STRUCTURAL PERFORMANCE ANALYSIS OF A REINFORCED CONCRETE ARCH BRIDGE

by

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Chair, Department of Civil and
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Ottawa, Ontario, Canada
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ABSTRACT

An investigation has been undertaken to study the temperature load characteristics of a three span continuous arch bridge. The bridge structure studied is the Plaza Bridge, an old reinforced concrete arch bridge which has recently been rehabilitated as part of the rehabilitation project, a monitoring system has been installed on the bridge to obtain information about the behavior and performance of the bridge. A brief description of the monitoring system is presented in the thesis. Detailed parametric studies have been conducted to find the influence of the temperature and truck loading on the performance of the bridge.

A two-dimensional analytical tool has been developed to simulate the non-linear inelastic behavior of reinforced concrete structures. A special procedure is formulated to evaluate the effect of air temperature changes on the deformation behavior of the bridge. The developed two-dimensional analytical tool is used to evaluate the exact load distribution and deformation characteristics of the bridge over the entire loading range up to the point of failure under the temperature and truck loading.

The analysis results show that the bridge behaves linearly under the maximum recorded temperature as well as the maximum design temperature in the summer as specified in the bridge design code. Under the combined loading condition of truckloads and low temperature in winter, it is found that the bridge deformations may exceed the elastic limit into the nonlinear range. Although the results show that some reinforcing steel may yield, the bridge still remains below the ultimate load carrying capacity.

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I would like to remember the person who I loved the most in whole my life, my late grandmother, Khatoon, who I lost during this thesis while I was far from her.

I wish to take this opportunity to express my appreciation to two persons that I owe my life to, my parents, Shahrokh Ostovari and Tooran Missaghi for their continuous love, support, patience, encouragement, and motivation in this study and through all my life. I would like to dedicate this thesis to them.

To My Parents,

Shahrokh Ostovari and Tooran Missaghi

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Chapter 1 Introduction

1.1 Introduction

Many present-day road systems require improvement and expansion, such as the rehabilitation of existing bridges and the construction of new ones. Technologies for rehabilitation of bridges are needed because of the deterioration of old bridges and the introduction of new safety standard requirements. Among the new technologies recently developed for rehabilitation of deteriorated structures, particularly for bridges, are the development of structural health monitoring techniques and analytical procedures to obtain information and to determine more accurately the actual strength and performance of bridges. The exact load distribution and deformation characteristics over the entire loading range up to the point of failure are important for the understanding of the performance behavior of bridges. To evaluate the load carrying capacities and the performance characteristics of bridges requires analytical methods that take into account the variations in material and geometric properties of different parts of bridges under different loading conditions. In Canada, consideration of temperature loading effect is particularly important in the assessment of the performance of bridge systems because of the large temperature differences between summer and winter seasons, which can lead to potentially critical loading and stresses in some bridges.

This thesis describes the nonlinear analytical procedures developed to evaluate the load carrying characteristics and behavior of reinforced concrete bridges, including the

temperature load effects. The research includes the case study of an old reinforced concrete arch bridge recently rehabilitated in the city of Ottawa. During construction, instruments and sensors were installed by Public Works and Government Services Canada and the City of Ottawa as part of a structural health monitoring system to monitor the performance of the bridge. These instruments include strain gauges, thermocouples, corrosion sensors and moisture probes.

1.2 Background

The Plaza Bridge located in downtown Ottawa was constructed in 1912 to replace the Sapper's (1827) and Dufferin (1872) Bridges originally constructed at the same site. The heavily used Plaza Bridge is a three-span reinforced concrete arch bridge, which forms an integral part of the Confederation Square in the heart of downtown area of Ottawa. At the time of its construction, it served as a link between the upper town area, which included the Parliament Hill, and the lower town area of the Byward Market and the Rideau Canal. Although the bridge has no historical designation, throughout the years, it remained a prominent landmark in the downtown area.

Due to the heavy traffic volume and corrosion damage because of the use of de-icing salt for ice and snow removal on the roadway surface of the bridge in winter, the condition of the bridge has deteriorated over the years. Corrosion of the reinforcing steel inside the main concrete arch span members has resulted in the spalling of surface concrete and reduction of safe load carrying capacity. In 1997 the bridge was extensively renovated and rehabilitated upon its ownership transfer from Public Works and Government Services Canada (PWGSC) to the Regional Municipality of Ottawa-Carleton (RMOC).

The structural repair and rehabilitation include the removal of approximately 150mm of the outer surface concrete from the arch panels, abutments and piers, and their replacement with new concrete and reinforcing steel. The intent of the repair work is to restore the load capacity and improve the durability of the structural components of the bridge. The structural rehabilitation of the arch barrel panels with new concrete helps to prevent the penetration of moisture and other corrosive chemicals, such as de-icing salt etc., to reach the reinforcing steel to cause structural damage.

1.3 Objectives and Scope

The objectives of the work presented herein are to determine the performance characteristics of the Plaza Bridge, a reinforced concrete arch bridge, which has been repaired and renovated recently in downtown Ottawa. An effective simplified modeling and analysis tool is developed which has the capabilities to analyze linear and nonlinear structural deformation behavior due to inelastic material properties, including the simulation of the stiffness degradation or softening behavior of concrete and yielding and strain hardening of reinforcing steel. In the present study, the temperature load-deformation behavior of the Plaza Bridge are examined in detail, including the loading conditions and the corresponding structural behavior under maximum and minimum temperature gradients in the structural members during the summer and winter seasons. In this study, temperature data and analysis results are analyzed to predict the structural behavior of the Plaza Bridge. The performance and behavior of the Plaza Bridge under the maximum and minimum temperature conditions expected during summer and winter conditions are examined. Also the temperature conditions, which may affect the structure

performance, such as the yielding of the reinforcing steel in the arch members, are evaluated. The analysis results are compared with the field measured data recorded by the monitoring system installed on the bridge to verify the monitoring system.

