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FATIGUE LIFE ASSESSMENT OF TTC STREETCAR CONCRETE PAVEMENTS

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In Partial fulfilment of the requirements for the degree of

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In the Program of Civil Engineering

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ABSTRACT

Fatigue Life Assessment of TTC Streetcar Concrete Pavement

Masc., Civil Engineering, Ryerson University, Toronto, Canada, 2006

The Toronto Transit Commission, TTC traditional embedded track for mechanically jointed rails, is performing satisfactory except for accelerated surface concrete deterioration. By far the most damaging group of deterioration processes in streetcars concrete pavement is due to wheel impact on the joints which will subsequently excite a response on the track. The enhancement of special trackwork, STW service life was the initiative to explore encapsulation technology advances in embedded track materials and construction methods. This aimed to extend the life cycle of the track from current average of 15 years towards target life of 50 years.

In the present study, attempts at finding and patterning the localized cracks, especially wheel impact fatigue cracks, are carried out. This issue was investigated during the course of an experimental program demonstrated by simulating the wheel impact loading in concrete under repeated load application. Specially designed specimens were used to represent sheet rubber and urethane encapsulation system proposal against the current assembly.

This research reviews the pros and cons of various factors influencing the life cycle of the current assembly and examining and assessing between alternative track construction methods and materials at the joints and examining the concrete's performance both during the impact fatigue loading resulting in inclined cracking and ongoing service life and durability issues under combined environmental and mechanical loadings.

Experimental results show that it is possible to achieve the targeted service life of 50 years, based on minimum of 10 time superior performance for either of the proposed encapsulation technologies vs. current construction methods.

To Sarah, Arvin
and
my mother
Jahan

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TERMINOLOGY

Alignment

Refers to both horizontal and vertical placement of the tracks within the corridor. The horizontal alignment refers to the location of the railroad as described by curves and tangents. The vertical alignment refers to the vertical top of rail elevation (i.e. precast grade or level).

ALRV (Articulated Light Rail Vehicle)

A term first used to refer to the articulated LRVs delivered to the Toronto Transit Commission in 1983. Used in some countries to refer to any articulated LRV.

Ballast

The crushed rock utilized as a base for the purpose of holding the track in alignment and profile.

CLRV (Canadian Light Rail Vehicle)

A term referring to the LRVs delivered to TTC (Toronto Transit Commission) in 1978. These vehicles were a European design, built in Thunder Bay, Ontario. These vehicles do not have couplers and operate in single car consists.

Crossing

A structure, used where one track crosses another at grade, and consisting of four connected frogs.

Crossover

Special track work including two switches and trackwork that allows trains to be routed from one track to another. Wooden ties are used in special trackwork, as many different lengths and shapes are needed to accommodate the diverging rails, the economy of making these in quantity is significantly increased.

Double Track

A term used to describe the type of track system used on the majority of LRT systems. Each track is separate and carries an LRV in one direction. When double tracks are used, tracks are usually delineated as:

- Northbound/Southbound
 - Eastbound/Westbound
-

Embedded Track

Rail that has been embedded in the roadway, street, or walkway.

Flangeway

The open way through a track structure, which provides a passageway for wheel flanges.

Flangeway Depth

The depth of the wheel flange passageway.

Flangeway Width

The distance between the gauge line and the guard line of a track structure, which provides a passageway for wheel flanges.

Frog

A track structure used at the intersection of two running rails to provide support for wheels and passageways for their flanges, thus permitting wheels on either rail to cross the other.

Guard Rail

A rail or other structure laid parallel with the running rail of a track to prevent wheels from being derailed; to hold wheels in correct alignment to prevent flanges from striking the points of turnout or crossing frogs or the points of switches; or to keep derailed wheels adjacent to the running rails. A Guard Rail is a passive rail and is designed to not be in contact with the inside of the wheels.

Joint Bar

A steel member, embodying beam-strength and stiffness in its structural shape and material; commonly used in pairs for the purpose of joining casting or rail ends together, and holding them accurately, evenly and firmly in position with reference to surface and gauge-side alignment.

Light Rail

An electric railway system characterized by its ability to operate single or multiple car consists along exclusive rights-of-way at ground level, on aerial structures, in subways or in streets, able to board and discharge passengers at station platforms or at street, track, or car-floor level and is normally powered by overhead electrical wires.

LRT (Light Rail Transit)

Used to describe a system that uses LRVs.

LRV (Light Rail Vehicle)

A tram, trolley, streetcar etc. constructed in the 1970s or later. The name introduced to try to give the tram or trolley a more up-market image. LRVs also have higher weights and horsepower than trolleys or streetcars.

Rail (Track)

A rolled steel shape, commonly a T-section or formed steel girder section, designed to be laid end to end in two parallel lines on cross ties or other suitable supports to form a track for railway rolling stock.

Rail Fixed Guideway System

Any light rail system, rapid rail system, monorail, inclined plane, funicular, trolley, or automated guideway transit system used for public transit and not regulated by the Federal Railroad Administration.

Rail Joint

A fastening designed to unite the abutting ends of contiguous rails.

Rail Section

The shape of the end of a rail, cut at right angles to its length. Rail mills identify the different shapes and types of rails by code numbers (i.e. 115 RE rail section).

Restraining Rail

A guard rail installed adjacent to and parallel with the inside-running rail of curved track. It bears against the backside of the wheels and steers the inside wheels of each truck around the curve, thereby discouraging the leading outside wheel flange from climbing the outside rail. A Restraining Rail is an active rail and is designed to be in constant contact with the inside of a wheel.

Route

A pre-determined course of travel assigned to each vehicle or consist as it is operated on the mainline.

Special Trackwork

All rails, track structures and fittings, other than plain unguarded track that is neither curved nor fabricated before laying.

Switch

A track structure used to divert rolling stock from one track to another.

Tangent Track

Any straight portion of a railway alignment.

Tie

The transverse member of the track structure to which the rails are spiked or otherwise fastened to proper gauge and to cushion, distribute, and transmit the stresses of traffic to the embedment concrete or subgrade material.

Tie Plate

A plate interposed between a rail or other track structure and a tie.

Track

An assembly of rails, ties, and fastenings over which rolling stock is moved.

Track Gauge

The distance between the rails measured from the inside face of one rail to the inside face of the other rail at a distance of $5/8$ " below top of rail. Standard Gauge is 4 feet, 8 $\frac{1}{2}$ inches (1435mm); TTC gauge is 4 feet, 10 $\frac{7}{8}$ inches on tangent track.

Turnout

A piece of special trackwork containing a switch and frog with guard rails by means of which an LRV may be diverted from one track to the other.

Yard

A set of tracks with defined limits for LRV cleaning, repair, storage or other purposes as designated by the rules and procedures.

NOMENCLATURE

σ_a	Stress amplitude
σ_m	Mean stress
σ_r	Range of stress:
β	Material parameter; equals to 0.0807 for the heavy and ordinary concrete and 0.0700 for light weight concrete
c_f	Loading frequency coefficient
C_f	Strengths varying with the loading frequency
DLA	Dynamic Load Allowance
f	Loading frequency in Hz
f_c	The static strength
f_{cf}	Static strength at f
$f_{c \min}$	Static strength corresponding to the loading states of $S_{\min}$
$f_{c \max}$	Static strength corresponding to the loading states of $S_{\max}$
HF	Hunting Force
L	Total Vehicle Load
LE	Emergency Breaking
LN	Normal Acceleration/Breaking
n_i	Number of cycles applied at a particular stress condition
N_f	Number of cycles to failure
N_i	Number of cycles for crack initiation
N_p	Number of cycles for crack propagation
R	Stress ratio
R'	Specialized stress ratio
RF	Rolling Force
$S_{\min}$	Minimum Stress
$S_{\max}$	Maximum Stress

Chapter 1

Introduction and Background

1.1 Introduction

There was recognition of the effects of repeated loading on the concrete pavement, even if repeated loading does not cause a fatigue failure. Repeated loading has lead to inclined cracking in the pavement at lower than expected loads, or repeated loading may cause cracking or displacement in component materials of fasteners that alters the static load carrying characteristics of the concrete at large.

The main objective of this study was threefold: fundamental understanding of the physical mechanisms governing fatigue failure of mechanical joints subjected to high cycle wheel loading and to identify and investigate all possible causes of the problem; develop alternative solutions based on available technologies in the market to extend the overall life of the system; and implement an experimental program to evaluate those alternatives versus the current assembly.

Although rubbers have been utilized in track assemblies in a number of ways, knowledge of the fatigue durability of encapsulation technology in a form useful for predicting its service life is not at present available. In the proposed research on encapsulation design for STW system's durability against fatigue, the mechanism of crack growth under fatigue loading, (across a concrete crack) response subjected to cyclic loading, as well as polymers performance are explicitly accounted for based on field observations, and accompanied with experimental program. Accordingly a model for service life of the concrete pavement encasing mechanically jointed rail was developed. This experiment confirmed that these encapsulation technologies are operative in jointed rails encased in concrete and subjected to high cycle fatigue loading.

The approach adopted in the proposed work was a combination of experimental and theoretical work. Because the immediate directed impact of the research is on concrete pavement durability, field investigation of concrete pavement performance as well as laboratory data of fatigue specimens were both emphasized. The nature of this research required combined knowledge of track analysis and engineering as well as composite materials technology engineering.

The final objective of this study was transferring knowledge from research to field application, and to develop TTC strategy of track replacement and renewal of STW, using the suitable to encapsulation technology.

1.2 Study Objectives

This study addresses the fatigue durability of concrete pavements encasing TTC streetcar mechanically jointed rails. It focuses on the fundamental understanding of items governing the lifecycle of jointed rail in concrete and on fatigue durability assessment by introducing sheet rubber and urethane encapsulation.

Present study deals with embedded track at the intersection, consisted of casting pieces which are bolted together, also known as special trackwork, STW. This is part of the track that is embedded in material forming the intersection of the road such that the plane of the rails is flush, more or less, with the adjacent surface.

The overall objective of this study is to provide recommendations as to the appropriate encapsulation technology for track replacement programs and/or the new lines and extension of the TTC Light Rail Transit, LRT. The recommendations will serve as the basis for the subsequent definition of the special trackwork design criteria to be utilized in the design and construction of the new track replacement programs. In addition, the study is to discuss other deterioration mechanisms, briefly, as related to the service life of the system.

1.3 Preface of Thesis

The fatigue life assessment of the mechanically jointed rail systems encased in the concrete, which is the subject of this research program, comprises the following tasks:

Chapter 1 presents the introduction of the study followed by historical and technical backgrounds of TTC streetcar rail system.

Chapter 2 comprises of a review of available literature on the fatigue properties of concrete, fatigue crack growth, concrete performance under repeated cyclic loadings, affecting factors, and existing models to predict the fatigue life of concrete.

Chapter 3 presents the statement of the problem, status quo, construction methodology covering jointed rail, fasteners, field observations, and degradation mechanisms including wheel impact.

Chapter 4 describes the encapsulation technology, the history of application of resilient material in TTC subway and streetcar system physical and mechanical properties of both sheet rubber and urethane.

Chapter 5 presents the experimental test program for nine specimens including 3 control samples, and 3 of sheet rubber and 3 urethane encapsulated specimens, design load, material properties for the test, equipment and instruments.

Chapter 6 presents the results of the experimental program, including the results of load, displacement and strain measurement. It compares the effect of sheet encapsulated specimens under repeated cyclic load application vs. current construction methodology. It further develops the service life assessment of in-street TTC concrete pavement on the basis of conducted test program, field observations and theoretical models.

Conclusions of the study and recommendations from the investigations are presented in chapter 8.

1.4 Historical Background

Streetcars have been an important and continuous part of Toronto public transit and urban scene for more than a century. Horse-drawn streetcars began operation in Toronto in 1861 and were replaced by electrically-powered streetcars in the 1890s. Until the opening of Toronto first three subway lines in the 1950s and 1960s, streetcars carried the majority of transit customers in Toronto. While the streetcar network was reduced in size between 1947 and 1976, largely as a result of the replacement of streetcar routes by subway service, streetcars today still provide the majority of surface transit service in the downtown core. Four of the TTC five busiest surface routes are streetcar routes. Approximately 260,000 customer-trips are made on streetcar routes each day.

As was common throughout the transit industry in North America and Western Europe after the Second World War, plans in Toronto called for the elimination of streetcars and their replacement by new subways, buses, or electric trolley buses. These plans were decisively reversed in 1972, largely as a result of citizen support for continued streetcar service. As a result, older streetcars were rebuilt, new streetcars were purchased, other infrastructure upgrades were made, and between 1990 and 2000, three significant additions were made to the streetcar route network. Major investment in renewed track and overhead wiring continues today as part of the TTC State of Good Repair budget.

Much of the development of the older parts of Toronto was strongly influenced by the presence of frequent, reliable transit service provided by streetcar routes. Residential and commercial neighborhoods that are today on well-used subway lines – Bloor West, North Toronto, or The Danforth, for example – were developed because of the streetcar service that was introduced to these areas in the first few decades of the 20th century. Other vibrant neighborhoods that remain on busy, highly-traveled streetcar lines – Little Italy on College Street, St Clair Avenue, The Beach – continue to be popular and successful places to live and shop. For significant new neighborhoods planned to be developed in future years, such as the East Bayfront and the West Don Lands, it is expected that only new streetcar routes can provide the necessary high-quality, high-capacity, and reliable public transport that will be necessary to support the growth of these new communities.

In addition to being an important part of Toronto heritage, streetcars offer a number of important customer service, community, and environmental benefits. They provide a notably smooth, quiet, and comfortable ride. Experience with the three new streetcar services introduced in the 1990s shows that ridership increases when bus services are replaced by new streetcar service. Municipal investment in streetcars demonstrates the city's long-term commitment to an area which, in turn, instills confidence and brings about higher levels of private investment and development. Streetcars are environmentally-friendly, producing approximately 93% fewer emissions than buses, on a per passenger-kilometre basis, even after taking into account the off-site generation of electricity required for streetcars expansion.

Streetcars continue to play a very important role in the TTC transit network. While only 11 of the TTC 149 surface routes are operated with streetcars, these routes carry 17% of all passengers travelling on the TTC surface network, or about 43 million passengers annually; these eleven streetcar (tram) routes total 305.8 km in length. Generally concentrated downtown and in proximity to the city's waterfront, much of Toronto streetcar network dates back to the 19th century. Unlike newer light rail systems, therefore, most of Toronto streetcar routes operate in the classic style on street trackage shared with car traffic. However, some routes operate (totally or partially) within their own right-of-ways. There are underground connections between streetcars and the subway at Union, Spadina, and St. Clair West stations and streetcars pass by the entrances of several other downtown stations, as depicted in Figure 1-1.

Figure 1-1 Toronto Streetcar on Queen Street

In addition to being an important part of Toronto heritage, streetcars offer a number of important customer service, community, and environmental benefits. They provide a notably smooth, quiet, and comfortable ride. Experience with the three new streetcar services introduced in the 1990s shows that ridership increases when bus services are replaced by new streetcar service. Municipal investment in streetcars demonstrates the city's long-term commitment to an area which, in turn, instills confidence and brings about higher levels of private investment and development. Streetcars are environmentally-friendly, producing approximately 93% fewer emissions than buses, on a per passenger-kilometre basis, even after taking into account the off-site generation of electricity required for streetcars expansion.

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Figure 1-1 Toronto Streetcar on Queen Street

1.4.1 Routes

The current TTC streetcar network is shown in Figure 1-2. In relation to the subway (coloured in dark blue); all eleven regular routes appear in light blue.

