E18, E200 and A370. All other tests performed in accordance with the requirements of applicable specifications unless otherwise noted above. We hereby certify that the above test results are representative of those contained in the records of the company.

Any modification to this certificate as provided by Gerdau Ameristeel - Minnesota without the expressed written consent of Gerdau Ameristeel - Minnesota negates the validity of this test report. This report shall not be reproduced except in full without the expressed written consent of Gerdau Ameristeel Minnesota. Gerdau Ameristeel - Minnesota is not responsible for the inability of this material to meet specific applications.

SIGNED:

DATE: 05-26-2006

APPROVAL:

QA Approval

Kon Wong

SWORN AND SUBSCRIBED TO BEFORE ME

THIS By: *[Signature]* Date: 6/1/06
Quality Assurance Mgr (date)

THIS CERTIFICATE IS NOTARIZED ONLY WHEN REQUESTED