In this research a detailed two-dimensional nonlinear finite element analysis of the effect of temperature loading on the bridge is carried out. In addition, to examine the influence of the geometry on the stress distribution of the bridge, a detailed three-dimensional finite element analysis of the bridge structure is also carried out. In order to perform the non-linear analysis a method for non-linear analysis of reinforced concrete structure is developed.

This thesis is organized into six chapters as follows:

Chapter 1 presents the introduction of the problem and the objectives and the scope of the thesis.

Chapter 2 presents the literature review relevant to the present study.

Chapter 3 describes the monitoring system and temperature data of the Plaza Bridge.

Chapter 4 explains the finite element modeling and the analysis methods developed in the present study.

Chapter 5 presents the analysis results.

Chapter 6 summarizes the results and provides conclusions for this study.

Chapter 2 Literature Review

2.1 General

Since the beginning of the 20th century, extensive analytical and experimental studies have been conducted to investigate the behavior of reinforced concrete structures.

As stated in chapter 1, the main concerns of the current study are to evaluate the behavior of the Plaza Bridge, which is a reinforced concrete arch bridge, and using the data and results from the monitoring instruments installed to predict the performance of the structure under critical loading conditions, such as extreme temperature and maximum truck loads. In this chapter, some of the previous works reported in the literature related to the research topics are briefly reviewed.

2.2 Finite Element Modeling of Reinforced Concrete Structures

Since concrete is strong in compression but relatively weak in tension, reinforcing bars are embedded inside concrete members to carry tensile stresses in order to inhibit the growth of cracks. For concrete structures subjected to a corrosive environment, the development and propagation of cracks are critical for the durability of the concrete structural members. Hence, this problem has to be considered carefully in the design of reinforced concrete structures, a composite material of reinforced concrete consisting of

brittle concrete and ductile reinforcing steel bars with very different nonlinear properties in both compression and tension. Although analytical methods are available to model the individual material components separately, these models are often very time consuming to use, and the behavior of the analyzed structure is obscured and difficult to interpret by large amount of numerical output results. Despite the advances in the computing speed and storage capacities in computers, there is still the obvious need and advantages of simplified methods of analysis of complex reinforced concrete structures. Simplified methods can give effective estimates of the load capacities and strength of the structures for design purposes.

In detailed finite element analysis of reinforced concrete structures, the steel reinforcements are usually modeled by one-dimensional bar elements. Many researchers have used this approach, but it is difficult and tedious to apply this detailed modeling approach in structures with complicated reinforcement layouts. A simpler approach is the layer model in which a reinforced concrete structural member is idealized by uniform layers of concrete and steel layer elements. In this model, the effect of the steel reinforcement, which is smeared over the entire area of the layer element is considered in the modeling of the reinforced concrete section. The area of the equivalent layer element is proportional to the amount of reinforcement in the cross-section of the structural member. Concrete layer elements are used to represent the behavior of the material in the member. These are placed adjacent to the steel layer element maintaining the same moment arm of the cross-section. This layering approach permits the monitoring of the growth and the progress of nonlinearities (e.g., cracking and crushing of concrete, yielding of reinforcement, etc.) through the thickness of the reinforced concrete cross

section. Lin and Scordelis (1975) and others (ASCE 1982) proposed the layering procedures to account for the nonlinear behavior in the steel and concrete materials of a reinforced concrete member at the element level. Although the layer modeling approach simplifies the computational process compared to the complete model, there are some limitations to this approach in that the transverse shear stresses and shear strains within the member cross-section are not properly represented. This is because variations in the transverse displacement and cross-sectional rotation are not permitted through the thickness of the section. Although in most situations the stresses mobilized in reinforced concrete structures (e.g., beams, walls, shells, etc.) are primarily normal stresses, in some special cases transverse shear stresses can be significant, while the transverse normal stresses remain negligible. Examples are in column-flat-slab connection regions, and ice loads on offshore structures. Figueras and Owen (1983) suggested some improvements to the layering approach. They proposed the use of energy-based shear correction factors in the transverse shear stiffness and shear stress computations, which resulted in an improvement in the representation of the shear stresses through the cross section. In this method they used the implicit method. In the implicit layering procedure, variations in the material properties are considered in the local finite element level. The element cross section is divided into layers of actual amount of concrete and steel. The implicit layering procedure is suitable for thick reinforced concrete section.

A more sophisticated modeling strategy was introduced by Barzegar (1989) using an explicit layering procedure. The reinforced concrete cross section is modeled by a stack of finite elements, each element composed of a single material of steel or concrete. In the

explicit modeling, the reinforced concrete cross section is broken up into separate finite elements along the material boundaries.

Ramasaeamy and Barzegar (1995) proposed a new post-cracking formulation for concrete, along with both implicit and explicit layering procedures, which are used in the analysis of RC flexural and torsional elements.

Suidan and Schnobrich (1973) presented a method of analysis for reinforced concrete members, which utilized three-dimensional, isoparametric 20-node rectangular elements with corner and mid-side nodes. Stresses and strains are evaluated at the numerical integration points. Cracking, yielding and crushing patterns are thus determined from the stress and strain evaluations at the integration points. The finite element technique in modeling of reinforced concrete structure behavior discussed in detail by Zienkiewicz (1971).

Phillips and Zienkiewicz (1976) developed an embedded representation of reinforcement in which the virtual work integration is performed along the reinforcing layer and the reinforcement is aligned with one of the local isoparametric element coordinate axes but does not pass through the nodes of the concrete element. This approach is more effective than the former two models but is still not suitable for modeling reinforcement with an arbitrary orientation. Elwi and Murray (1980) developed separately similar modeling technique as Phillips and Zienkiewicz (1976). In their presentation by Elwi and Murray (1980), the finite element mesh of the model needs not to be rectilinear. This is useful when analyzing structures with curved walls of constant thickness and regular reinforcement. Embedded representations that allow for layers placed at an angle to the local isoparametric element axes have been restricted to problems having straight

reinforcing layers and rectilinear meshes. Examples include the model incorporated in the program ABAQUS (1983). The original formulation by Phillips and Zienkiewicz (1976) was modified by Chang et al. (1987) to allow for a straight-rebar segment to be placed at any angle with respect to the local axes of an isoparametric concrete element. Balakrishnan and Murray (1986) introduced an embedded formulation with bond-slip behavior between the reinforcement and concrete.