Figure 1-2 Toronto Streetcars Route

1.4.2 Current and Future Streetcar Requirements

There are a number of factors, some of which are quite recent initiatives, which will cause the number of streetcars required for service on the TTC system to increase in the future. These are:

Additional ridership due to population and employment growth: Toronto new official plan calls for an increase in population and employment of up to 750,000 people over the next 20 years. The Plan also calls for measures which will encourage more people to travel by transit, rather than by automobile. This growth in population, and policies aimed at changing people's travel behaviour, are expected to increase travel demand on streetcars.

Ridership Growth Strategy (RGS): This plan, endorsed by the Commission in March, 2003, calls for, among other things, improvements to peak-period service on major arterial routes in Toronto. These improvements will require more streetcars to be put into service.

Combating deteriorating traffic conditions: The majority of the TTC streetcar routes currently operate in mixed-traffic, sharing the roadway with private automobiles, trucks, and other users of the road. The TTC experience has been that, as traffic volumes grow and traffic operating speeds decline, more streetcars are required in service simply to offset this slowing-down of transit operating speeds. This trend will require an increase in the TTC streetcar fleet.

Development of Toronto Waterfront: One of Toronto major planning initiatives is the opening-up and redevelopment of its waterfront for a wide range of residential, employment, and recreational uses. In approving land-use plans for the waterfront, City Council also approved a “Transit First” policy which requires that high-capacity transit service be in place before development proceeds in the waterfront. This major initiative and policy position will likely require an increase in the TTC streetcar fleet.

Building a Transit City (BTC): In support of Toronto new Official Plan, City Planning, City Transportation, and the TTC have been working together to develop a new transportation strategy for the City of Toronto which would introduce high-quality, high-capacity transit services, in dedicated rights-of-way, on a network of major arterial roads and utility corridors, in order to significantly enhance transit service across the entire city. Work is ongoing regarding the choice of streetcars or buses for the various corridors which constitute this network, but it is certain that implementation of the BTC network will require an increase in the TTC streetcar fleet.

All of these factors, combined, will require a significant increase in the size of the TTC streetcar fleet.

1.4.3 Vehicles - The CLRVs and ALRVs

When the TTC reversed their decision to eliminate streetcars in the 1970s, they were faced with the problem of how to replace their ageing fleet of PCC streetcars given that most cities in North America were switching entirely to buses, and so there were no new mass-market streetcar designs already being built that Toronto could purchase as it had before. While Edmonton and Calgary chose to adapt German metro trains for the new

systems they were installing around the same time, the TTC instead had a new streetcar designed in the traditional style, and so the two models of streetcars the TTC uses for revenue service today remain unique to the city. It was hoped that the new models could also be sold to the few other cities that continued streetcar service, such as Boston and Philadelphia, but this strategy proved unsuccessful.

Figure 1-3 shows the CLRV (Canadian Light Rail Vehicle, ordered 1977 - version L1 and L2) and the double-length ALRV (Articulated Light Rail Vehicle, ordered 1983 - version L3) were designed by Ontario Crown Corporation Urban Transportation Development Corporation (UTDC). They are built by Hawker-Siddeley Canada Limited in Thunder Bay, with a propulsion system by Brush of England and bogies by MAN of Germany.

Figure 1-3 CLRV Streetcar, used on most of the TTC Streetcar Routes

The CLRVs and ALRVs retain many features of traditional streetcar design: they collect their electric power by trolley pole rather than pantograph, and are unidirectional, with a driving position at only one end and doors on only one side, requiring track loops in order to turn around. Even the ALRVs, which are double-length with an articulated section at the centre, are not very long compared to some other modern trams, which may have as many as four articulations. Unlike the TTC earlier PCC and Peter Witt streetcars, the current models are never run in two-unit trains, or with trailers; the replacement of the two highest-volume routes with subway lines has decreased the number of passengers streetcars must cope with, and a single ALRV is long enough to provide sufficient capacity on today's busiest routes

1.5 Technical Background

In this study, streetcar track elements have been categorized by function. Clarifying the purpose for each component is intended to assist in the analysis of alternatives, as well as further evaluating their performance under fatigue loading experimental program. The track element categories are:

Track support elements, i.e., elements of the track structure that provide overall support. Track gauge and alignment control elements, i.e., elements of track structure (temporary or permanent) that hold the rail in place. Rail support elements, i.e., the material directly under the rail that influences ride quality, rail corrugation and wear, and noise and vibration characteristics of the rail. Rail encasement, i.e., the material in contact with the rail web and rail head, usually intended as filler but which can also influence the electrical and dynamic isolation of the rail. These elements are shown schematically in Figure 1-4. The concept of the "elements" reveals a distinction between track elements and their function that is not necessarily obvious when examining the current construction method.

Figure 1-4 Cross Section of Guarded Track

1.5.1 Rail Material

The principal materials used in mechanically jointed special trackwork are manganese steel for castings, rail steels for closure rails and carbon steel for the joint bars; the chemical composition conforms to the following AREMA and CSA Standards:

1. Manganese steel shall be to AREMA specifications for Special Trackwork, Plan 100-96, and Section M2.
2. Rail Steel shall be to AREMA specifications for 3 HB and HH (Head hardened) Steel
3. Carbon Steel components shall be to CSA G40.21, Type 44W steel.

1.5.2 Track Gauge

The tracks of Toronto streetcars and subways (apart from the Scarborough RT) are built to the unique track gauge of 1.495 m (4 feet 10⁷/₈ inches), slightly wider than the usual standard gauge of 1.435 m (4 feet 8 ½ inches). There are arguments over the reason for this. The more practical reason is that early tracks were used to pull wagons smoothly in the days before paved roads, and the wagon axles required this wider gauge. The use of standard-gauge tracks on the Scarborough RT makes it impossible for there to be any track connection between it and the other lines, and so when RT vehicles need anything more than basic service, they are carried by truck to the Greenwood subway yard.

1.5.3 Track Structure

Among its various functions, the track system serves to distribute the large, concentrated wheel loads longitudinally, laterally, and vertically away from the wheel contact area on the rail surface. A well designed, constructed, and maintained track will distribute the loads in a relatively uniform fashion, with each component supporting its share of the load. Therefore, the intent is that the wheel loads to be properly supported, without overstressing any of the four main track system components: rail, ties, concrete pavement, and subgrade. It is important to note that while track construction is relatively simple, its behavior under load is not. An improvement (or increase in strength) in one track component may cause an increased load on another.

For typical tie and ballast track, when a wheel is centered over a tie, the tie directly beneath the load will generally carry less than half of that wheel load, with the remainder supported by the nearest two ties on either side of the wheel location. Beneath the wheel, the pressures are distributed. With a wheel-rail contact area of about 0.5 in², stresses are reduced from 15 ksi at the top of the rail to about 10 psi at the depth of the subgrade surface. While the actual load and pressure distributions will vary with wheel load, track design, and track condition. The main requirements of a TTC streetcar concrete pavement include the following:

- Resistant to wheel impact load,
- High wear resistance of components; (i.e., low life cycle cost)
- Low level of vibration
- Economic construction
- Minimization of disruption to road vehicular traffic during construction or replacement

1.5.4 Existing TTC LRT (Streetcar) In-Street Track

There are a number of means of constructing embedded track. The traditional method of installing in-street track involved simply constructing standard tie and ballast railway-type track and filling the space from the top of rail with suitable materials. That is, standard rails on timber ties were set to line and surface in ballast and the entire construction was buried in concrete, asphalt or topped with granite sets in sand. Historically it was found that the granular ballast base was not sufficiently stable to withstand heavier road vehicle wheel loads.

The current TTC standard in-street track construction, illustrated in Figure 1-4, consists of a 230 mm (9.0”) thick, un-reinforced concrete slab above which is installed track consisting of standard tee rail and/or girder guard rail, joined by means of electric flash butt or thermite welds to eliminate joints. Rails on tangent track and in curves are supported on rolled W 6x15 I-beams that are cut to a length of 2.06 m with press shoulders welded on top flange to achieve correct gauge, as shown in Figure 1-5.

Figure 1-5 Typical Cross Section of a tangent T-rail

Track gauge is $4'-10\frac{7}{8}"$ except for curves of less than 1000 ft where it is widened to $4'-11"$. Rails are clipped to coated steel ties at 42.0" centers (reduced to 36.0" on curves to 1000 ft; and 24.0" on curves less than 1000 ft).

The skeleton track is shimmed to establish road grade with the top of rail approximately 350 mm (14.0") above top of base slab (i.e. 580 mm (23.0") from base of slab to top of rail). A second pour of concrete is placed above the base slab to approximately the top of ties. A third pour of concrete is placed from top of tie to top of rail. This differs from early TTC LRT construction methods which used wood ties and cut spikes and sometimes omitted the base slab. Also, the remaining concrete was sometimes placed in a single pour from below bottom of tie to top of rail. Such methods were found to be unsatisfactory from a track durability point of view.

For new construction, on curved track with a radius between 305 m (1000') and 30.5 m (100') girder guard rails, designated as NP4aM are used on the inside rail and head hardened tee rail (115 RE) on the outside rail. Concrete is placed to top of rail with transverse control joints at every cross tie (i.e. 1.8 m (6.0')). The flangeways for tee rail construction are formed in the last concrete pour and caulked adjacent to the rail.

The current method of track construction assumes that all surface water can be directed away from the track. There are no provisions for draining the water that inadvertently reaches the interior of the track structure, nor for preventing moisture migrating up from below the sub-base.

1.5.5 Existing TTC LRT In-Street Turnouts

All turnouts on the TTC streetcar system are a tongue-and-mate-in-street design. There are approximately 350 of these embedded turnouts, excluding shops and yards, with approximately 100 of the facing point turnouts equipped for automatic operation. Switch tongues are generally 12' or 15' in length. Frogs are cast manganese flange-running construction. There are a number of standard frog angles to which embedded track is designed whenever possible.

In-street turnouts, junctions and unions are constructed on shimmed, creosoted timber switch ties, but with flat tie plates, and with gauge rods. Special trackwork are constructed on a concrete base slab, with concrete infill beside and between the ties and rails as described for standard track.

Chapter 2

Fatigue in Concrete

2.1 General

Many structures are often subject to repetitive cyclic loads. Examples of such cyclic loads include machine vibration, sea waves, wind action and automobile traffic. The exposure to repeated loading results in a steady decrease in the stiffness of the structure, which may eventually lead to fatigue failure. In concrete, these changes are mainly associated with the progressive growth of internal microcracks, which results in a significant increase of irrecoverable strain. At the macrolevel, this will manifest itself as changes in the material mechanical properties.

Although concrete is a widely used construction material, the understanding of fatigue failure in cementitious composites is still lacking compared to the ferrous materials. This incomplete understanding is even more pronounced for composite materials such as FRC. The earliest research on fatigue properties of concrete materials is traced back to the end of the 19th century, which is significant for concrete structures (such as bridges, crane beams, hydraulic foundations, pressure vessels etc.) subjected to long-term cyclic loading (Lu et al, 2004)

In recent years, considerable interest has developed in the fatigue strength of concrete members. There are several reasons why fatigue consideration is important. First, the widespread adoption of ultimate strength design procedures and the use of higher strength materials require that structural concrete members perform satisfactorily under high stress levels. Hence there is concern about the effects of repeated loads for example crane beams and bridge slabs.

Second, new or different uses are being made of concrete members or systems, such as prestressed concrete railroad ties and continuously reinforced concrete pavements. These uses of concrete demand a high performance product with an assured fatigue strength. Third, there is new recognition of the effects of repeated loading on a member, even if repeated loading does not cause a fatigue failure. Repeated loading may lead to inclined cracking in prestressed beams at lower than expected loads, or repeated loading may cause cracking in component materials of a member that alters the static load carrying characteristics (ACI 215).

The intent of this chapter is to present the fatigue characteristics of concrete in the form of diagrams and algebraic relationships, as related to direction of experimental program. This information is from the results of tests conducted on different types of specimens subjected to various loading conditions, presented in a range of papers, journal articles, and text books. However, a comprehensive attention was given in the course of selecting the materials and further setting up the associated experimental program and performing analytical justifications. The characteristic and performance of reinforced and FRP concrete are briefly discussed as well.

2.2 Fatigue Concept

First, it is important to carefully distinguish between static, dynamic, fatigue, and impact loadings. Static or sustained loading, remains constant with time. A load which increases slowly is often called static loading; the maximum load capacity under such conditions is referred to as static strength. Dynamic loading varies with time in any arbitrary manner. Fatigue and impact loadings are special cases of dynamic loading. Fatigue may be defined as a process of progressive, permanent internal structural changes in a material subjected to the repeated loading (Lee and Barr, 2004). Under the fatigue state, the material has a cyclic loading nature and is subjected to a repeated stress loading condition, in which the stress varies between zero and a maximum tensile/compressive stress in a cyclic manner. These cyclic loading are commonly in one of the three forms; periodic and symmetrical about zero; periodic and asymmetrical about zero; and random stress fluctuation (See Appendix A).

Cyclic stresses are further characterized by maximum, minimum and mean stress, the range of stress, the stress amplitude, and the stress ratio (Varvani, 2005; Gangloff, 2005). Various approaches have been used in the fatigue life assessment of structural elements. A widely accepted approach for engineering practice is based on empirically derived S–N diagrams, also known as Whöler curves, shown in Figure 2-1.

This is a plot of repeated stress, (expressed as stress ratio, or percentage of the static strength), versus the logarithm of the number of cycles to cause failure, N .

Figure 2-1 Typical S-N Curve (Gangloff, 2005)

Figure 2-2 Typical Fatigue Limit Curve (Gangloff, 2005)

Fatigue (endurance limit) occurs for some materials. In this case, as shown in Figure 2-2, the S—N curve becomes horizontal at large N. The fatigue limit is a maximum stress amplitude below which the material never fails, no matter how large the number of cycles is.

2.3 Fatigue in Concrete

Based on ACI 215 fatigue on concrete is a process of progressive permanent internal structural change as concrete is subjected to repetitive stresses. These changes may be damaging and result in progressive growth of cracks and complete fracture if the stress repetitions are sufficiently large. Fatigue fracture of concrete is characterized by considerably larger strains and microcracking as compared to fracture of concrete under static loading.

Fatigue strength of concrete for a life of ten million cycles-for compression, tension, or flexure-is roughly about 55 % of static strength. In stress range, a fatigue loading consists of a sequence of load repetitions that may cause a fatigue failure in about 100 or more cycles (ACI 215).

Compressive, tensile and flexural bending fatigue are common in concrete structures; however, based on the loading conditions, a combination of above or other stresses can occur.

Similar to other materials, fatigue loading in concrete is usually divided into two categories (RILEM, 1984) i.e. low-cycle and high-cycle loading. Low-cycle loading involves the application of a few load cycles at high stress levels. On the other hand, high cyclic loading is characterized by a large number of cycles at lower stress levels. Very high level repeated loadings due to earthquakes or other catastrophic events may cause failures in less than 100 cycles, which are referred to as low-cycle fatigue. Hsu et al (1981) provides a wider range of fatigue load spectrum with the inclusion of super-high cycle loading. Table 2-1 summarizes the different classes of fatigue loading, based on his classification.

Table 2-1 Classes of Fatigue Load (Hsu, 1981)

<u>Low Cycle Fatigue</u>			<u>High Cycle Fatigue</u>			<u>Super High Cycle Fatigue</u>			
<u>1</u>	<u>10¹</u>	<u>10²</u>	<u>10³</u>	<u>10⁴</u>	<u>10⁵</u>	<u>10⁶</u>	<u>10⁷</u>	<u>10⁸</u>	<u>10⁹</u>
Structures subjected to Earthquake			Airport pavement and bridges		Highway and Railway bridges, highway pavements		Mass Rapid Transit Structures		Sea structure

As a matter of fact, concrete is a composite material and its performance under fatigue loading is widely related to the properties of component materials such as reinforcing bars, prestressing tendons and FRP composites. Majority of the models proposed to describe the cyclic behaviour of concrete are empirical and based on damage mechanics, which can be interpreted as decay in mechanical properties with increasing stress and/or strain. As well, due to testing procedure difference, not all the findings are always appropriate to apply even in similar conditions.

In the following sections properties of plain, reinforced and FRP concrete will be discussed separately. However, the focus will be given to the fatigue in plain concrete which stands as the main core of this report.

2.3.1 Fatigue of Plain Concrete

Concrete is a heterogeneous material which is inherently full of flaws (such as pores, air voids, lenses of bleed water under coarse aggregates and shrinkage cracks). Plain concrete, when subjected to repeated loads, may exhibit excessive cracking and may eventually fail after a sufficient number of load repetitions, even if the maximum load is less than the static strength of a similar specimen. The fatigue strength of concrete is defined as a fraction of the static strength that it can support repeatedly for a given number of cycles (ACI 215).

2.3.1.1 Mechanism of Fatigue Crack Growth

The mechanism of fatigue failure in concrete or mortar can be divided into three distinct stages (Gangloff, 2005; Shah, 1995; Gao, 1998).

1. Crack initiation: this stage involves the weak regions within the concrete or mortar and is termed flaw initiation. Crack initiation is observed at the sites of stress concentration such as notches, microcracks, aggregate/paste bond, interfacial transition zone, ITZ
2. Incremental crack propagation in the fracture process zone, developing in macro and micro level as per Figures 2-3 and 2-4, consists of two stages:

Figure 2-3 Fatigue Fracture Growth in Concrete (Shah, 1995)

Stage I: initial slow and progressive growth of the inherent flaws to a critical size, generally known as microcracking.

Stage II: faster propagation perpendicular to the applied stress; accompanied with crack growth by a repetitive blunting and sharpening process at crack tip.

3. Final rapid crack propagation after crack reaches critical size. In the final stage, a sufficient number of unstable cracks have been formed, causing a continuous or macrocrack development, eventually leading to failure.

The total number of cycles to failure is the sum of cycles at the first and second stages (Gangloff, 2005):

$$N_f = N_i + N_p \quad (2-1)$$

N_f : Number of cycles to failure

N_i : Number of cycles for crack initiation

N_p : Number of cycles for crack propagation

For high cycle fatigue (low loads), N_i is relatively high; with increasing stress level, N_i decreases and N_p dominates.

Figure 2-4 Features of Fatigue Crack Growth Process (Toumin & Bascoulz, 2002)

Horii et al. (1992) provided another approach, in which, fatigue crack growth can be divided into two distinct stages; the first stage is a deceleration stage: where the rate of crack growth decreases as the crack grows; and the second stage is an acceleration stage, where there is a steady increase in the crack growth rate right up to failure (Kolluru et al., 2000).

A more governing and general approach has been taken by Hsu et al (1984) suggesting that different loading cycle regimes produce different failure mechanisms within concrete. For low-cycle fatigue, the dominant mechanism is the formation of mortar

cracks leading to continuous cracked networks. On the other hand, high-cycle fatigue produces bond cracks in a slow and gradual process. It can be interpreted that this approach is more representing the concrete composite structure and can justify any of foregoing mechanisms.

2.3.1.2 Fatigue Strength

Fatigue strength is influenced by rate of loading, load history, size, material properties, stress gradient, and concrete strain. Fatigue data for concrete are usually presented graphically by an S-N diagram. S-N curves are plotted using strength (or stress) values, which are made dimensionless by relating them to the static strength. The dimensionless term, S, in part, eliminates influences such as specimen shape, w/c ratio, type and grading of aggregates, concrete strength, curing and moisture conditions, and age at loading etc.

Lee and Barr (2004) plotted some of previous data presented in the literatures, hoping to identify some general trends. Figures 2-5 and 2-6 show plots of compressive and flexural fatigue tests, respectively, based on various researchers carried out on plain concrete.

Figure 2-5 S-N Curve for Plain Concrete under Compression (Lee and Barr, 2004)

Figure 2-6 S-N Curve for Plain Concrete under Flexure (Lee and Barr, 2004)

Cyclic tensile loading, as shown in Figure 2-7, has followed the same fashion.

Figure 2-7 S-N Curve for tensile fatigue, $S_{min} = 0.15$ (Lu et al., 2004)

A similar approach has been taken by many other researchers resulted in analogous patterns, regardless of the nature of cyclic stresses. This is due to heterogeneous composition of concrete; additionally, the interpretation of fatigue test results is complicated by scatter.

Another approach in assessing the fatigue life of concrete, which is also recommended by ACI, utilizes S-N-P diagrams. McCall (1958) first suggested that fatigue test data should be handled statistically, and the need to replace the conventional S-N curve, relating the level of repeated stress and fatigue life, with the concept of a S-N-P relationship, thereby introducing the probability of failure as an additional variable, as shown in Figure 2-8 (Raju, 1969).

This approach has been employed by ACI 215 as shown in Figure 2-9, where the ordinate is the ratio of the maximum stress, $S_{\max}$ to the static strength. In this case, $S_{\max}$ is the computed flexural tensile stress, and the static strength is the modulus of rupture, f_r ; the abscissa is the number of cycles to failure, plotted on a logarithmic scale.

Figure 2-8 Frequency Distribution of Fatigue Life (Raju, 1958)

Figure 2-9 Fatigue Strength of Plain Concrete Beams (ACI 215)

In fact, a single S-N curve is inadequate to describe fatigue behaviour; therefore, it is also necessary to take into account the large amount of scatter inherent in fatigue tests of concrete. Figure 2-10 demonstrates this concept in a simple form, where the usual S-N curve is that shown for $p = 0.5$.

Figure 2-10 Incorporation of Probability into S-N Curve (Sidney Mindess, 2002)

2.3.1.3 Concrete Fatigue Limit

Unlike ferrous metals and regardless of analytical approach, concrete does not appear to have a fatigue limit. It has been reported that plain concrete subjected to repeated uniaxial tensile stresses exhibits no fatigue limit under 10^7 cycles (Sidney Mindess, 2002). On the

other hand, ACI 215 indicates that concrete does not exhibit an endurance limit up to 10 million cycles and in other words, there is no limiting value of stress below which the fatigue life will be infinite as depicted in the Figure 2-11.

Figure 2-11 Typical Form of No Fatigue Failure Zone (Castillo & Canteli, 2001)

2.3.1.4 Load History

According to ACI 215, effects of different values of maximum stress can be approximately estimated, although not always conservatively, from constant stress fatigue tests by using the Miner hypothesis. According to this rule, failure occurs if:

$$\sum (n_i/N_i) = 1 \quad (2-2)$$

Where n_i is the number of cycles applied at a particular stress condition, and N_i is the number of cycles which will cause fatigue failure at that same stress condition.

The effect of rest periods and sustained loading on the fatigue behaviour of concrete is not sufficiently explored. Laboratory tests have shown that rest periods and sustained loading between repeated load cycles tends to increase the fatigue strength of concrete. In these tests, the specimens were subjected to relatively low levels of sustained stress.

If the sustained stress level is above about 75% of the static strength, then sustained loading may have detrimental effects on fatigue life. This contradictory effect of creep loading may be explained from test results which show that low levels of sustained stress increase the static strength, whereas high levels of sustained stress resulted in increased microcracking and failure in some cases.

2.3.1.5 Rate of Loading

Several investigations indicate that variations of the frequency of loading between 70 and 900 cycles per minute have little effect on fatigue strength provided the maximum stress level is less than about 75% of the static strength. For higher stress levels, a significant influence of rate of loading has been observed (ACI 215). Under such conditions, creep effects become more important, leading to a reduction in fatigue strength with decreasing rate of loading.

The influence of load range can be seen from comparison of Curves “a” and “c” in the Figure 2-9. Curves “a” and “c” indicate that the fatigue strength of concrete decreases with increasing number of cycles. It may be observed that the S-N curves for concrete are approximately linear between 10^2 and 10^7 cycles. The curves were obtained from tests with loads ranging between a maximum and a minimum which was equal to 75% and 15% of the maximum strength, respectively. It is evident that a decrease of the range between maximum and minimum load results in increased fatigue strength for a given number of cycles. When the minimum and maximum loads are equal, the strength of the specimen corresponds to the static strength of concrete determined under otherwise similar conditions.

2.3.1.6 Stress Gradient

Concrete is very sensitive to the range of the cyclic stress (i.e. the difference between the highest and lowest stresses). Stress gradient has been shown to influence the fatigue strength of concrete. From the mean S-N curves shown in Figure 2-12, it can be seen that the fatigue strength of eccentric specimens is 15 to 18% higher than that for uniformly stressed specimens for a fatigue life of 40,000 to 1,000,000 cycles. These are in accord

with results of static tests showing that strain gradient retards internal microcrack growth (ACI 215).

Figure 2-12 Influence of Stress Gradient (ACI 215)

Figure 2-13 Effect of Repeated Load on Concrete Strain (ACI 215)

2.3.1.7 Concrete Strain

Similar to the behaviour of concrete under sustained loads, the strain of concrete during repeated loading increases substantially beyond the value observed after the first load application, as shown in Figure 2-13. The strain at fatigue failure is likely to be higher if the maximum stress is lower (ACI 215).

2.3.1.8 Size Effects

Strength of materials is influenced by the specimen or structure size. In particular, several experimental campaigns have shown a decrease of the material strength under static or fatigue loading with increasing structural size (Carpinteri, 2004). The nominal strength of a given material changes by scaling the size. Consequently, strength ceases to be a material constant, and such a phenomenon is called size effect. Size effects on both strength and fracture toughness of structural materials have been experimentally observed. In particular, tensile strength and fatigue strength decrease by increasing the specimen or structure size. Griffith (1921) explained the tensile strength decrease for glass filaments by assuming the presence of microcracks whose size is proportional to the diameter of filament cross-section, shown in Figure 2-14; Peterson (1933) observed size effect for brittle fracture under fatigue loading.

Figure 2-14 Size Effect in Fatigue Life of Concrete (Carpinteri, 2004)

2.3.1.9 Material Properties and Mix Design

Concrete under repeated loading sustains a gradual fatigue damage. At the macro level, strength and Young's modulus decrease with increasing loading cycles; at the micro-meso level, internal structures degenerate and micro-defects (pores, cracks, etc.) develop.

Pore structure is an important micro-meso property which determines macro properties of concrete such as strength, elasticity, shrinkage, durability, etc. Cement paste, mortar, and concrete are all porous materials. Any change in pore structure may lead to sharp changes in performance. Furthermore, the fatigue strength of mortar and concrete are about the same when expressed as a percentage of their corresponding ultimate static strength. Many variables such as cement content, w/c ratio, curing conditions, age at loading, amount of entrained air, and type of aggregates that affect static ultimate strength also influence fatigue strength in a similar manner (ACI 215).

In contrast, Sidney Mindess (2002) stated that at stresses below about 75% of the static strength, not only does the strength appear to be independent of the type of the type of loading, it is also seems to be independent of the details the mix design. Cement, mortar, and concrete seem to have approximately the same S-N diagrams, which do not change significantly for different ages, w/c ratios, aggregates, air entrainment, etc. At stresses above, 75% of the static strength (representing low cycle fatigue), cement paste appears to have a longer fatigue life than does mortar or concrete with the same w/c ratio.

2.3.1.10 Effect of High Compressive Strength and Silica Fume

There is a significant amount of conflicting information in the literatures regarding the fatigue behaviour of concrete when it comes to higher compressive strength, as well as using silica fume. Yan and Chen (1999) studied the impact and fatigue performance of high-strength concrete (HSC), silica fume high-strength concrete (SIFUHSC), steel fibre high strength concrete (SFRHSC), and steel fibre silica fume high-strength concrete (SSFHSC) under the action of repeated dynamic loading. As is seen in Figure 2-15, their results indicated that steel fibre effectively restrained the initiation and propagation of

cracks during the failure of an HSC structure, mitigated the stress concentrations at the tips of cracks, and delayed the damage process under impact and fatigue.

Figure 2-15 The Impact of SF in Concrete Fatigue (Yan & Chen 1999)

On the contrary, Kim et al. (1996) as per Figure 2-16, suggested that the fatigue life of concrete decreases with increasing compressive strength, and consequently the fatigue strength of HSC is smaller than of LSC.

Figure 2-16 Effect of f'_c on Concrete Fatigue (Kim et al., 1996)

It can be interpreted that the latter seems to be more reasonable as with the increase in compressive strength, as well as with the application of silica fume, the concrete tends to demonstrate a more brittle behavior and consequently becomes more vulnerable under fatigue loading.

2.3.1.11 Fatigue Design

Design for fatigue may be facilitated by use of a modified Goodman diagram, as illustrated in Figure 2-17. This diagram is based on the observation that the fatigue strength of plain concrete is essentially the same whether the mode of loading is tension, compression, or flexure.

Figure 2-17 Fatigue Strength of Plain Concrete in Tension, Compression or Flexure (ACI 215)

The diagram also incorporates the influence of range of loading. For a zero minimum stress level, the maximum stress level the concrete can support for one million cycles without failure is taken conservatively as 50% of the static strength. As the minimum stress level is increased, the stress range that the concrete can support decreases. The linear decrease of stress range with increasing minimum stress has been observed, at least approximately, by many investigators. From Figure 2-17, the maximum stress in tension, compression, or flexure that concrete can withstand for one million repetitions and for a given minimum stress can be determined.

For example, to design a structural element to for one million cycles, if the minimum stress is 15% of the static ultimate strength, then the maximum load that will cause fatigue failure is about 57% of static ultimate load.

2.3.1.12 Fatigue Equation

Another formula to quantify the fatigue load was offered by Zheng & Wu (1997). Three aggregates (slag, gravel and ceramsite) to form heavy, ordinary and light weight concrete to investigate the effects of various parameters (loading frequency, stress reversal w/c ratio, aggregate type, etc.) on the fatigue properties of concrete. Based on the results, a fatigue equation to predict the fatigue life of concrete, considering load frequency (rate) effect and other factors was proposed as follows:

$$\frac{S_{\max}}{f_c} = c_f [1 - (1 - R')^{\beta \log(N - N_1)}] \quad (2-3)$$

Where f_c is the static strength; β is a material parameter which is equal to 0.0807 for the heavy and ordinary concrete, and 0.0700 for light weight concrete.

c_f is the loading frequency coefficient, defined as:

$$c_f = ab^{-\log f} + c \quad (2-4)$$

Where f is the loading frequency in Hz, and a , b and c are material properties as:

$a = 0.249$, $b = 0.920$, $c = 0.796$ and R' is the specialized stress ratio:

$$R' = \left(\frac{S_{\min}}{f_{c\min}} \right) / \left(\frac{S_{\max}}{f_{c\max}} \right) \quad (2-5)$$

$S_{\min}$ and $S_{\max}$ are the minimum and maximum stresses; $f_{c\min}$ and $f_{c\max}$ are the static strength corresponding to the loading states of $S_{\min}$ and $S_{\max}$, respectively.

Actually, C_f stands for the strengths varying with the loading frequency or rate, $C_f = f_{cf} / f_c$

Where f_{cf} is the static strength at f ; so C_f can also be called the relative strength.

C_f decreases with decreasing f , especially when f is near to zero or the loading time becomes infinite, C_f (near to c) becomes the relative long term strength of concrete.

The residual fatigue strength can be defined as the static strength, or the relative static strength (the strength ratio) for a known loading cycles. It is a very important parameter for assessing the fatigue damage state of existing concrete structures so that maintenance and repair can be done if necessary. A new formula for predicting the residual fatigue strength can be derived from residual.