Elwi and Hrudey (1989) have presented a formulation for a general reinforcing bar embedded in general two-dimensional elements. The bar can be oriented in an arbitrary direction and may not pass through the nodes of concrete element (i.e., independent of mesh). El-Mezaini and Citipitioglu (1991) introduced isoparametric elements with movable nodes to arrive at a more efficient formulation when bond-slip behavior is modeled.

The inherent requirement in the current embedded formulations is that the global coordinates of the intersection points of individual reinforcement and concrete elements should be provided by the analyst. While such data input in 2D models may be manageable, the tasks of identifying such points, their association with the concrete element, and preparation of data input of coordinates in 3D model are formidable. These problems together with the difficulties in calculation of stiffness of bar elements at arbitrary locations embedded in a 3D finite element mesh have led to some restrictions, simplifications, and approximations in 3D finite element analysis. Isenberg and Levine (1985) restricted the embedded bars to pass through the centers of hexahedral concrete elements. Cervara (1986) smeared a group of bars at a given location and used them as embedded layers inside the concrete-solid elements. Zeinkiewicz et al (1972) and Bhatt et

al. (1989) restricted the embedded bars to being parallel to the local-isoparametric coordinates. Barzegar and Maddipudi (1994) presented an approach which facilitated the use of embedded formulation in 3D finite element analysis of concrete structures. In this procedure, only the end point coordinates in the global axes of each straight reinforcement segment and its intersections with the faces of solid elements need to be provided by the analyst. Then by applying the principle of virtual work, the stiffness of each segment of reinforcement can be computed and added to the corresponding concrete element stiffness.

Cheng, Y.M (1995) extended the concept of Elwi and Hrudey (1989) to general three-dimensional elements with improvement and correction in the transformation process.

Chapter 3 Monitoring System and Temperature Loads

3.1 Site Description

The plaza bridge was built in 1912 to replace the two older bridges of Dufferin and Sappers Bridges. The Plaza Bridge is a three span arch bridge. It is located near the Parliamentary Precinct in downtown Ottawa next to the War Memorial, the chateau Laurier hotel, the Government Conference Centre, and the National Art Centre. The central span of the bridge crosses over the Rideau Canal, whereas the west arch is over the access roadway to the Ottawa Locks of the Rideau Canal and the Bytown Museum. At the time of its construction, the east arch spanned the railway and tramway tracks from the train station, which was converted in the 1960's to the Government Conference centre with the train track removed. The Canadian Museum of Contemporary Photography is located under the north end of this span. The east-west traffic on Wellington Street and the north-south traffic on Elgin Street converge on the Plaza Bridge.

3.2 Description of Structure

The Plaza Bridge structure is of reinforced concrete construction with spandrel filled arches. It is asymmetric with an 18.3 metre span over the canal and the west opening and a 15.4 metre span over the east opening, as shown in Figures 3-1 and 3-2. The arch vaults have a varying depth cross section, and are comprised of 14, 15, and 16 independent barrel panels in the east, west and canal arches respectively as shown in Figure 3-3. The solid arch barrels are supported on concrete piers and abutments on bedrock with the exception of the northwest foundation areas, which are founded on a varying stratum of material. The width of the arch structure at the east end is 65.2 metres and at the west end is 83.5 metres. The bridge has an asphalt roadway constructed on top of granular fill materials supported by the solid arch barrels and spandrel walls. The reinforcing of the bridge consists of cold drawn steel strand triangular-mesh woven wire fabric. The longitudinal strands consist of three 10 gauge wires twisted together and spaced at 4 inch centres. The fabric ties are individual 12 gauge wires wrapped around the longitudinal strands in a triangular wave. Two layers of fabric were installed on each face of the arch panels. The bridge abutments and piers were founded on rock and have rigid connections.

Figure 3-1 Geometry of the Plaza Bridge

Figure 3-2 Profile of the Plaza Bridge

3.3 Monitoring System of Plaza Bridge

The monitoring system of the Plaza Bridge are designed to collect information and data on the behavior and performance of the structure due to the following loading effects:

- (a) Thermal effects (temperature changes);
- (b) Live load (traffic loading).

The main objectives of the monitoring system of the Plaza Bridge are summarized as follows:

- (a) To obtain performance data of the bridge structure;
- (b) To assess the short- and long-term performance of the bridge structure;
- (c) To assist maintenance engineers in the development of a cost effective maintenance program for the bridge structure.

In order to collect data, transducers are installed at different parts of the bridge and at different depths of the bridge members. The transducers are connected and controlled by a system of data loggers, which record the voltage signals generated by the transducer and convert the signals to appropriate engineering units.

3.3.1 Measurement Instrumentations

The measurement instrumentation consists of thermocouples, strain gauges, tilt sensors and half-cell electrodes.

The transducers are installed along three 'Monitoring lines' on the Plaza Bridge structure, as illustrated in Figure 3-3. The locations of the monitoring lines and the transducers' locations are selected in order to obtain temperature and loading information of the three arches at different orientations to the solar effects along approximately the positions of the traffic lanes. The distributions of the transducers also give information about the behavior of both normal and skewed arch panels.

Figure 3-3 Plan of Plaza Bridge and Location of Monitoring Lines

Thermocouples, which measure the temperature behavior of the bridge, are installed at four different depths of concrete arch members, as shown in Figures 3-2 and 3-3. The four depths are as described follows:

- (a) Traffic surface slab at the depth of the top layer of steel reinforcing.
(TC x01 in Figure 3-5)
- (b) Top of arch member at the depth of the original steel wire fabric reinforcement. (TC x02 in Figure 3-5)
- (c) Mid-depth of the arch (TC x03 in Figure 3-5); and
- (d) Underside of the arch at the depth of the original steel wire fabric reinforcement (TC x04 in Figure 3-5).

The thermocouple sensors are numbered according to the numbering scheme shown in Figure 3-5. The thermocouple sensors are numbered in increasing order from top to bottom depth and from west to east plan locations on a monitoring line. Also in labeling the thermocouple sensors, the notation "x" refers to the line number on which the thermocouple is located. For example, TC 201 refers to the thermocouple at traffic surface on line 2.

Figure 3-4 Location of Thermocouples and Strain Gauges

**Figure 3-5 Thermocouple and Strain Gauges Locations
Through Arch Depth**

Strain gauges, which measure the deformation responses of the structure, are installed in pairs of two, at two depths through the arch barrel panels of the bridge structure at each monitoring location, as shown in Figure 3-5. The two depths are described as follows:

- (a) Top of arch surface in the longitudinal direction on the strand of the original steel wire fabric reinforcement; and
- (b) Underside of the arch in the longitudinal direction on the strand of the original steel wire fabric reinforcement.

The strain gauges are installed at the same locations of the thermocouple installations. Tilt sensors, which measure the deformations of the bridge resulting from traffic and temperature loading, are installed along monitoring line 1, as shown in Figure 3-6.

Half-cell electrodes, which measure corrosion of reinforcing steel, are installed across one of the north-bound traffic lanes on the Elgin Street, as shown in Figure 3-7.

Figure 3-6 Location of Tilt Sensors

Figure 3-7 Locations of Half-Cells

3.4 Temperature Measurements

In the following sections the sample data measured by the thermocouple sensors of the temperature monitoring system of the Plaza Bridge are presented. The temperature profiles through the depth of the concrete arch member along monitoring line 2 are obtained by interpolation of the measured data. The behavior of the Plaza Bridge in summer and winter subjected to loads resulting from temperature changes are analyzed and discussed. The rate of temperature gradient change is determined from the measured data. These measured data are used in analysis to predict the performance of the bridge under extreme temperature conditions.

The numbering scheme of the sensor locations on monitoring line 2 is shown in Figure 3-8. The numbering system is referred to in the later sections of the thesis.

Figure 3-8 Sensors Location on Line 2

3.4.1 Summer Temperature Measurements

In this study, the temperature behavior of the arch structure along monitoring line 2 is analyzed by a 2D model of the continuous arch bridge. The applied temperature loading is determined from the temperature measurements by the thermocouple sensors installed in the bridge. The temperature data recorded during the month of June and the first week of July 2001 are shown in Figures 3-8 to 3-14. The maximum recorded temperature data within the measurement period are presented in Table 3-1.

Figures 3-9 to 3-15 show the temperature measured by the thermocouple at location #1 to location #7 on the monitoring line 2, respectively. As stated earlier in the numbering scheme of the thermocouples, TC 201 shows the temperature measurement at the top near the traffic surface, TC 202 shows the temperature at the top surface of the arch, TC 203 shows the temperature at the middle of the arch, and TC 204 shows the temperature at the bottom surface of the arch. TC205 to TC208, TC209 to TC212, TC213 to TC216, TC217 to TC220, TC221 to TC224 and TC225 to TC228 give the corresponding temperature measurements at the depth locations as the group TC201 to TC204 at the sensor locations #2 to #7, respectively, as shown in Figure 3-8.

Figure 3-9 Temperature Variation Measured at Sensor Location 1

Figure 3-10 Temperature Variation Measured at Sensor Location 2

Figure 3-11 Temperature Variation Measured at Sensor Location 3

Figure 3-12 Temperature Variation Measured at Sensor Location 4

Figure 3-13 Temperature Variation Measured at Sensor Location 5

Figure 3-14 Temperature Variation Measured at Sensor Location 6

Figure 3-15 Temperature Variation Measured at Sensor Location 7

From the summer temperature data, the following observations on the behaviour of the arch can be made:

- The thermocouples (TC 206 and TC 214) at the top surface of the arch at the second and forth locations, respectively, at the mid-span of the west and centre arch have similar temperature variation behaviour over the measurement period. These two thermocouples are among the sensors embedded in the arch members the most exposed to radiation loss and gain of heat energy than the other sensors installed at deeper locations. The two thermocouples measure higher temperature than the bottom sensors at the same locations, as shown in Figures 3-10 and 3-12.
- The arch members have a uniform temperature distribution through the depth of the concrete at the first, third and the fifth locations, as they are closer to the end of the arches, as indicated by the temperature measurements showed in Figures 3-9, 3-11 and 3-13.
- The thermocouples (TC202, TC 210 and TC218) at the top surface of the arch at the first, third and the fifth locations, respectively, near the end of the arch have similar temperature variation behavior as the temperature measurements of the thermocouples TC 203, TC 211 and TC 219 at the middle depth of the arch. Because of the insulating effect from the thicker layer of crushed stone filler materials between the roadway asphalt surface and the supporting concrete arch compared to the sensors at location 2 and 4, the sensors at location 1, 3 and 5 are less exposed to radiation loss and gain of heat energy, so that they have similar temperature behavior as the sensors

located at the middle depth of the arch, as can be observed in the results shown in Figures 3-9, 3-11 and 3-15.

- By comparing the temperature measurements from the thermocouples located at the bottom with the others in the arch, it is observed that the thermocouples with thicker concrete covers are less sensitive to ambient temperature changes. The temperature measured by the bottom thermocouple changes more closely with the air temperature fluctuation as shown in Figures 3-9 to 3-15.

3.4.2 Winter Temperature

In order to find the behavior of the arch structure during winter season, a 2D model of the continuous arch bridge is analyzed to determine the temperature behavior of the arch structure along monitoring line 2. The applied temperature loading is deducted from the temperature measurements by the thermocouple sensors during the month of November 2000 to January 2001, as shown in Figures 3-16 to 3-22.