After N_1 cycle applied onto concrete at S_{max} , the residual strength is f_{cr} and the residual life is $N - N_1$, then during $N - N_1$, the concrete still obeys residual as:

$$\frac{S_{max}}{f_{cr}} = C_f [1 - (1 - R')^{\beta \log (N - N_1)}] \quad (2-6)$$

Solving the equations (2-3) and (2-6), the relative residual fatigue strength f_{cr} is obtained as:

$$\bar{f}_{cr} = \frac{f_{cr}}{f_c} = \frac{1 - (1 - R')^{\beta \log N}}{1 - (1 - R')^{\beta \log (N - N_1)}} \quad (2-7)$$

2.3.2 Fatigue of Reinforced Concrete

It is believed that the degradation of the bond between rebar and concrete, under cyclic shear loading can be determined, non-destructively by measuring the contact electrical resistance of the joint. Degradation, which caused a decrease in bond strength but no visual damage, was indicated by an abrupt increase in the resistance at a small fraction of the fatigue life. (Cao, 2001)

2.3.2.1 Reinforcing Bars

Fatigue of rebars has not been a significant factor in their application as reinforcement in concrete structures. However, more concern shall be given due to the tendency of using ultimate strength design and higher yield strength. It is noteworthy, that the lowest stress range known to have caused a fatigue failure of a straight hot-rolled deformed bar embedded in a concrete beam is 21 ksi (145 MPa). This failure occurred after 1,250,000 cycles of loading on a beam containing a # 11, Grade 60 test bar, when the minimum stress level was 17.5 ksi (121 MPa).

A typical fatigue fracture of a reinforcing bar is shown in Figure 2-18. This is a #11, Grade 60 bar which at one time was embedded in a concrete beam that was subjected to repeated loads until the bar failed. In this figure, the orientation of the bar is the same as it was in the beam; the bottom of the bar was adjacent to the extreme tensile fibres in the beam. The smoother zone, with the dull, rubbed appearance, is the fatigue crack. The remaining zone of more jagged surface texture is the part that finally fractured in tension after the growing fatigue crack weakened the bar. It is noteworthy that the fatigue crack did not start from the bottom of the bar. Rather it started along the side of the bar, at the base of one of the transverse lugs. This is a common characteristic of most bar fatigue fractures.

2.3.2.2 Consistency of Tests Results

A large number of laboratory investigations of the fatigue strength of rebars have been conducted in the United States, Canada, Europe, and Japan. The rebars were either embedded in concrete or tested in air which brings up a contradiction in the technical literature as to whether a bar has the same fatigue strength when tested in air or embedded in a concrete beam. However, more recent studies indicate that there should be little difference in the fatigue strength of bars in air and embedded bars if the height and shape of the transverse lugs are adequate to provide good bond between the steel and concrete. This idea is further supported by (ACI 215).

Additionally, despite the differences in laboratory conditions for these tests, rates of loading up to several thousand cycles per minute and temperatures up to several hundred degrees C are normally not considered to have a significant effect on fatigue strength.

In a statistical analysis of an investigation of rebars, differences in fatigue strength due to rates of loading of 250 and 500 cycles per minute were not significant. Accordingly, unlike concrete fatigue results, ACI 215 suggested that most of the data reported in these investigations are directly comparable, even though it may have been obtained under quite different testing conditions.

Figure 2-19, shows a plot including a number of S-N curves obtained from tests on concrete beams containing straight deformed bars made in North America. These curves are for bars varying in size from #5 to #11, with minimum stress levels ranging from -0.10 to 0.43 of the tensile yield strength of the bars.

Figure 2-18 Fatigue Fracture of a Reinforcing Bar (ACI 215)

Figure 2-19 Stress Range-Fatigue Life Curves for Rebars (ACI 215)

2.3.3 Fatigue of Fibre Reinforced Concrete

Common applications for FRC include paving applications such as in airports, highways, bridge decks and industrial floors (Krstulovic, 1955), which endure significant cyclic loading during their service life. Within these areas of application, the fatigue characteristics of FRC are important performance and design parameters.

Generally, the FRP composites have proved to increase the stiffness and ultimate strength of the strengthened reinforced concrete structures. FRP is a composite material, and as a result its fatigue life is usually characterized by a combination of matrix cracking, ply delamination, fibre pullout and fibre breakage. The extent of improving the fatigue capacity of FRC can be expected to depend upon the fibre volume content, fibre type and geometry (Johnston, 1991; Grzybowski, 1993; Zhang, 1998).

Various combinations of these parameters will give rise to different fatigue characteristics. In concrete pavements, steel fibres are particularly suitable both for limiting shrinkage effects and for increasing their bearing capacity and fatigue resistance. The damage accumulation will degrade the stiffness and ultimate strength of the composite. The residual strength prediction of composite laminates after fatigue load have been addressed by several authors (Hahn, 1975; Yang, 1978; Reifsnider, 1986; Diao, 1995; Liao, 2004).

Mitrovic (1999) investigated the influence of loading parameters, such as stress amplitude and spectrum loading, on impact-induced delamination growth of the laminate. He suggested that the durability testing can be accelerated if one omits those applied stress levels less than 20% of the compression strength after impact. Whitworth (2000) proposed a residual strength degradation model based on the applied fatigue cycles and maximum applied stress.

In another comprehensive study, Lee and Barr (2004) evaluated the fatigue strength of plain and FRP concrete in compression and flexural bending. Based on this study, the presence of fibres does not seem to enhance the fatigue life of concrete under compressive fatigue loading. However, fibre addition does benefit the fatigue performance under flexural fatigue loading. A possible explanation is that under tensile forces, the fibres are able to bridge cracks and prolong fatigue life. On the contrary, the presence of fibres cannot display their true effectiveness under compressive loading, as the mode of failure is different, as shown in Figures 2-20 and 2-21.

Figure 2-20 S-N Curve for Concrete under Compressive & Flexure Stresses (Lee & Barr, 2004)

Figure 2-21 S-N Curves for FRC Concrete under Compressive & Flexure (Lee and Barr, 2004)

2.4 Chapter Summary

- When a material fails under a number of repeated loads, each smaller than the ultimate static strength, a fatigue failure has occurred. In concrete, these changes are mainly associated with the progressive growth of internal microcracks, which results in a significant increase of irrecoverable strain. At the macrolevel, this will manifest itself as changes in the material's mechanical properties.
- The lack of a well-established test procedure for executing and evaluating fatigue tests makes it difficult to correlate or extend published test results. A compilation of fatigue data from numerous sources has been carried out to observe the general trends, to select the appropriate parts and further to link them together.
- The majority of fatigue life prediction and design of plain, reinforced and FRC structures have been carried out empirically. These methods involve time consuming testing for a broad range of design cases, which in principle may be inapplicable to other design cases. Strictly speaking, fatigue data obtained from a particular test setup cannot be directly compared to data obtained from different loading configurations. Most laboratory fatigue data are idealized, since in these tests the loads alternated between constant minimum and maximum values. Concrete in structural members may be subjected to randomly varying loads. Currently, limited data are available showing the effect of random loading on fatigue behaviour of concrete, which seems to be a more.
- There is a significant amount of conflicting information in the literatures regarding fatigue behaviour of concrete. Dimensionless S–N curves are thought to represent the concrete behaviour subject to fatigue loading. Moreover, introducing the probability of failure, P , as recommended by ACI 215 and many others, provides a more accurate and true representation of the concrete fatigue strength.
- Degradation of the bond between steel rebar and concrete under cyclic loading is the main issue with reinforced concrete. Fatigue evolution in reinforced concrete

structures exhibit rather global character with multiple cracks due to redistribution of the loads through the rebars.

- It was found that the inclusion of fibres can benefit the fatigue performance of concrete, especially under flexural bending fatigue.

Chapter 3

Statement of the Problem

3.1 Status Quo

The average life of a TTC Streetcar concrete pavement is determined by many factors, including initial design details, material properties, construction methodology, traffic load and frequency, environment exposure and salt application, as well as the presence and effectiveness of protective systems, and changing maintenance practices, among others. All these factors influence the development of cracks in surface track structures during their service life. It has been well documented that cracking in slabs reduce the load capacity and is the ultimate cause of fatigue failure (Matsui 1997; Perdikaris & Beim, 1988; Perdikaris et al 1989; Kumar & GangaRao 1998). In addition, cracks allow water and other chemical agents, such as deicing salt, to penetrate the cover layer and come into contact with the rail and other metallic components, leading to corrosion and rupture.

In surface track structures, eventually, buried wood ties are exposed to ingress water and, over time, deteriorate, leaving voids with the concrete consequently weakening its structure. As it relates to durability, the TTC has long recognized the shortcoming of the existing special trackwork construction and have attempted to explore and evaluate alternative materials to enhance the service life of the track replacement programs, and reduce City road maintenance costs. To accomplish this objective, a number of innovative technologies were employed and variety of redesign and improvements were implemented. While these efforts have contributed to many improvements in track construction, replacement and maintenance, still there is a long road ahead.

The Toronto Transit Commission has a long term track replacement program for the overall streetcar network. Based on the current practice, the replacement interval varies

between 15 to 20 years for special trackwork, depending on the frequency of streetcar traffic, severity of loading and associated track conditions. This figure is almost identical to other transit authorities such as Boston; however, it is much short of desired 50 year service life for roads and streets as targeted by the City. From a financial point of view, special trackwork replacement costs from \$100,000 to \$2,000,000 per intersection, depending on the complicity of the geometry involved and the number of connections required to service the existing network.

3.2 Field Observations

Field observation suggested that the surface concrete is breaking up prematurely at many locations. A wide range of cracks were noticed at new and old construction works throughout the network, some as early as two years after construction. The primary understanding was that such cracks were related to the type of track construction and its contributing factors, i.e.:

1. Movement of the track structure due to resilience, wood decay or freeze-thaw action of wood track ties, promotes the cracking of surface concrete.
2. The ingress of water at lateral and longitudinal concrete control joints and subsequent freeze-thaw cycles further contribute to concrete deterioration.
3. The weakening of concrete from micro cracks formed by running LRT vehicles over concrete during the early stages of curing may cause premature cracking.

With the existing construction, the concrete pavement is intended to provide a solid support for the track that distributes streetcar and road vehicle wheel loads to the underlying soil. The track ties provide a means of maintaining track gauge, setting precise top of rail, and maintaining the track layout to the required uniform plane. Tie plates provide a rail-locating mechanism and reduce the stress transferred to wood surfaces. The concrete adjacent to and between the rails (continuation of second pour or a third pour) provides a running surface for roadway vehicles. On the other hand, excessive track stiffness has been identified as a primary cause of rail corrugation.

Comprehensive field observation, design review and construction control were conducted in this study to address the factors contributing to the life cycle of track, to identify the degradation and deterioration mechanisms, and to further implement remedial measures designed to enhance the service life of the system.

This was a major task, given the lack of robust information for jointed rail interaction when embedded in concrete and subjected streetcar traffic loads. Toronto Transit Commission assisted with this study and provided all required resources to assess alternative designs and construction technologies.

For this purpose, a thorough investigation was made and many measurements, inspections, and analyses were conducted to explore and identify the root cause of premature concrete failure. The summary of the findings are as followings:

- The life cycle of TTC streetcar track is governed by concrete degradation and failure as opposed to worn rails and fasteners. Track rails removed during demolition show the rails to be chemically intact (not corroded) and have limited mechanical damage, as can be seen in Figure 3-1.

Figure 3-1 Rails from Gerrard Street and Greenwood Track Replacement

- The most severe damaged concrete prior to replacement are at intersections, turnouts and basically in areas where special trackwork is used.

- At intersections, the majority of cracks are concentrated around mechanical joints. Some scattered cracking due to shrinkage was also observed.
- Inspection of new track revealed a similar pattern. It was noticed that inclined cracks at the joint appear as within the early as first two years of service life in some areas.
- A distinct repetitive wheel/rail contact noise was noticed when the streetcar traveled over mechanical joints.

These observations show that the inclined cracks initiate, primarily, around mechanical joints. As a matter of fact, it can be interpreted that the repeated movement of streetcar wheels are the key factor in initiation and progression of this damage mechanism. Figures 3-2 demonstrate this cracking.

a) 8 Years Old Concrete Pavement

b) 4 Years Old Concrete Pavement

c) 2 Years Old Concrete Pavement

Figure 3-2 Formation of Cracks at different Service Life Duration

The localized cracking appeared to be a pattern wherever mechanically jointed rails exist. Similarly, identical cracking was noticed in the surface tracks at Amsterdam light rail transit, despite the difference in track materials. Figures 3-3 demonstrate these cracks.

a) Plan of Inclined Cracking

b) Side View of Localized Inclined Cracking

Figure 3-3 Surface Track in Amsterdam, Netherlands

3.3 Joined Rails

The purpose of the rail joint (made up of two joint bars) is to hold the two ends of the rail in place; and act as a bridge or girder between the rail ends, as shown in Figure 3-4. The joint bars prevent lateral or vertical movement of the rail ends but permit the longitudinal movement of the rails from thermal effects.

Figure 3-4 A Schematic View of Mechanically Jointed Rail

The joint is considered to be the weakest part of the track structure and should be eliminated wherever possible. Joint bars are matched to the appropriate rail section. Each rail section has a designated drilling pattern (spacing of holes from the end of the rail as well as dimension above the base) that must be matched by the joint bars.

Although many sections utilize the same hole spacing and are closely matched in web height, it is essential that the correct bars are used so that curved areas of the web and rail head radii are matched. Failure to do so will result in an inadequately supported joint, and will promote rail defects such as head and web separations and bolt hole breaks.

3.4 Susceptibility of Mechanical Joints in Fatigue

Figure 3-5 shows a typical rail and casting joint in the early stage of construction. It can be interpreted that all three fatigue components of load, geometry and material come to play. The rail is made of rolled steel while the casting is of manganese, as previously described in Chapter 1. The difference in geometry is obvious from Figure 3-5. The resulting geometrical discontinuity causes stress risers in the embedment concrete surrounding this assembly.

Figure 3-5 Rail and Casting Joint in Early Stage of Construction

Despite the design objective, to the contrary, a rail joint assembly cannot preserve the same strength, stiffness, flexibility, and uniformity as the rail itself. These effects are even more significant in compromise joints where a T rail connects to grooved rail, as shown in Figure 3-6.

Figure 3-6 Detail of a Compromise Joint

By comparison, the field observation of welded joints experience very limited and negligible damage as seen in Figure 3-7. This notable difference reveals yet another fact which relates to the load transfer mechanism at standard joint connections.

a) Plan View

Welded
Joints

b) Side View

Figure 3-7 Detail of a Welded Joint in 10+ Year Old Pavement

3.5 Wheel Impact

By far, the most damaging group of deterioration processes in concrete pavement is due to wheel impact on the joints which will subsequently excite a response on the track. If this response exceeds the endurance limit of the concrete, transverse cracking will initiate proceedings to progressive crack growth until the concrete is no longer bounded to the rail assembly.

Wheel impacts have been known to cause progressive detachment of small fragments of rail, ultimately leading to breakage (Lee et al., 2005). Furthermore, evidence of dynamic wheel impacts breaking thermite welds on track have also been found (Grohman, 2002). Apart from being a direct cause of rail breakage, various of studies have also linked dynamic wheel impacts to being a catalyst for Rolling Contact Fatigue initiation (Transportation Safety Board of Canada, 1998 & 1999; Grohman, 2002).

In addition to causing rail damage, wheel impacts can also damage other components of the track structure. Wheel impacts, especially when they occur directly over mechanical joints initiate cracks in the concrete pavement that ultimately lead to progressive failure of the road surface.

3.5.1 Source of Wheel Impact

Wheel/Rail forces clearly play a significant role in the fatigue and failure of the track structure, rail wear, vibration and other track problems. Figure 3-8 shows the forces generated by dynamic wheel and rail contact.