Figure 3-16 Temperature Variation Measured at Sensor Location 1

Figure 3-17 Temperature Variation Measured at Sensor Location 2

Figure 3-18 Temperature Variation Measured at Sensor Location 3

Figure 3-19 Temperature Variation Measured at Sensor Location 4

Figure 3-20 Temperature Variation Measured at Sensor Location 5

Figure 3-21 Temperature Variation Measured at Sensor Location 6

Figure 3-22 Temperature Variation Measured at Sensor Location 7

Comparing the temperature variation behavior shown in the winter temperature graphs, the following observations can be made:

- The thermocouples (TC206 and TC 214) at the top surface of the arch at the second and forth locations, respectively, at the mid-span of the west and centre arch have similar temperature variation behaviour. These sensors are more exposed to the ambient temperature than the other sensors in the deeper locations of the arch, as shown in Figures 3-17 and 3-19.
- The thermocouples show a uniform temperature through the depth of the concrete at the first, third and fifth locations, as they are closer to the end of the arches, as shown by results plotted in Figures 3-16, 3-18, 3-20.

By comparing the summer and winter temperature behaviour, it is observed that in winter the temperature measured by the thermocouples at the top and bottom surface of the arch member vary in similar behavior as the temperature variation measured by the sensors at the traffic surface. While in summer, only the thermocouples at the bottom surface of the arch give the same temperature variation as the thermocouples at the traffic surface.

3.4.3 Maximum and minimum Temperature in 2000 and 2001

By comparing the summer and winter temperature data from monitoring measurements, the maximum and minimum temperature during the period of measurement can be found. From the available data, the maximum temperature occurs on June 27th, 2001 at 13:00 p.m., and the minimum temperature is recorded on Dec 26th, 2000, at 05:00 a.m. The maximum and minimum temperature data for each of the top and bottom thermocouple sensors in the concrete arch along monitoring line 2 are shown in Tables 3-1 and 3-2.

	Location-1	Location-2	Location-3	Location-4	Location-5	Location-6	Location-7
Top	23.12	26.21	21.82	26.94	21.58	24.36	25.43
Bottom	22.92	25.59	23.71	25.9	22.55	24.76	22.92

* The unit for temperature in this table is degree C.

Table 3-1 Maximum Temperatures Measured

	Location-1	Location-2	Location-3	Location-4	Location-5	Location-6	Location-7
Top	-4.70	-10.00	-4.00	-10.30	-1.30	-5.30	-5.50
Bottom	-10.10	-13.20	-11.50	-13.10	-5.00	-5.60	-5.30

* The unit for temperature in this table is degree C.

Table 3-2 Minimum Temperatures Measured

The temperature data at top and bottom surface of the arch measured by the monitoring system are later used as input data for further analysis of the temperature behavior and performances of the Plaza Bridge in this study. The temperature profile at a cross-section of the arch member is assumed to have a linear variation through the depth of the member, which can be expressed as the combination of a uniform temperature change and a linear temperature gradient change. The uniform temperature change is determined as the average of the top surface and the underside temperature. The temperature gradient is determined from the difference between the temperature at the top surface of the arch and the underside of the arch.

3.4.4 Temperature Effect

There are many factors which can affect the temperature behaviour of concrete structures, such as:

- Ambient air temperature
- Time of day
- Cloud height
- Sun position
- Wind
- Humidity
- Traffic over the surface

- Exposure

All of the above factors can affect the temperature of the structure and the rate of change in the temperature of the structure. Among the various factors of influence, the ambient air temperature may be considered to have the more significant influence. In order to predict the structural behaviour under extreme temperature conditions, the possible maximum and minimum temperature distribution of the bridge is estimated from the measured data. The slopes or rates of change in temperature at the top and bottom surface of the arch are calculated. By examining the temperature data plotted, the temperature data, which are assumed to change linearly with a constant slope over a period of time, are used to determine the rate of change in temperature. The different thermocouple sensors may change with different slopes. The consistency of the slope determined for the different sensors shows that the temperature data are not significantly affected by other influence factors listed above. Having the slopes or the rates of change of the top and bottom gauges of the arch and the air temperature, the extreme temperature gradient can be estimated for each sensor corresponding to the extreme temperature conditions specified in the design code.

The trend graphs of the sensors located at the monitoring line 2, as shown in Figures 3-23 to 3-29, which show the temperature data during the first week of December 2000, are used to find the extreme temperature conditions.

Figure 3-23 Temperature Variation Measured at Sensor and Air Temperature at Location 1

Figure 3-24 Temperature Variation Measured at Sensor and Air Temperature at Location 2

Figure 3-25 Temperature Variation Measured at Sensor and Air Temperature at Location 3

Figure 3-26 Temperature Variation Measured at Sensor and Air Temperature at Location 4

Figure 3-27 Temperature Variation Measured at Sensor and Air Temperature at Location 5

Figure 3-28 Temperature Variation Measured at Sensor and Air Temperature at Location 6

Figure 3-29 Temperature Variation Measured at Sensor and Air Temperature at Location 7

To find the rate of temperature change in each sensor, a period of time which has a constant rate of change in temperature is chosen. The observation shows that the period of time between 12/1/2000 at 19:00 and 12/2/2000 at 6:00 has constant rates of change in temperature for the sensors at different depths of the arch. During this period of a 4.7 °C air temperature drop, the temperature drops at the top and bottom surface of the arch are summarized in Table 3-3.

	Location-1	Location-2	Location-3	Location-4	Location-5	Location-6	Location-7
Top	-0.126	-0.252	-0.077	-0.192	-0.092	-0.463	-0.075
Bottom	-0.048	-1.179	-1.225	-1.115	-0.459	-0.356	-0.326

* The unit for temperature in this table is degree C.

Table 3-3 Temperature slopes for Gauges

With the rate of change in temperature determined for each sensor, the temperature at the top and bottom surface of the arch corresponding to the extreme air temperature conditions as specified by the Ontario Highway Bridge Design Code (OHBD) during the service life of the bridge can be found.

According to OHBD, the minimum effective temperature for type C structures (Type C superstructure include concrete system with concrete) is 5 °C below minimum daily mean temperature. Consequently as, minimum daily mean temperature for Ottawa is -35 °C, therefore the effective temperature is equal to -40 °C. The minimum temperature

at the top and bottom surface of the arch members expected during the service life of the bridge corresponding to the minimum air temperature (-40°C) are shown in Table 3-4:

	Location-1	Location-2	Location-3	Location-4	Location-5	Location-6	Location-7
Top	-5.45	-11.49	-14.06	-11.44	-1.84	-8.04	-5.94
Bottom	-16.30	-20.18	-18.74	-19.69	-7.71	-7.70	-7.23

Table 3-4 Minimum Bridge Temperature

The same procedure is used to find the expected maximum temperature during the service life of the bridge. However because the air temperature data was not available for the month of June, the rates of change in temperature for the sensors determined from the winter temperature data are used for finding the maximum temperature in summer. According to OHBDC, the maximum daily temperature mean temperature for Ottawa is 30°C , so the effective temperature is 40°C which is for type C structure is 10°C above the maximum daily mean temperature as specified. However for the present study, the maximum temperature measurements recorded on June 27th, 2001 are used to find the performance of the Plaza Bridge under the maximum air temperature. The bridge is analyzed by subjecting to the temperature loads as determined from the maximum temperature measured on June 27th, 2001, shown in Table 3-5.

	Location-1	Location-2	Location-3	Location-4	Location-5	Location-6	Location-7
Top	23.10	26.20	21.80	26.90	21.60	24.40	25.40
Bottom	22.90	25.60	23.70	25.90	22.50	24.80	22.90

Table 3-5 Maximum Bridge Temperature

Chapter 4 Finite Element Modeling and Numerical Examples

4.1 General

It has been mentioned at the beginning of this thesis that both linear and non-linear finite element techniques are employed in the present study to evaluate the behavior of the Plaza Bridge. This chapter describes the finite element techniques and the equivalent model developed for non-linear analysis of reinforced concrete structures.

4.2 Finite Element Model

In the 2D finite element analysis of the Plaza Bridge, two-dimensional beam elements are used to model the arch bridge girders. The 2D beam element has three degrees-of-freedom (two translations and one rotation) per node in the analysis model. In non-linear analysis the loading process is divided into load increments. The modified Newton-Raphson iterative method is employed in the nonlinear analysis. The numerical integration is carried out by Gauss Quadrature integration. In the Gauss Quadrature, the stresses at 5 points of the section are calculated, where the top and bottom points are almost at the same level of the reinforcing bars.

4.3 Equivalent Model

A simplified equivalent model, which accounts for the non-linear material behaviour of reinforcing steel and concrete under compression, has been developed in the present study. This model is used for non-linear frame analysis of reinforced concrete structures with beam-column members. The equivalent model can be applied to represent the non-linear structural behavior of reinforced concrete structures up to failure. The equivalent model is developed based on specifying an equivalent stress-strain relation for the section of the reinforced concrete structural member, which gives the same moment-curvature behaviour as the original reinforced concrete section. In this way, the equivalent structural member behaves the same as the reinforced concrete section in non-linear analysis. To derive the equivalent stress-strain relationship, the cracking moment, the yielding moment and the ultimate moment of the reinforced concrete member are determined. In the derivation of the equivalent model, the model developed by Kent and Park (1982) is adopted for the modeling of the concrete material. The formulations of the stress-strain relations of confined and unconfined concrete by the Kent and Park model (1982) are briefly described in this section.

Figure 4-1 Concrete Material Model

Figure 4-1 shows the concrete material model derived by Kent and Park (1982) for confined and unconfined concrete. The model consists of two parts: a curvilinear part followed by a linear part. When the strain is less than ε_0 in unconfined concrete or less than $k\varepsilon_0$ in confined concrete, the stress value is determined by Equation 4.1 in the curvilinear part.

$$f_c = k f'_c \left[\frac{2 \varepsilon_c}{\varepsilon_0 k} - \left(\frac{\varepsilon_c}{\varepsilon_0 k} \right)^2 \right] \quad \varepsilon_c \leq k \varepsilon_0 \quad (4.1)$$

where f_c is the longitudinal concrete stress, ε_c is the longitudinal concrete strain, and k is a coefficient obtained from Equation 4.2.

$$k = 1 + \frac{\rho f_{yh}}{f'_c} \quad (4.2)$$

where ρ is the ratio of the volume of hoop reinforcement to the volume of concrete core measured to the outside of the hoops, f_{yh} is the yield stress for the hoop reinforcement. The coefficient k has a unit value for unconfined concrete. For the strains more than ϵ_0 in unconfined concrete or more than $k\epsilon_0$ in confined concrete, the longitudinal concrete stress behavior falls within the linear part of the stress-strain relationship, as given by Equation 4.3.

$$f_c = k f'_c [1 - Z_m (\epsilon_c - \epsilon_0 k)] \geq 0.2 k f'_c \quad \epsilon_c > k \epsilon_0 \quad (4.3)$$

In Equation 4.3, Z_m is determined by equation 4.4

$$Z_m = \frac{0.5}{\epsilon_{50u} + \epsilon_{50h} - \epsilon_0 k} \quad (4.4)$$

ϵ_{50u} and ϵ_{50h} is derived from Equations 4.5 and 4.6 as follows:

$$\epsilon_{50u} = \frac{3 + \epsilon_0 f'_c}{f'_c - 1000} \quad (4.5)$$

$$\epsilon_{50h} = 0.75 \rho \sqrt{h/s} \quad (4.6)$$

In Equations 4.5 and 4.6, h is the width of the concrete core measured to the outside of the hoops, and s is the centre-to-centre spacing of the hoops. In the concrete material model, the tensile strength of concrete is ignored.

In the derivation of the equivalent model here, the reinforcing bars are assumed to behave in an elasto-plastic manner. A bilinear stress-strain relationship is assumed for the

reinforcing steel, as shown in Figure 4-2. The material parameters, F_y , F_u , E_s and ϵ_u , the yield stress, the ultimate stress, the Young's modulus and the ultimate strain, respectively are required as inputs in the definition of the bilinear model for the reinforcing steel. In the bilinear model, the stress increases with the constant stiffness E_s up to the yield stress F_y . After yielding, a linear strain-hardening behavior is assumed up to the ultimate stress at the level of $2F_y$ with a corresponding ultimate strain at fracture of ϵ_u .