Figure 3-8 Forces on Wheel and Rail

When a wheel rolls along a rail, it does not simply apply sequential point loads to the structure. Actually, there is a contact patch within which the force is distributed smoothly from zero at the edge of the patch, to a maximum load at its centre.

Ideally, the contact patch resulting from wheel–rail interaction is elliptical in shape, depicted in Figure 3-9.

a) Typical Cross section of a T Rail

b) Detail of rail head

Figure 3-9 Typical Rail Section Nomenclature Construction

This contact patch also relocates depending on the gauge and lateral alignment of track as shown in the Figure 3-10.

Figure 3-10 Contact Area of Wheel on a Curve with the High Rail and Low Rail

The process of wheel impact is shown in Figures 3-11. When the wheel travels from one side of the joint (point A) to the other side (point B) it either hits or drops. This phenomenon is referred as wheel impact. For a given rail to rail gap, dipper joints produce a much more force impact than level joints. Similarly, for a given joint geometry, wide gap produce more impact than narrower gaps.

a) Schematic Diagram of Wheel Movement on the Joint

b) Actual Wheel over a Mechanical Joint

Figure 3-11 Geometrical Discontinuity or Mismatch of the Rail Head

Another source of wheel impact results from wheel ‘out-of-roundness’ (OOR) defects, which is not relevant to this study.

The joint bar acts as a simple beam to distribute the applied load over several adjacent ties. Load is applied at the middle of the bars by the rail head ends. Since the bars are not as stiff as the adjoining rails, the flexural bending of the joint assembly and rail differ, and the applied load is found to be distributed over a smaller area of the bar than the reaction. Hence the unit intensity of pressure at the point of load application, that is, under the rail head ends, will be greater (Hay, 1982).

The primary cause of dynamic wheel impacts at mechanically jointed rails can be related to alignment and geometrical discontinuity, and/or mismatch of rail heads rather than the dimension or location of contact patch or other wheel dynamics. This is further excited by the lack of adequate load transfer at the joint. Due to varying geometry, and further, the tolerances of joint components, perfect bearing connection is rarely accomplished at a mechanical joint. Therefore, similar strength, stiffness, flexibility, and uniformity relative to adjoining rail cannot be maintained at joint locations.

Huck Fasteners (Appendix B) are one of the newest fasteners for rail applications. These fasteners provide a friction type connection by pretensioning the bolts, as depicted in Figure 3-12. Huck bolts significantly improve and maintain the load transmitting mechanism at the joint but cannot prevent the wheel impact. Figure 3-13 demonstrate this gap in side and cross sectional views.

Figure 3-12 Huck Bolt Installation at a Mechanical Joint

a) Side View of a Gap in Jointed Assembly

Typical gaps
at the joint

b) Cross Section of a Mechanical Joint

c) Side View of Connected Joint

Figure 3-13 Gap in a Mechanical Joint

The railway industry tends to eliminate this concern by eliminating or reducing the number of mechanically jointed rails replacing them with thermite welded connections. This, however, is not the case at surface track.

In examining the possible alternatives to the existing type of streetcar concrete pavement it was useful to review and reassess:

- The purpose of the various track components
- The way in which the existing components fulfill their purpose
- Alternatives to present track component design, materials and construction methods

Chapter 4

Encapsulation Technology

4.1 General

During the course of this study, it was postulated that to develop a rational load transmitting mechanism, to eliminate or at least delay the onset of fatigue cracking, would improve the service life of the surface special trackwork and thereby fulfill a very important research need.

The main requirement targeted for alternative design was to provide a means to better resist against repeated cyclic wheel impact loading at mechanical joints. While the improvement of concrete characteristics subject to fatigue is insight, as reviewed and discussed in chapter 2, it is evidenced that a resilient layer, introduced between the jointed rail assembly and concrete, could better manage this issue.

In addition to resisting fatigue loading, the joints must allow movement to occur due to temperature changes, creep and shrinkage, traffic loading, construction tolerances, or other effects (Commentary on CAN/CSA-S6-00, 2001).

Similarly bridge joints and bearing elastomers are widely used. Based on current practice CAN/CSA-S6-00 permits to use natural rubber or polychloroprene (most commonly known by the trade name Neoprene). In theory, the application of this resilient layer will benefit the track performance in two ways:

1. Reduce the amount of impact force transmitted from rail to the concrete; and
2. Reduce vehicle dynamics due to changes in track stiffness

The general criteria for the alternative resilient should also address the followings:

4.1.1 Constructability

The embedded track should be able to be constructed in a confined area, ideally not outside the limits of the track allowance, using economical and readily available materials. Construction should not require the use of overly elaborate or specialized tools and equipment, and be affected by differing weather conditions.

4.1.2 Noise Mitigation

Streetcar embedded track should minimize wheel impact, and reduce squeal and other wheel-and rail- generated noise in the relevant audible range. This can be done by careful selection of resilient component thicknesses and compositions as it relates to overall track stiffness. It does not necessarily follow that thicker or softer elastomers, and a decreased spring rate will proportionally reduce noise and vibration. Also, there is a physical limit to the vertical and lateral movement that can be tolerated in the rails which limits and works contrary to elastomers effectiveness. For design purposes, the allowable deflection of the rail is 0.01 in (0.25 mm); therefore, a trade off between the benefits of elastic load distribution and restricted deflection is required.

4.1.3 Vibration Mitigation

Embedded track should also minimize ground vibration in the 40 to 125 Hz range. Normally, these vibrations propagate through the ground over great distances and adversely affects nearby residents and/or commercial building occupants.

4.1.4 Road Surface Quality, Durability

The top road-surface materials should be durable under both streetcar and road vehicle traffic. The design should provide for a smooth running surface for vehicular traffic running both along and across the tracks. Figure 4-1 demonstrates the flaws of a pilot project, attempting to use a resilient encapsulation material. The material separated from the cast trackwork and eventually, the concrete embedment after one year in service.

Figure 4-1 Problems with Resilient Material in a Pilot Project

4.1.5 Low Maintenance

The streetcar track should be easily maintained. The embedded track should be capable of rapid and economical removal of worn rail and replacement with new rail with a minimum of roadway material removal or wastage of the original rail-confining material. Ideally this material replacement should be possible with a minimum of disruption to roadway traffic. Replacement rail should be easily installed in the same horizontal and vertical alignment as the removal rail.

4.1.6 Low Life Cycle Cost

The streetcar embedded track should have a combination of first cost, useful life and ongoing maintenance costs that are minimal compared to alternative types of construction. Amortized life cycle costs should be as low as possible despite the premium in capital cost required to encapsulate.

4.2. Encapsulation Concept

The focal point at this stage is to develop, provide and experiment with alternative solutions to enhance the fatigue durability of mechanical joints under dynamic loads, and most of all, from wheel impact loads. The material should grip the joint bars tightly, without adhesive, and remain in situ. Accordingly, the application of resilient joint bar covers, configured to the bolted connections is proposed. Rubber has a long history of application in the railway and transit industry. It has been used in a variety of shapes, size and thickness to accommodate a number of impact and vibration purposes, such as rubber pads as shown in Figure 4-2.

Figure 4-2 Rubber Pad in Subway Tie Plate

The idea of employing an encapsulation technology was initially developed on the basis of successful application of rubber enclosures on tangent rails, while providing a uniform load transmission interface.

The enclosure is made of extruded moulded thermoplastic rubber, to attenuate shock and vibration from streetcar loading, as well as protect the rail against corrosion by decoupling it from concrete and water pockets. The rail is basically wrapped p in an extruded rubber boot. This boot also enhances the electrical isolation of the rail and is thought to protect the encasement concrete from impact damage.

Figures 4-3 to 4-5 demonstrate the geometry, cross section, and in-street applications of rail enclosures.

Figure 4-3 Geometrical Dimension of Rubber Enclosure around T Rails

Figure 4-4 Rubber Enclosure around T Rails

Figure 4-5 Rubber Enclosures on Tangent Rails at Gerrard Street

4.2.1 Resilient Encapsulation Performance

The urethane or sheet rubber, proposed for encapsulating special trackwork components, was also considered primarily to attenuate shock and vibration from streetcar loading. The dampening characteristic was intended to be similar to 115 RE and NP4aM enclosures, as used to isolate adjoining rails. To satisfy this requirement, the coating should attain a maximum dynamic deflection of 10 thousandths of an inch under maximum “crush” service loading.

Additionally, the coating should mitigate stray current, from the 120 Ampere negative return rails and accommodate the recommended minimum values of dielectric strength and other pertinent physical characteristics. The material must retain its physical characteristics despite extended exposure to the following (TTC Specifications, 2005):

- UV Radiation
- Temperature fluctuations ranging from -40°C to $+50^{\circ}\text{C}$
- Repeated, heavy streetcar loading (W5 “crush” dynamic wheel load taken to be 14,000 lbs; W4 “full service” loading at 11,000 lbs; and W3 “light service” loading at 9,000 lbs)
- Abrasion from rubber-wheeled traffic.
- Combined wheel load applications estimated at approximately one-half million per year
- Chemical attack from chlorides
- Road salt and sand
- Gasoline and oil spills
- Stray current
- Ozone

The elastomer must also maintain a watertight seal to manganese steel, iron, paint, and itself; and resist tearing, cracking and permanent deformation during construction, and throughout a minimum service life of 30 years.

There are different opinions in terms of mechanical properties advantages of urethane vs. rubber. The information and comparisons provided by suppliers of each type seem to be conceited toward their product. Gallagher Corporation (2006), a well known supplier of urethane products, demonstrated the tensile and compressive strength of different types of rubber and urethane as shown in Figure 4-6.

a) Tensile Strength

b) Compressive Strength

Figure 4-6 Mechanical Properties of Urethane vs. Rubber Gallagher Corporation (2006)

TTC, in consultation with industry professionals, has introduced a number of technical properties deemed necessary to maintain the minimum physical and mechanical requirements for either of systems as presented in the coming section and Appendix C.

4.2.2 Material

The resilient encapsulation material shall exhibit properties in accordance with applicable referenced Standards. In general, all material furnished shall be new, and free from impurities, defects, and imperfections impairing strength, durability and resilience, with structural properties to withstand the strains and stresses applied during normal service.

The following Standards were used in parts to develop the material specification as per TTC Specifications, 2005):

RMA Rubber Handbook Specifications for Rubber Products

AAR M-1003	Specification for Quality Assurance
ASTM D256	Test Method for Determining the Pendulum Impact Resistance of Notched Specimens of Plastics
ASTM D257	Test Methods for D-C Resistance of Conductance of Insulating Materials
ASTM D395	Test Methods for Rubber Property – Compression Set
ASTM D412	Test Methods for Vulcanized Rubber and Thermoplastic Rubbers and Thermoplastic Elastomers - Tension
ASTM D470	Test Methods for Crosslinked Insulations and Jackets for Wire and Cable
ASTM D471	Test Methods for Rubber Property-Effects of Liquids
ASTM D570	Test Method for Water Absorption of Plastics
ASTM D572	Test Method for Rubber Deterioration by Heat and Oxygen
ASTM D573	Test Method for Rubber- Deterioration in an Air Oven
ASTM D575	Test Methods for Rubber Properties in Compression
ASTM D624	Test Method for Tear Strength of Conventional Vulcanized Rubber and Thermoplastic Elastomers
ASTM D638	Test Method for Tensile Properties of Plastics

ASTM D696	Test Method for Coefficient of Linear Thermal Expansion of Plastics
ASTM D865	Test Method for Rubber-Deterioration by Heating in Air
ASTM D955	Test Method for Measuring Shrinkage from Mold Dimensions of Molded Plastics
ASTM D1149	Test Method for Rubber Deterioration- Surface Ozone Cracking in Chamber
ASTM D1171	Test Method for Rubber Deterioration-Surface Ozone Cracking Outdoor or Chamber
ASTM D2137	Test Method for Rubber Property-Brittleness Point of Flexible Polymers and Coated Fabrics
ASTM D2240	Test Method for Rubber Property – Durometer Hardness
ASTM D2632	Test for Rubber Property – Resilience by Vertical Rebound

The Encapsulation Elastomer was designed to provide excellent electrical insulation, vibration isolation, abrasion resistance and load bearing ability. It shall be formulated to provide long service life with excellent UV and ozone resistance, good water absorption resistance and excellent oil resistance. The fully cured Elastomer must exhibit the following physical properties (TTC Specifications, 2005):

<u>Physical Property</u>	<u>Unit</u>	<u>Requirement</u>	<u>Limit</u>	<u>Test Method</u>
Hardness	Shore A Duro Points	65	Min	ASTM D2240
Tensile Strength	psi	1500	Min	ASTM D412
Elongation	%	700	Max	ASTM D412
100% Modulus	psi	500	Max	ASTM D412
300% Modulus	psi	600	Max	ASTM D412
Tear Strength	lb/lin in.	150	Min	ASTM D470 Die C
Rebound, Bashore	%	30	Min	ASTM D2632
Compression Set (22 hrs. @158°F)	%	30	Max	ASTM D395 Method B
DC Volume	ohm-cm	2.5×10^{12}	Min	ASTM D257

4.2.3 Tolerances

Castings shall be encapsulated to the following dimensions:

- i) underside of casting base – $\frac{1}{2}$ " thick
- ii) topside of casting base – $\frac{3}{8}$ " thick
- iii) outside edge of casting base – $\frac{3}{8}$ " thick
- iv) sides and web – $\frac{3}{8}$ " thick
- v) running surface (top) and other designated "areas"— to be left bare

Tolerances shall conform to permissible limits for fabrication to A4 standards, as stated according to the RMA Handbook.

Uniform coating thickness is crucial to the intended use of these materials, especially as it relates to "bottom of casting" and "top of base" encapsulation. Strict compliance with stipulated fabrication tolerances are required for all units completed under this contract. In particular, the final product shall be made to the following dimensions:

<u>Castings:</u>		<u>Dimension</u>
i)	bottom of base	$\frac{1}{2}$ " ($+\frac{1}{16}$ " / -0)
ii)	top of base	$\frac{3}{8}$ " ($+\frac{1}{16}$ " / -0)
iii)	side edge of base	$\frac{3}{8}$ " ($+\frac{1}{16}$ " / -0)
iv)	sides and web	$\frac{3}{8}$ " ($+\frac{1}{16}$ " / - 0)

4.2.4 Compression Set

Compression set tests are intended to measure the ability of elastomeric vulcanizates to retain elastic properties during the prolonged action of compression stresses.

Method B of ASTM D395 will be used in determining compression set where the specimen is compressed to 75% of its original thickness. The test assembly is then conditioned for the specified time at the selected test temperature.

Compression set determined by Method B is the difference between the original thickness of the specimen and the thickness after testing, as a percentage of the deflection employed.

The Compression Set for a ½” thickness of castable elastomer must not exceed 1/16” in 25 years.

4.2.5 In-Street Considerations

For the in-street scenario, at bolted joints, certain “areas” must remain bare until after final installation of splice bars and Huck fasteners. At such locations, not only the running surface, but the sides/web, and top of base must be kept bare for a distance of 14 inches from the casting end. Once completed, a separate, custom pre-formed cover shall be installed directly over these areas. In addition, other discrete areas on the top of casting base shall also be left bare to accommodate the fixation of electrical continuity bonds. These areas shall be 5” long and full width, from the web to the edge of the base; they shall be located opposite the transition point at the end of the casting “body”.

Wedges shall be installed to fill the space between casting legs that are aligned at angles less than 80 degrees, relative to one another. Generally, wedges must be load bearing, flush with the road surface and shall extend from the apex of the angle to the beginning of joint bar (i.e. 18” from end of the casting). Wedges shall be custom fitted to each location and must accommodate joint bar installation. Where the shape of the filler is locally constricted by joint bars, the wedge shall be made of two or more pieces, and the installation sequenced appropriately.

Material used for the Black Flexible Rubber Wedges shall be 100% EPDM and must have superior weathering, aging, abrasion, UV, chloride and heat resistance characteristics, and provide high impact and energy absorbing properties over a wide range of ambient temperatures. The Black Flexible Rubber Wedges must be load bearing and exhibit the following physical properties:

Physical Property	Unit	Requirement	Limit	Test Method
Hardness	Shore "A" Duro Points	70 ± 5	Min	ASTM D2240
Heat Resistance 70h @ 257°F (125°C), Change in Hardness	Pts.	15	Max	ASTM D573
Compression Set, 22h @ 158°F (70°C)	%	25	Max	ASTM D395
22h @ 257°F (125°C)	%	70	Max	Method B
Ozone Resistance, Bent Loop, 70 hr @ 104°F (40°C) @ 50 pphm		No Cracks		ASTM D1171 Method B
Tear Strength	lb/lin in.	200	Min	ASTM D624 Die C
Low Temperature Brittleness		No Cracks After 3 Minutes @ -67°F (-55°C)		ASTM D2137 Method A, Non Brittle 9.3.2
Water Resistance 70h @ 212°F (100°C), Volume Change	%	5	Max	ASTM D471

Chapter 5

Test Program

5.1 General

A programme of laboratory research was initiated to understand the contribution of jointed rail performance, under repeated wheel loading, to the initiation and growth rate of cracks in specimens configured to the current assembly as well as the proposed encapsulation technology. The work was undertaken with the cooperation and support of the Toronto Transit Commission, Performance Polymers Inc and Polycorp Inc. and conducted in the structural lab of Ryerson University. (See Appendix D - Schedule)

Three types of joints were adopted for the experimental program. The first type mimicked the current construction of jointed rails, and is referred to here as the control specimen. Two other types, one using sheet rubber and the other, urethane were also fabricated. For test purposes, a total of nine jointed rails with identical shape and geometry were manufactured, three of each system, to ensure the consistency of the results.

5.2 Specimen Geometry

Wheel impact fatigue creates localized damage adjacent to the joint. For this reason, a minimum length of rail for each side of joint was used. Individual pieces were cut to a 590 mm length, sufficient to mount the joint bars and also to make the optimum use of 39 ft length rails donated for the test. The minimal length also made it easier to transport and test full size samples.

A dismantlable formwork was designed and fabricated to accommodate multiple casting of specimens. To allow approximately 50 mm cover the overall dimension of each concrete specimen was 1280 mm long, 900 mm wide and 300 mm high.

Figure 5-1 shows the design drawing of the formwork. The installed formwork of the first control specimen is shown in Figures 5-2 and 5-3.

Figure 5-1 Dismantleable Formwork of Specimens

Figure 5-2 Assembled Formwork

Figure 5-3 Specimen C1 Prior to Placing Concrete

Figures 5-4 to 5-6 show the overall dimension and detail of all three types of specimen. The lateral movement of the in-street pavement is restrained by its mass volume. To prevent the lateral movement of test specimens in the lab, four anchorage bolts were utilized to tie them to the rigid floor. Dimension of the formwork also permits the installation of anchorage sleeves at 600 mm centre to centre.

Figure 5-4 Control Specimens, C Type

Figure 5-5 Urethane Encapsulation Specimens, U Type

Figure 5-6 Sheet Rubber Encapsulation Specimens, R Type

5.3 Test Equipment

A MTS 243-35T hydraulic actuator was used for cyclic loading, as seen in Figure 5-7. These types of actuators are heavy-duty, and their force generating components are designed for structural fatigue and static test applications. Single-ended models are available with a compression relief valve to provide equal tensile and compressive forces. They are suitable for a broad range of structural fatigue and static test applications requiring precise force generation and accurate control of piston rod displacement.

The actuator works either in load or displacement mode. For this test program the load mode (constant loading) was set, and capable of applying cyclic load of up to 3 to 4 Hz frequency. Figure 5-8 shows the Swivel Head-Base detail. To replicate the exact imposing load condition, a piece of streetcar wheel was cut, reinforced and mounted on the Swivel Head-Base by means of a supporting, high strength bolted T-section as shown in Figure 5-9. (See Appendix E and Appendix F)

Figure 5-7 MTS 243-35T Hydraulic Actuator

Figure 5-8 Swivel Head-Base Detail

a) Streetcar Wheel

b) Actuator Head

Figure 5-9 Actuator Head Detail

5.4 Design Load

The wheel impact load used in this study is based on the equivalent static method, as presented in TTC Design Manual One (2005). This is due to the fact that there is a contradiction in the technical literature as to what would be the most appropriate impact load factor for the purpose of dynamic analysis. Measurements and predictions have failed to suggest a consistent value for the wheel impact load factor.

In most of the experimental investigations, commercially available Wheel Impact Monitors are used that can be broadly categorized into two groups, either strain gauge based or accelerometer based systems. The strain gauge based systems utilize a series of strain gauges, bonded onto the rail web and configured into a full shear bridge; accelerometer based systems utilize accelerometers mounted on the rail foot to receive track responses.

The strain gauge based system is advantageous in that it allows an impact load to be reported in units of force. However, its disadvantages include an inability to provide impact detection over the sleeper areas and to provide full wheel circumference coverage.

Furthermore, a true impact force can only be detected if the strike occurs directly in between the shear bridge. Typical, accelerometer based systems utilise a number of transducers set up to provide coverage along one full wheel revolution. This system is reported to be better than strain gauge based systems in that it can cover 100% of the wheel circumference.

The track acceleration response, measured as a result of an impact event, can be processed and reported in many ways. Some researchers use peak acceleration as a measure of damage, while others convert the acceleration response into an output bearing units of force. There is currently much conjecture regarding the conversion of raw acceleration readings into units of force. The analysis process by which it is accomplished is also not well documented (Grohman, 2002; Lee et al., 2005).

A wide range for the load factor from cyclic wheel load application and wheel impacts has been reported by different researchers. The AASHTO LRFD 2000 Specification is silent on the rail impact load distribution and transfer. Alternatively, this factor is suggested as 50% for deck joints by CAN0CSA-S6 2000 and TTCI (Otter, 2004) for rail way mechanical joints. For this study, the load factor was selected as 43% based on the TTC design Manual (2005), for both ALRV and CLRV series of streetcars as shown in Figure 5-10 and explained hereafter.

a) ALRV Streetcar

b) CLRV Streetcar

Figure 5-10 Streetcar Wheel Load Model in the TTC Design Manual (2005)

As can be interpreted from this figure, the ALRV series, with a maximum weight of 53000 kg will impact a larger wheel load than the CLRV series, having a maximum weight of 34000 kg over 4 wheels. Therefore, the wheel load of ALRV cars was selected which provides a more conservative approach.

Accordingly, design load condition can be derived from the followings:

Total Vehicle Load (L) = 53,000 kg

	<u>Front</u>	<u>Centre</u>	<u>Rear</u>
Axle Load	89 kN	82 kN	91 kN
Wheel Load	44.5 kN	41 kN	45.5 kN
Dynamic Load Allowance (DLA)	30% (L)		
Rolling Force (RF)	$\pm 10\% \times \text{Wheel Load}$		
Centrifugal Force	$\frac{V^2}{Rg} \times (L)$		
Hunting Force (HF)	45 kN (Lateral) @ Lead Axle of Train		
Normal Acceleration/Breaking (LN)	15% (L)		
Emergency Breaking (LE)	30% (L)		

The static load was calculated based on the design table provided by the streetcar manufacture, with the maximum static load in Crush Mode, as demonstrated in Table 5-1.

Table 5-1 Vehicles Capacities and Loading (TTC Specifications, 2005):

a) General Information

<u>Vehicle Class</u>	<u>Empty Weight(lb)</u>	<u>Seating Capacity</u>
CLRV	50,000	46
ALRV	80,900	61

b) Static Loading Modes

<u>Load Definition</u>	<u>Symbol</u>	<u>Definition</u>	<u>Passengers</u>	<u>Total Weight</u>
	W1	Empty	NIL	Light Weight (Column1- Table a)
	W2	Seated	Column 2 of Table a	Number of passengers plus operator at 150 lb. each added to W1 weight
	W4 (Table c)	Service	Seated Plus Standeers @ 2.3 Sq. ft.	
	W5 (Table c)	Crush	As for W4 Except @ 1.5 Sq. ft.	

c) Vehicle Capacities

<u>Loading</u>	<u>CLRV</u>	<u>ALRV</u>
W4	102	155
W5	132	205

Accordingly the design load was calculated as 65 kN. The frequency of 3 Hz was adopted to represent actual travel of streetcar with the higher speeds. This value complies with the average of 2 to 5Hz fundamental frequencies as suggested by CAN/CSA-S6.1 (2001).

The number of cycles selected for each test is considered as 200,000 cycles which represents a timeline of approximately 2 years. This limit is based simply on the number of expected vehicle passes on a typical streetcar line. Assuming a 24 hour line (Queen Street) with average 5-min headway, we could expect over 100,000 vehicle passes per year. Field observation has proved that initial cracking appears as early as year one or year two of the track being in service.

5.5 Mix Design

The specimens were made of ordinary concrete, using Portland cement and crushed aggregates with the compressive strength of 32 MPa. The ratios by weight of the mix components to cement were as per Appendix G for surface track.

5.6 Specimen Preparation

18 rail pieces, or 9 joints, were cut from straight, new NP4 rails, each of 590 mm length. Polycorp and PPI manufactured rubber sheet and urethane encapsulation for each of three pairs, respectively. Later, all pieces, including those left bare for control specimens, were connected with standard joint bars using the Huck bolt system (See Appendix B)

The concrete was then mixed, cast and vibrated as a conventional concrete. Elaborated samples were 150mm in diameter and 300 mm long. They cured in the steam room for 28 days. After their curing process, the samples had a density of $2.47 \times 10^6 \text{ g/m}^3$. Mechanical tests carried out with cylindrical pieces, yielded an average compressive strength of 34 MPa.

5.7 Fatigue Testing

The specimens were positioned vertically between the actuator head and floor, and were anchored to the strong floor by four 1¼" A325 bolts to prevent lateral movement. The test arrangement was closely related to represent the impact phenomenon in the streets.

Sinusoidal loading was defined from discrete data acquisition controlled by system. However, the model does not account for the effect of temperature, rate of loading, or different strength of embedment concrete. The peak values, displacements and corresponding time (and number of cycles) were recorded throughout the test.

For given design values of load (or stress), and using constant displacement, damage initiation was monitored as a function of the number of load applications; specifically, the criterion for damage measurement was the development of inclined cracks at the rail joint as a function of the number of load cycles applied.

As a complement to damage assessment, the strain at the field and gauge side of rail was measured on both sides of selected joints. For the last sheet rubber and urethane specimens, two transverse strain gauges were installed adjacent to the joint to evaluate and compare the strain difference of these two systems.

All tests were performed with identical boundary conditions for geometry, anchorage and material properties with an initial load of 65 kN, displacement of 200 to 210 mm, frequency of 3 Hz up to 200,000 cycles. In addition, investigations on crack growth, in the post crack zone under repeated loading, were carried out using a manual crack detector.

Figures 5-11 to 5-19 show various stages of specimen preparation and testing.

Figure 5-11 Sheet Rubber Specimen, R2 in Formwork

Figure 5-12 Urethane Specimen, U2

Figure 5-13 Control Specimen, C1 after Concreting

Figure 5-14 Mobilization of Steel Structure Frame for Testing

Figure 5-15 Steel Structure Frame used for testing

Figure 5-16 Installation of Strain Gauges on the Rail Head

Figure 5-17 Actuator Alignment, R2

Figure 5-18 Final Positioning of Actuator Head

23.02.2006

Figure 5-19 Specimen C1 under Fatigue Testing

Chapter 6

Results and Discussion

6.1 Experimental Results

A cyclic load of 65 kN at a 3 Hz frequency of was applied to all specimens during 200,000 cycles of load application. To maintain similar boundary conditions for each specimen, all aspects of loading, anchorage, height of the actuator, as well as specimen geometry and mechanical properties were identical.

Table 6-1 represents the summary of experimental results obtained during this study.

Table 6-1 Summary of Experimental Results

Specimen	Acronym	Load (kN)	Frequency (Hz)	Initial Crack	Number of Cycles
Control 1	C1	65	3	14000	200,000
Control 2	C2	65	3	15000	200,000
Control 3	C3	65	3	18000	200,000
Urethane 1	U1	65	3	No Crack	200,000
Urethane 2	U2	65	3	No Crack	200,000
Urethane 3	U3	65	3	No Crack	200,000
Sheet Rubber 1	R1	65	3	No Crack	200,000
Sheet Rubber 2	R2	65	3	No Crack	200,000
Sheet Rubber 3	R3	65	3	No Crack	200,000

All three control specimens started to crack at or below a maximum of load cycles. The six other specimens of both sheet rubber and urethane encapsulation, however, remained uncracked for up to the target 200,000 cycles. The first cracks developed at the top face of control specimens, adjacent to the joint, and had transverse direction absolutely identical to inclined cracks observed in the field. Figure 6-1 illustrates the point of crack initiation for each of the control specimen while Figure 6-2 demonstrates the development of first cracks in specimen C1.

Figure 6-1 Initiation of First Crack in Control Specimens

Figure 6-2 Inclined Crack at Joint, Specimens C1

Prior to load application some minor cracks due to concrete shrinkage were observed, but their impact was insignificant in the direction of the test.

Additionally, if we have a slab (of moderate dimension) resting freely on its supports, it could contract to accommodate the shortening of its length produced by shrinkage. This however would not be the case in situ where pavements and other members are jointed rigidly to other parts of the track (i.e., rail, fasteners and base and therefore cannot contract freely). During testing, the field condition was mimicked by tying all specimens to the strong floor with high strength A325 bolts, as shown in Figure 6-2.

With reference to the control specimens, it was observed that the first inclined cracking began as early as 14000 cycles. At about 40,000 cycles, the hair line cracks were quite noticeable on the surface. The crack growth, both in terms of length and width, were slowed down dramatically after 100,000 cycles. Figure 6-3 demonstrates the length of the crack between 30000 to 60000 cycles for specimen C3. The overall crack length varied from 359 mm in C2, to 383 mm in C3 and 425 mm in the C1 specimen.

Figure 6-3 Crack Progress Length, Specimen C3

It was also observed that in some cases more than one branch developed as the crack propagates, as demonstrated in Figure 6-4. Accordingly, it can be interpreted that the crack development is a function of the number of developed cracks and/or their branches

which relates to energy absorption by the crack. This progress is proportional to the number of cracks, as well as the concrete composite at the surface. In the latter, it was noticed that the crack progressed at a faster rate through paste than through aggregates.

Figure 6-4 Crack Progress Length, Specimen C3

When the crack reached a smaller size aggregate, it generally propagated along the surface of the aggregate; for the larger particles (Figure 6-5) the crack passed through the aggregate.

Crack passed
through the
aggregate

Figure 6-5 Crack Progress through a Fine Surface Aggregate

The cyclic impact test revealed that the anticipated inclined cracks due to wheel impact at the joint became a typical failure mode on the control specimens. This is consistent and representative of the current construction experience and can be correlated to the brittle nature of the concrete composites and lack of ductility. In fact, the damping behaviour of the concrete against fracture is very small and contributes little to total energy absorption.

Accordingly, an outstanding improvement of fatigue properties for concrete near mechanical joints is observed for both urethane and sheet rubber encapsulation. This improvement appears to be very much dependent on the damping mechanism which was introduced by either of these transition layers. When a rubber or urethane jacket is used in the assembly, it contributes by mitigating the actual stresses transmitted to the concrete.

This greatly enhances the performance under repeated loading, by prolonging the service life of the system. In addition, it can be anticipated that the rate of the decrease in post-crack impact stresses with aging would be slower than the reference samples.