Figure 4-2 Bilinear Stress-Strain Relationship for Reinforcing Steel

In the analysis using the equivalent model, the reinforced concrete section is replaced by an equivalent section made of nonlinear material with the derived stress-strain curve. The objective in the formulation of the equivalent model is to have the same moment-curvature behavior for the equivalent section as the original reinforced concrete section of the beam member. The derivation of the required stress-strain curve, which results in the

same moment-curvature relationship as the reinforced concrete member cross-section, is presented herein.

In the derivation of the stress-strain relationship of the equivalent section, first the cracking curvature is determined by finding the neutral axis depth c , as shown in Figure 4-3, at the first cracking of concrete under tension as given by Equation 4.7

Figure 4-3 Cracked Section of Reinforced Concrete Member

$$(b c) \left(\frac{c}{2} \right) + (c - d') (n - 1) A_{s'} - (d - c) n A_s = 0 \quad (4.7)$$

In Equation 4.7, b is the section width, c is the neutral axis depth, d' is the cover thickness, n is the ratio of steel modulus of elasticity to concrete modulus of elasticity, $A_{s'}$ is the area of the compression steel bars, A_s is the area of the tension steel bars, and d is the section depth.

The cracking strain for the actual reinforced concrete can be found from the cracking stress of concrete σ_{cr} and the modulus of elasticity of concrete E by Equation 4.8.

$$\varepsilon_{cr} = \frac{\sigma_{cr}}{E} \quad (4.8)$$

where the cracking stress of concrete σ_{cr} and the modulus of elasticity of concrete E are given by Equations 4.9 and 4.10.

$$\sigma_{cr} = 0.6 \sqrt{f'_c} \quad (4.9)$$

$$E = 4500 \sqrt{f'_c} \quad (4.10)$$

In Equations 4.9 and 4.10, f'_c is the concrete 28 days strength.

Knowing the neutral axis depth and the cracking strain, the curvature of the section as shown in Figure 4-4(b) can be found by Equation 4.11.

$$\phi_{cr} = \frac{\varepsilon_{cr}}{h - c} \quad (4.11)$$

where, h is the section height.

As stated earlier, the equivalent model should have the same moment-curvature behavior as the reinforced concrete section. Assuming uniform homogeneous material in the equivalent model (Figure 4.4d), the behavior of cracking in the equivalent section can be simulated by considering the same curvature and shifting the neutral axis to the middle height of the section. Figures 4.4 (e) and (f) show the equivalent cracking stress $\sigma_{cr(eq)}$

and cracking strain $\varepsilon_{cr(eq)}$. The equivalent stress $\sigma_{cr(eq)}$ and strain $\varepsilon_{cr(eq)}$ of the uniform equivalent section, which give the same cracking curvature in the representation of the cracking behavior of the original reinforced section, can be calculated by Equations 4.12 and 4.13.

$$\varepsilon_{cr(eq)} = \phi_{cr} \left(\frac{h}{2} \right) = \left(\frac{\varepsilon_{cr}}{c} \right) \left(\frac{h}{2} \right) \quad (4.12)$$

$$\sigma_{cr(eq)} = E \varepsilon_{cr(eq)} \quad (4.13)$$

Figure 4-4 (a) Reinforced Concrete Section (b) Cracking Strain (c) Cracking Stress (d) Equivalent Model Section (e) Equivalent Cracking Strain (f) Equivalent Cracking Stress

To simulate the yielding behavior in the original reinforced concrete section by the equivalent model, the yielding moment and curvature in the reinforced concrete section are determined. Figure 4-5 shows a reinforced concrete section where the bottom

reinforcement bars reach yielding. A trial and error procedure is followed to find the neutral axis depth by equilibrium of the forces in the reinforcement bars and in the concrete.

Figure 4-5 (a) Reinforced Concrete Section (b) Strain in the Section (c) Stress in the Section

Figure 4-5 shows the strain profile and the corresponding internal forces of the compressive concrete and the compressive and tensile steel. The compressive force in the concrete is determined by Equation 4.14.

$$C_c = \alpha f'_c b c \quad (4.14)$$

where f'_c is the 28 days strength of the concrete, b is the section width, c is the neutral axis depth and α is a coefficient which gives the equivalent compressive force of the rectangular block area as the area of the curved area in the concrete. As the equivalent

force of the curved area shown in Figure 4-5 (c), α is determined by Equation 4.15

$$\alpha = \frac{\varepsilon_{cc}}{\varepsilon_0} \left[1 - \frac{\varepsilon_{cc}}{3 \varepsilon_0} \right] \quad (4.15)$$

In the equation above, ε_{cc} is the strain at the top fibre of the concrete section, and ε_0 is the corresponding strain of the maximum stress in the concrete model shown in Figure 4-1. Another internal force in the section is the force in the compressive steel bars. This is shown in Figure 4-5 (c). The force is determined by Equation 4.16

$$C_s = f_s A_s = (E_s \times \varepsilon_s) A_s \quad (4.16)$$

In Equation 4.16, E_s is the steel modulus of elasticity, A_s is the area of the compressive steel bars, and ε_s is the strain in compressive steel bars. In the simulation of the yielding behavior of the reinforced concrete section, ε_s and ε_{cc} in Equation 4.15 are derived by obtaining the yielding strain in the tensile steel bars using Equations 4.17 and 4.18.

$$\frac{\varepsilon_s}{d - c} = \frac{\varepsilon_{cc}}{c} \quad (4.17)$$

$$\frac{\varepsilon_s}{d - c} = \frac{\varepsilon_s'}{c - d'} \quad (4.18)$$

The force in the tensile steel bars is determined by Equation 4.19

$$T_s = f_s A_s \quad (4.19)$$

At the yield condition of reinforcing steel, the stress in the tensile steel bars, f_s , is equal to the yield stress f_y .