6.1.1 Force and Displacement Measurement

Forces seem to fluctuate normally for all specimens having sheet rubber and urethane encapsulation as well as the control specimens, when loaded between 62 to 65 kN, depending on the relative duration which reading took place. The other variable was the distance which the actuator took to reach the targeted load of 65 kN from the time it touched the rail head. This value is referred to as displacement and measures the rigidity of each specimen.

Figures 6-6 demonstrates the absolute displacement vs. time for control, urethane, and sheet rubber specimens. The result was calculated from the average recorded readings for each type of specimen as their reaction was identical under imposed cyclic loading.

As depicted, both urethane and rubber specimens exhibit a greater range in displacement due to their resiliency and damping mechanism as opposed to the control specimens which have a smaller range.

As shown in the Figure, urethane and sheet rubber specimens exhibit an average of 107% and 109% increased resiliency, respectively as compared to control specimens, under applied loading.

The difference between urethane and sheet rubber is considered insignificant and consequently, this result shows that both encapsulation methodologies have a higher resilience than the control specimens thus a better energy absorption characteristic. This cushion allows the concrete pavement, near encapsulated joint assemblies, to accommodate repeated wheel impacts without developing cracks under the imposed shock loading.

Figure 6-6 Average Displacement- Time Graph for Specimens

6.1.2 Strain vs. Time Measurement

Figure 6-7 shows the position of strain gauges installed for this test to record the strain level on both sides of the rail head.

Figure 6-7 Position of Strain Gauges

Figures 6-8 to 6-10 present the strain vs. time graphs for control, urethane and sheet rubber specimens respectively. In these graphs, compressive strain is shown as positive and tensile strain as negative. Because the load applied on the field side, the recorded strain for the field sides were all in negative associated to compression strain.

In general, the strain time graphs had a constant pattern for each type of specimen. It was noticed that 2.5 second interval is sufficient to include two full cycles, and accordingly, this interval was selected to graph the results approaching 200,000 cycles.

These graphs suggest that both urethane and rubber specimen experience lower strains compared to control specimens. The average strain level recorded during this study reached amplitudes of $-47 (x10^{-6})$, $-26(x10^{-6})$ and $-21.5(x10^{-6})$ for control, urethane and rubber specimens, respectively. These results indicate that, in terms of load application and regardless of other contributing factor such as temperature gradient, the rubber provides slight advantage in damping the impact load. This advantage might also be attributed to other factors such as differences in product geometry and size available in the current market.

Figure 6-8 Strain - Time Graph for Control Specimen, C2

Figure 6-9 Strain - Time Graph for Urethane Specimen, U2

Figure 6-10 Strain - Time Graph for Rubber Specimens, R2

To measure the strain level transmitted to the concrete, additional strain gauges on the concrete surface adjacent to the joints, as shown on Figure 6-11, were installed on the last rubber and urethane specimens. Figures 6-12 and 6-13 suggest similar pattern for strain level, with maximum strain level as -23.2×10^{-6} and -60×10^{-6} for rubber and urethane specimens, respectively.

Figure 6-11 Complete Set of Strain Gauges

Figure 6-12 Strain - Time Graph for Urethane Specimen, U3

Figure 6-13 Strain - Time Graph for Rubber Specimen, R3

Apart from the slight advantages of rubber, both systems prove to be workable for the purpose of jointed rail encapsulation. This is supported by the fact that all urethane and rubber specimens remained uncracked for the target 200,000 cycles in comparison to the control specimens which cracked well below 20,000 cycles.

Accordingly, it can be interpreted that both systems exhibit a superior performance, withstanding a maximum of 10 times more load cycles than control specimens which represent the current design and construction. In practice, however, the required improvement factor is around 3 as related to 50 years of desired service vs. the current average of 15 years. Regardless of the practical limit of additional extended life beyond a multiple of 3, the track will be replaced after 50 years due to the deterioration of castings and rail, as well as, the City plan for long term road reconstruction.

6.2 Fatigue Mechanism of TTC Streetcar Pavement

The average life of TTC streetcar concrete pavement is determined by many factors including initial design, material properties, traffic (both streetcar and road traffic), environmental and durability issues, the presence and effectiveness of protective systems, maintenance practices and above all, its resistance against failure mechanisms.

As previously described in Chapter 2, the fatigue life of concrete has traditionally been divided into three stages: crack initiation, crack propagation and final fracture. The life cycle of TTC streetcar concrete has shown some similarities to this model. However, it can be noticed that it does not follow this pattern.

The distinguishing factor is serviceability of the system as opposed to load capacity and failure. This system falls in serviceability limit state in contrast to ultimate limit state and the service life of a streetcar concrete pavement is terminated when it loses its functionality. Track replacement takes place when the overall condition of surface concrete deteriorates to a point where damaged road vehicle becomes a probable occurrence. Although locally, castings and rail joints can be excavated, tightened and patched to prolong life, but eventually it becomes more practical to provide a new surface track for both streetcar and road traffic and to eliminate excessive noise and vibration.

6.2.1 Proposed Damage Mechanism

The following model for fatigue life assessment of TTC concrete pavement has been generated to consider its special circumstances. This model consists of five stages; it is compatible with the results of the experimental program, as it relates to the fatigue life of concrete pavements, as well as field observations.

One of the aspects of this model is that each stage does not necessarily end prior to the start of the next stage. As a matter of fact, these stages typically overlap, and in some cases, have increasing influence on subsequent stages so as to reduce the service life of the system.

The failure of concrete pavements, encasing mechanical joints, progress through the following five stages:

Stage I: Crack Initiation

Deterioration of the streetcar concrete pavement mainly begins by the initiation of surface cracks, mostly concentrated around mechanical joints. During the course of field observation, three major sources of cracks were identified.

a) Wheel Impact Fatigue Cracks

Fatigue cracks are the most dominating source of cracks. In form, they consist of inclined surface cracks at the joint, resulting from repeated wheel impact loads. As previously discussed, a wheel impact is caused whenever a streetcar wheel drops or hits one side of jointed rail while moving from the other side.

This, in turn, is the result of a geometrical discontinuity and/or mismatch of the head section across two jointed rails, and has proved to be the most common cause of deterioration mechanism requiring repair, rehabilitation and eventually, track replacement. Figure 6-14 shows the movement of one wheel as it approaches the jointed section, while Figure 6-15 shows fatigue cracks in a typical intersection.

Figure 6-14 Geometrical Discontinuity and Mismatch of Rail Head at the Joints

Wheel impact loading cracks at the early stages of the pavement's service life

Figure 6-15 Fatigue Cracks at a Joint

b) Shrinkage Cracks

Shrinkage is the second source of cracks in streetcar concrete. Shrinkage cracks are developed on the top surface of the pavement in a transverse direction to traffic, and progress deep into the concrete.

The photo shown in Figure 6-16 reveals such transverse cracks on the side of the concrete pavement in early stage of construction.

Shrinkage cracks observed right after the completion of construction works

Figure 6-16 Shrinkage Cracks after Construction

c) Thermal Cracks

The third source of cracks in concrete pavement is associated with thermal stresses in the concrete. Thermal stresses develop tensile stress (due to restrain) in the longitudinal direction of the pavement as a result of the thermal expansion and contractions of the rail. It seems that current construction technology has increased the likelihood of this type of cracking at the joints. This is due to the fact that rubber enclosures, installed as a jacket throughout the tangent portion of the rail, will serve to isolate the rail from the encased concrete along the track except at the joint locations.

Accordingly, the concrete at the joints will act as the fixed support and must accommodate this tensile loading. The poor tensile characteristics of plain concrete at the joint, however, will not be sufficient to mitigate this loading and eventually, the concrete will crack.

Rubber enclosure
decouples the rail
from concrete

Bare area of the
joint in contact
with concrete
serves as fixed
support during
thermal
contractions

Figure 6-17 Mechanical Joint before Concreting, TTC Hillcrest Yard

As shown in Figure 6-17, the enclosure is installed just short of the joint, creating a fixed point to restrain the thermal expansion and contraction of the rail. Figure 6-18 reveals such transverse cracks on the top surface of the concrete of the TTC Hillcrest Yard, which can be correlated to shrinkage as the track experiences a limited traffic.

Thermal
cracks in
concrete

Figure 6-18 Mechanical joint before concreting, TTC Hillcrest Yard

A change in temperature may have an effect similar to shrinkage, which also results in stresses in the pavement. Increasing temperature results in compressive stress and decreasing temperature results in tensile stress.

As the tensile stress produced by shrinkage and temperature attains the tensile strength of concrete, cracks will form. These cracks typically occur on both top and bottom faces of a slab, in a transverse direction to traffic; occasionally, they overlap or connect to other cracks to create a network of cracks.

The other benefit of the encapsulation work is to mitigate in large and even eliminate major thermal cracks by decoupling the rail from concrete at the joint, as shown in Figure 6-19. A better performance is achievable when the encapsulation layer covers the whole joint, in a single piece, and overlaps the enclosures at both ends.

Encapsulation
must cover the
whole joint

Figure 6-19 Encapsulation at a Joint

Stage II: Crack Progression

The second stage begins when inclined cracks progress until they reach the asphalt pavement and concrete separates in transversal direction. Some longitudinal cracks are developed on the top as well. At this stage, due to the transverse cracks formed in the concrete, the slab loses load distribution in the longitudinal direction. This might lead to development of cracks in the longitudinal and transverse direction, under extreme loading conditions. On the top face of the pavement, repeated traffic loading leads to the initiation and growth of transverse cracks, as shown in Figure 6-20.

a) Side View

Inclined cracks progress until the concrete pavement separates in transverse direction

b) Top View

Figure 6-20 Continuous Transverse Cracks

Since these top transverse cracks are formed in weak sections, they often join other transverse cracks, forming through cracks. Eventually, these of cracks form a network (Figure 6-21) together with first set of fatigue, shrinkage and thermal induced cracks.

Figure 6-21 Crack Network and Formation of Through Cracks

Stage III: Environmental Impact

In the third stage, water penetrates into the through cracks. Water often penetrates into the concrete pavements, including the joint area, which has no means of drainage; consequently, it tends to retain this water for a long period of time. Cracks facilitate the movement of water and other deteriorating agents through the concrete. The surface cracks are also subjected to freeze and thaw cycles. Freezing water expands, contributing to additional damage such as an increase of crack width and depth. Each passage of a heavy wheel may cause a strain/stress concentration which leads to further cracking and disintegration of the cracked pavement.

Additionally, as shown in Figure 6-22, continued concrete shrinkage cyclic temperature change lead to the growth of the transverse cracks. These factors working together lead to the formation of through-thickness cracks. Water, particularly when containing chlorides (salt used as a deicing agent), penetrates the cracks and causes corrosion as it reaches the steel components. Corroded steel parts expand, which results in further cracking and disintegration of cracked pavements.

Damaged concrete
due to salt scaling
and freeze and thaw

Figure 6-22 Durability Issues in Streetcar Concrete Pavement

Stage IV: Erosion

Next, the through cracks are gradually worn out under repeated traffic loads. The wearing out of the cracks leads to the loss of load distribution in the longitudinal direction. As a result, the concrete pavement does not behave as a plate any longer, but rather acts as transverse isolated concrete supports. The presence of the water accelerates the wearing out of the cracks, as shown in Figure 6-23.

Rails are still in good
shape while the
concrete is worn out

Figure 6-23 Erosion Effect in Concrete Pavement

Stage V: Serviceability Failure

Finally, the pavement is no longer serviceable due to insufficient integrity of the concrete excited by lack of steel reinforcement of the slab. Spalling of concrete becomes more widespread, and eventually depression of the slab takes place, leading to service life termination.

Rail is bent and has lost straightness

Streetcar concrete pavement at the final stage; deep cracks and pot holes observed.

a) Disintegrated Concrete at a Joint

b) Disintegrated Concrete in between Rails

Figure 6-24 Severe Serviceability Issues in Streetcar Track

Chapter 7

Conclusion

7.1 Summary

This study presents an experimental program on the potential application of resilient encapsulation technology in concrete pavements, to encase mechanically bolted rail joints in TTC light rail embedded surface track. It is recognized that concrete fatigue cracking due to wheel impact as well as thermal stresses and shrinkage crack (as a result of temperature gradient in the transverse direction) in concrete pavement strongly influences the service life of slabs composing the track allowance.

Deterioration of concrete pavements often requires significant repair, rehabilitation, or replacement of concrete pavements. Therefore, innovative technologies for repair and for new construction of concrete slabs were investigated.

The deterioration of concrete slab is caused by concrete shrinkage, temperature changes, freeze/ thaw cycles, and repeated loading. However, this study indicated that, in streetcar concrete pavements, the importance of fatigue loading is higher than any other degradation and deterioration mechanism. It is apparent that maintaining the integrity of the concrete is actually key to improving the service life of the system. Therefore, the prevention of fatigue failure in concrete pavements is crucial, and sequence of five stages described in Chapter 6 must be interrupted before final crack failure. Specifically, the failure mechanism of concrete pavements that encase mechanically bolted rail shows the formation of through cracks, under repeated wheel impact loading, to play a major role in the sequence of five stages.

The formation of through cracks is completed relatively early in the service life while the remaining life shows progressive erosion, and the wearing out of concrete cracks together

with other environmental and mechanical deterioration. This implies that increased fatigue crack resistance should extend the service life of streetcar concrete pavement, since progressive crack growth is promoted by the low fatigue crack resistance of the present design.

Furthermore, if fatigue cracking can be arrested, none of the subsequent stages becomes a problem, thus preventing the early service life termination of streetcar concrete pavements. Therefore, research efforts are needed to investigate and improve the fatigue durability of concrete pavements that encase the mechanical rail joints and, in turn, the fatigue crack resistance of this concrete material.

7.2 Conclusions

Based on a detailed review of parameters affecting the fatigue life performance of both control and encapsulated specimens, as observed during the experimental investigation conducted in this study, the following conclusion can be made:

- Concrete materials suffer from fatigue, i.e. they lose load bearing capacity or in this case serviceability when fractured by a fatigue crack after a large number of subcritical load repetitions. Typically, subcritical stable crack growth occurs under tensile or flexural loads. However, as established during the experimental program cyclic compressive load has also been shown to produce fatigue crack growth in concrete.
- By far, the most damaging group of deterioration processes in concrete pavement are due to wheel impact at the joints, which subsequently excite a response on the track. If this response exceeds the endurance limit of the concrete, then transverse cracking will initiate proceedings to progressive crack growth until the concrete is no longer bounded to the rail assembly. Excessive noise and vibration during normal traffic arise, plus, the extensive cracking and potholes have a direct effect on traffic safety and the serviceability of the track.

- Any new embedded track design must incorporate a resilient material to surround the rail; otherwise, the track structure will be too stiff and vulnerable to cracking under repeated impact loading. Any embedded track design which places concrete directly against the sides of the rail head will negate the resiliency of the rail support, again resulting in a track structure that is too stiff.
- Initial cracks due to shrinkage and temperature change: Concrete shrinks as the cement paste hardens, consequently, the value of shrinkage can be reduced by using a mix design with the smallest possible amount of water and cement compatible with other requirements (such as strength and workability) and by moist-curing for a sufficient duration.
- A damage mechanism for fatigue life assessment of TTC concrete pavement has been generated to consider its special circumstances. This model consists of five stages and is compatible with the results from the experimental program, as it relates to the fatigue life of concrete pavements, as well as to field observations.
- An outstanding improvement of fatigue properties at mechanical joints were observed for both urethane and sheet rubber encapsulation. This improvement appeared to be very much dependent on the damping mechanism which was introduced by either of these transition layers. When a rubber or urethane jacket is used in the assembly, it contributes to mitigating the applied stresses transferred to the concrete due its improved damping characteristics. This greatly enhances the performance under repeated loading thus prolonging the service life of the system. In addition, it is anticipated that the rate of the decrease in post-crack impact stresses with aging would be slower than for the reference samples.
- Both urethane and sheet rubber specimens showed an approximate 10 times superior performance for the range of cyclic loads applied. In practice, however, the required improvement factor is closer to 3, due to planned track replacement per the City's plan for road reconstruction.

7.3 Recommendations for Further Research

The encapsulation technology proved to be an appropriate means for enhancing the fatigue life of pavement concrete encasing special trackwork. During the course of this study it was determined that many aspects could be expanded and further investigated, including:

- Effect of varying wheel loads and vertical displacement. TTC is investigating the use of low streetcars which have an approximate 50% heavier axle loads.
- Effect of hardness and thickness of resilient materials in combination with temperature response. The shape factor and thickness of material contributes in large to the performance of encapsulation system.
- Effect of combined environmental and mechanical loading

Additionally, a formal cost-monitoring procedure should be established to record construction costs, track performance, and ongoing track maintenance costs. This will permit life cycle cost comparisons when new track construction materials and techniques are to be evaluated.

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Appendix A – Cyclic Stresses

Cyclic Stresses can be commonly categorized in three following forms:

**Periodic and
symmetrical
about zero
stress**

**Periodic and
asymmetrical
about zero
stress**

**Random
stress
fluctuations**

Figure A-1 Form of Cyclic Stresses (Gangloff, 2005)

Cyclic stresses are further characterized by maximum, minimum and mean stress, the range of stress, the stress amplitude, and the stress ratio. (ACI 215)

Figure A-2 Mean Stress (Gangloff, 2005)

Mean stress: $\sigma_m = (\sigma_{\max} + \sigma_{\min}) / 2$

Range of stress: $\sigma_r = (\sigma_{\max} - \sigma_{\min})$

Stress amplitude: $\sigma_a = \sigma_r / 2 = (\sigma_{\max} - \sigma_{\min}) / 2$

Stress ratio: $R = \sigma_{\min} / \sigma_{\max}$

Appendix B - Huck C50L® Lockbolt

The rugged C50L® PowerBolt fastener is an alternative to Grade 5 bolts. Available in diameter ranging from 1/2" to 1-3/8", a variety of materials and head styles, the C50L® is ideal for applications where consistent, uniform clamp force is required.

Fastener Dimensions

Round Head – C50LR

		Round Head		Truss Head C50LT		Truss Head C50L30		90° Flush Head	
Dia.	A	B	C	B	C	B	C	B	C
1 (32)	1.022 - .990	1.53/64	39/64	1.53/64	15/32			1.27/32	1/

Grip Data & Dimensions

Grip is the actual thickness of material to be fastened. Grip includes 'C' dimension on 90° flush head and 60° water tight fasteners. Grips listed are not necessarily available in every head style.

Contact HUCK for grips not shown.

Tolerances: Dimension D- + 1/32, Dimension E +3/16, -1/16

Grip	Grip Range	1" Diameter (-32)	
No.	min - max	D	E
56	3 1/2 - 3 3/4	5.073	8 3/8
64	4 - 4 1/4	5.573	8 7/8
72	4 1/2 - 4 3/4	6.073	9 3/8

Collar Dimensions

Flanged Collar - 3LC

3LC	1/2	.515 - .535	.807 - .793	.746 - .716	.156 - .094	1.031 - .969
	1	1.030 - 1.050	1.575 - 1.550	1.515 - 1.485	.281 - .218	1.906 - 1.843

Inspection Data

Dia.	Pin	Collar	Collar	A max	B max	C min	D max
	Part No.	Type LC*	Type 3LC*				
1	C50L(-)(-)32	LC-2R32G	3LC-2R32G	1/16	3/8	7/8	1.465

Installation Tooling

Dia.	Installation Tool	Nose Assembly	Type	Use
1	507	<u>99-5016</u>	Hydraulic	Production

Hole Data

Dia.	Maximum Hole
1	1.1/16

- 1 When using 3LC collars, include J dimension in grip dimension to determine grip number.
- 2 Optional collar at reduced clamping force and tensile strength. Contact HUCK for data. When using 8LC collars, subtract J dimension from grip dimension to determine grip number.

TAB-LOK™ TAB-LOK™ The Tab-Lok feature makes sure the collar stays on the pin, before installation, in overhead and down slanted pin placements.

Installed Values

Dia.	Shear	Clamp	Tensile
1	56500	51500	72700

1 Minimum values, in pounds, of installed fasteners. Round head, in carbon steel equivalent to ASTM A325 high strength bolts. Meets requirements of MIL-P-23469.

Appendix C - Technical Properties of Encapsulated Materials

Technical properties of the thermoplastic rubbers are as followings:

<u>Method</u>	<u>Property as Received</u>	<u>Unit</u>	<u>Requirement</u>	<u>Limit</u>	<u>Test</u>
D2240	Durometer Hardness	Shore A	70	± 5	ASTM
D412	Tensile Strength	psi	1200	Min.	ASTM
D412	Ultimate Elongation	%	300	Min.	ASTM
D257	Volume Resistivity (Saturated NaCL Solution)	ohm-cm	10 ¹²	Min.	ASTM
D395	Compression Set 22 hours @ 158°F Method B	%	35	Max.	ASTM
D573	Oven Aging 70 hours @ 212°F				
	a) Tensile Change	%	-20	Max.)	ASTM
	b) Elongation Change	%	-30	Max.)	
	c) Durometer Change	-	+10	Max.)	
D1149	Ozone Resistance 70 hours @ 100°F 20% elongation of T-50 specimen, 50 pphm in air	-	No cracks shall be apparent to the unaided eye	Min.	ASTM
D2137	Low Temperature Resistance Method A	°F	Non-brittle @-40°F after 3 min.	Max.	ASTM
D570	Water Absorption % by weight	%	2%	Max.	ASTM
D471	From Immersion in Oil #1 70 hours @ 158°F				ASTM
	a) Volume Change	%	+70	Max.)	ASTM
	b) Tensile Strength	%	-50	Max.)	
	c) Elongation	%	-40	Max.)	

Compression Deflection (psi)

<u>Method</u>	<u>Aging Properties</u>	<u>Unit</u>	<u>Requirement</u>	<u>Limit</u>	<u>Test</u>
D575	a) Unaged	%	25	Min.)	ASTM
	b) Aged in Oil #1 70 hours @ 158°F	%	-65	Max.)	
	c) Aged in Oxygen Bomb 48 hours @ 158°F, 300 psi	%	-25	Max)	

2 EPDM-based Compounds:

<u>Method</u>	<u>Property as Received</u>	<u>Unit</u>	<u>Requirement</u>	<u>Limit</u>	<u>Test</u>
D2240	Durometer Hardness	Shore A	70	±5	ASTM
D412	Tensile Strength	psi	1200	Min.	ASTM
D412	Ultimate Elongation	%	300	Min.	ASTM
D257	Volume Resistivity	ohm-cm	10 ¹²	Min.	ASTM
D395	Compression Set 70 hours @ 212°F	%	60	Max.	ASTM
D865	Oven Aging 70 hours @ 257°F				ASTM
	a) Tensile Change	%	-30	Max.)	ASTM
	b) Elongation Change	%	-50	Max.)	
	c) Durometer Change	Shore A	+15	Max.)	
D1171	Ozone Resistance 70 hours @ 104°F, 50 pphm in air	-	No cracks shall be apparent to the unaided eye	Min.)	ASTM
D2137	Low Temperature Resistance Method A	°F	Non-brittle @-40°F after 3 min.	Max.	ASTM
	Water Absorption % by Weight	%	5%	Max.	ASTM D471

Appendix D - Test Schedule

No.	Description	Dur. (Day)	Months											
			Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.
1	Test Set Up Start	0	◆											
2	Manufacturing and Delivery the test Specimens	90												
4	Manufacturing and Delivery of the Formwork	10												
5	Manufacturing and Delivery of Actuator's Connecting Plate	20												
7	Test Set Up - Concrete Mix Design	10												
8	Test Set Up - Actuator Frequency and Loading Calculations	20												
9	Concreting the Test Specimens	40												
10	Installing the Test Equipment	10												
11	Testing the Specimens	180												
12	Analyzing the Results	90												
13	Writing the Report	100												
14	Defence	1												◆

Appendix E – MTS 243-35T Hydraulic Actuator

Specifications

The specifications for the single-ended Series 243 Hydraulic Actuator models are listed in the following tables according to

their force ratings. Contact MTS Systems Corporation for information on double-ended actuator models.

Series 243 and 243T Actuator Force Rating, Piston Area

Model	Tension		Compression		Tension		Compression	
	in. ²	cm ²	in. ²	cm ²	Kip	kN	Kip	kN
243.17	2.50	16.13	4.91	31.68	7.0	31.1	13.7	61.2
243.20	5.15	33.23	8.30	53.55	14.4	64.1	23.2	103.4
243.25	7.86	49.42	12.57	81.10	21.4	95.4	35.2	156.6
243.30	12.57	81.10	19.63	126.84	35.2	156.6	55.0	244.5
243.35	18.05	120.32	28.27	182.39	52.2	232.3	79.2	352.1
243.40	22.58	145.68	38.48	248.26	63.2	281.2	107.7	479.2
243.45	34.36	221.68	50.27	324.32	96.2	427.9	140.8	626.1
243.60	50.27	324.32	78.54	506.71	140.8	626.1	219.9	978.2
243.70	74.81	481.35	113.10	729.88	208.9	929.2	318.7	1408.8
243.80	103.67	668.84	153.94	993.16	290.3	1291.1	431.0	1917.2
243.90	150.80	972.96	201.06	1297.16	422.2	1878.1	593.0	2594.1

Note: Actual force rating may vary based on varying line pressure losses

Series 243 and 243T Actuator Basic Cylinder Dimensions

Model	Rod Diameter		A		B		C		D		E		F	
	in.	mm	in.	mm	in.	mm	in.	mm	U.S. Cut.	SI Metric	in.	mm	in.	mm
243.17-01	1.75	44.5	3.00	76.2	5.50	139.7	1.50	38.1	.63-16	M16x1.5mm	1.75	44.4	3.00	76.2
243.20-01	2.00	50.8	3.50	88.9	5.75	146.1	1.75	44.5	1-14	M27x2mm	1.75	44.4	3.00	76.2
243.25-01	2.50	63.5	3.62	91.95	6.00	152.4	2.00	50.8	1-14	M27x2mm	1.75	44.4	3.00	76.2
243.30-01	3.00	76.2	4.25	108.0	6.25	158.8	2.50	63.5	1 1/2-12	M36x2mm	2.44	62.0	3.00	76.2
243.35-01	3.50	88.9	4.88	124.0	7.38	187.5	2.88	73.2	1 1/2-12	M36x2mm	2.44	62.0	3.00	76.2
243.40-01	4.50	114.3	5.38	136.7	8.50	215.9	3.00	76.2	2-12	M52x2mm	3.44	87.4	2.00	50.8
243.45-01	4.50	114.3	6.12	155.4	9.50	241.3	3.50	88.9	2-12	M52x2mm	3.44	87.4	2.00	50.8
243.60-01	6.00	152.4	8.50	215.9	12.12	307.8	4.75	120.6	3-12	M76x2mm	4.00	101.6	1.00	25.4
243.70-01	7.00	177.8	9.88	251.0	14.50	368.3	5.82	142.7	3-12	M76x2mm	4.00	101.6	1.00	25.4
243.80-01	8.00	203.2	10.38	263.7	15.62	396.7	5.88	149.4	4 1/2-8	M125x4mm	5.00	127.0	1.00	25.4
243.90-01	8.00	203.2	17.62	447.5	20.00	508.0	5.88	149.4	6-12	M150x4mm	6.00	152.4	1.00	25.4

Specifications are subject to change without notice. Contact MTS for verification of specifications critical to your needs.

Note: Series 243.xxT have a Tezosonics™ Transducer and are four inches (101.6 mm) longer than other models.

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Appendix F – Actuator Head Detail

MATERIAL:

#1 PLATE - AISI 4140

ELECTRODE - CWB #7038

(4) #1 DIA. x 1 1/2" LG. SQ. HD. BOLT and SQ. NUT (A.S.T.M. A192)

(8) #1 DIA. x 4 1/2" LG. SQ. HD. BOLT and SQ. NUT (A.S.T.M. A325)

- INSTRUCTION
1. FLAME CUT THE WHEEL 3" LARGER THAN THE REQUIRED DIMENSION AND SAW IT AFTERWARDS TO EXACT DIMENSIONS.
 2. CUT PLATES #2 TO THE EXACT DEPTH OF THE RM.
 3. CONNECT THE PLATES #2 AND RM BY MEANS OF H.T. #1 x 4 1/2" SQ. HD. BOLT and NUT (A.S.T.M. A-325).
 4. CUT and DRILL THE PLATE #1 TO THE REQUIRED DIMENSION.
 5. WELD PLATES #2 TO PLATE #1.
 6. WELD GUSSET PLATES (STIFFENERS) BETWEEN PLATES #1 and #2.

enrpld.dgn

Appendix F- Concrete Mix Design

CONCRETE TYPE	STRUCTURAL USE	MINIMUM COMP. STRENGTH @ 28 DAYS	CEMENT CONTENT M ³	NOMINAL SIZE COARSE AGG	MAX W/C RATIO	SLUMP	AIR ENTR.	CONCRETE PLACING TEMP.	REMARKS
STRUCTURAL CONCRETE	Post tensioned structures and where shown on structural drawings	35 MPa	395 kg. min. Type 10 A/E	20 mm - 5 mm	.40	75 mm ± 20 mm	6% ± 1%	27°C Max. 10°C Min.	Internal curing temperature not to exceed 43°C
STRUCTURAL CONCRETE	Sections 1 m and under in average thickness	32 MPa	330 kg min. Type 10 or 30	20 mm - 5 mm	.45	75 mm ± 20 mm	6% ± 1%	27°C Max. 10°C Min.	Internal curing temperature not to exceed 43°C
STRUCTURAL CONCRETE	Sections over 1 m min average thickness or 500 mm slab-on- grade or 500 mm wall poured against adjacent wall / excavation	32 MPa	330 kg min. Type 10	40 mm - 5 mm	.45	75 mm ± 20 mm	6% ± 1%	20°C Max. 10°C Min.	Internal curing temperature not to exceed 43°C
UTILITY CONCRETE	Maintenance Holes	32 MPa	330 kg min Type 10 or 30	20 mm - 5 mm	.50	75 mm ± 20 mm			
	Duct Banks, Pipe Bedding	20 MPa	250 kg min. Type 10 or 30	10 mm - 2.5 mm	.70	50 mm ± 10 mm			
	Fence Posts & Guide Rail Posts	20MPa							
	Lean Concrete	8 MPa	Type 10	20 mm - 5 mm	.70	75 mm ± 20 mm			
SURFACE TRACK	Foundation base (First lift)	32 MPa	330 Kg min Type 10	20 mm - 5 mm	.45	75 mm ± 20 mm			
	Track base (Second lift)	32 MPa	330 Kg min Type 10 See Note #1	20 mm - 5 mm	.45	See Note #2			
	Track infill (Third lift)	32 MPa	330 Kg min Type 10 See Note #1	20 mm - 5 mm	.45	75 mm ± 20 mm			
PAVING CONCRETE	Bus Driveway, Roadbase, Sidewalks, Curbs and Gutters	32 MPa	330 Kg min Type 10 or 30 See Note #1	20 mm - 5 mm	.45	60 mm ± 10 mm	6% ± 1%	30°C Max. 10°C Min.	Note #1: Use high early strength for bus driveway Type 30. Note #2: See paving drawings for additional details.

5

162-85-111