A trial-and-error procedure to determine the neutral axis, c , by equilibrium of the internal forces is carried out for the reinforced concrete section. The yielding moment of the section is determined by Equation 4.20.

$$M_y = C_c (d - \gamma c) + C_s (d - d') \quad (4.20)$$

where the coefficient γ , shown in Figure 4-5©, is found by Equation 4.21.

$$\gamma = 1 - \frac{\frac{2}{3} - \frac{\epsilon_{cc}}{4 \epsilon_0}}{1 - \frac{\epsilon_{cc}}{3 \epsilon_0}} \quad (4.21)$$

This yield moment is used to find the corresponding stress at the outer fibre of the concrete under compression in the equivalent section later in this section.

Knowing the neutral axis depth c and the strain at the top fibre of the reinforced concrete section, the yielding curvature of the reinforced concrete section, ϕ_y , is determined by Equation 4.22

$$\phi_y = \frac{\epsilon_{cc}}{c} \quad (4.22)$$

By equating the yielding curvature in the reinforced concrete section, the equivalent yield strain of the equivalent model section can be found by Equation 4.23 by taking the neutral axis at the middle of the section.

$$\epsilon_{eq(y)} = \phi_y \left(\frac{h}{2} \right) \quad (4.23)$$

Once the yielding moment and equivalent yielding strain are found, the equivalent yielding stress in the section can also be determined. Figure 4-6 shows the strain profile in the reinforced concrete section and the equivalent strain and stress distributions in the equivalent model. In the equivalent model, the homogeneous material is assumed to have the same stress-strain behavior in both tension and compression as it can be seen in Figure 4-6(e).

Figure 4-6 (a) Yielding of Reinforced Concrete Section (b) Strain in the Section (c) Equivalent model Section (d) Equivalent Yield Strain in the Equivalent Model Section (e) Equivalent Yield Stress in Equivalent model.

Corresponding to the equivalent strain distribution shown in Figure 4-6(d), the equivalent stress distribution of the equivalent model is shown in Figure 4-6(e). From the equivalent stress distribution, the yield moment can be expressed as follows:

$$M_y = \frac{1}{2} \sigma_{cr(eq)} d_1 \left(\frac{2}{3} \right) d_1 (2) + \sigma_{cr(eq)} (d_2) \left(\frac{h}{2} - \frac{d_2}{2} \right) (2) + \Delta \sigma (d_2) \left(\frac{1}{2} \right) \left(\frac{h}{2} - \frac{d_2}{3} \right) (2) \quad (4.24)$$

By equating the equivalent yielding strain and having the equivalent cracking strain in the equivalent model, d_1 and d_2 in Equation 4.24 are found as follows:

$$\frac{\epsilon_{y(eq)}}{\frac{h}{2}} = \frac{\epsilon_{cr(eq)}}{d_1} \quad (4.25)$$

$$d_2 = \frac{h}{2} - d_1 \quad (4.26)$$

Equation 4.24 is then solved to obtain the stress $\Delta \sigma$, which is the difference between the cracking stress and the yielding stress. Consequently, the equivalent yield stress $\sigma_{y(eq)}$ is determined by Equation 4.27.

$$\sigma_{y(eq)} = \sigma_{cr(eq)} + \Delta \sigma \quad (4.27)$$

At the ultimate condition, the top fibre of the compressive concrete in the reinforced concrete section reaches the ultimate strain. The same procedure employed in finding the equivalent yield stress is used again to find the equivalent ultimate stress and strain for the equivalent section. First the curvature and ultimate moment in the reinforced concrete section are found. Figure 4-7 shows a reinforced concrete section in ultimate state. The

neutral axis depth is found from the equilibrium equation of the internal forces in the section.

The compression force in the concrete can be found from Equation 4.14 by taking the strain at the top fibre of the concrete as the ultimate strain. The parameter α , which determines the concrete compressive force in the equivalent section, is derived from the area under both the curvilinear and linear parts of the stress-strain curve in Figure 4-1, as given by Equation 4.28.

$$\alpha = \frac{\int_0^{\epsilon_0} f_c d\epsilon + \int_{\epsilon_0}^{\epsilon_{cu}} f_c d\epsilon}{f'_c \epsilon_{cu}} \quad (4.28)$$

Figure 4-7 (a) Reinforced Concrete section (b) Strain the Section at the Ultimate State (c) Stress and Internal Forces in the Section at the Ultimate State

The forces in the compressive and tensile steel bars, are determined by Equations 4.16 and 4.19, respectively.

Following the derivations of the internal forces in the concrete and steel bars, the same trial-and-error procedure as employed earlier in the derivation of the yielding state is applied to find the neutral axis depth in the reinforced concrete section under the ultimate state. The ultimate moment of the section is determined by Equation 4.20. However in this case, the coefficient γ is found by integrating the area under the curvilinear and linear parts in Figure 4-1, as indicated in Equation 4.29. The ultimate moment is then used to find the ultimate stress at the outer fibre of concrete under compression in the equivalent section.

$$\gamma = 1 - \frac{\int_0^{\epsilon_0} f_c \epsilon d\epsilon + \int_0^{\epsilon_{cu}} f_c \epsilon d\epsilon}{\epsilon_{cu} \left(\int_0^{\epsilon_0} f_c d\epsilon + \int_0^{\epsilon_{cu}} f_c d\epsilon \right)} \quad (4.29)$$

Then knowing the neutral axis depth c and the ultimate strain in the top fibre of the concrete section, the ultimate curvature in the reinforced concrete section is found by Equation 4.30.

$$\phi_u = \frac{\epsilon_{cu}}{c} \quad (4.30)$$

After finding the ultimate curvature in the reinforced concrete section and by equating the curvature of the equivalent model section to the curvature of the reinforced concrete section, the equivalent ultimate strain in the equivalent model section, as shown in Figure 4-8, is determined by Equation 4.31

Figure 4-8(a) Reinforced concrete Section in Ultimate State (b) Strain in the Section (c) Equivalent Model Section (d) Equivalent Strain in Equivalent Model Section (e) Equivalent Ultimate Stress in Equivalent Model.

$$\epsilon_{eq(u)} = \phi_u \left(\frac{h}{2} \right) \quad (4.31)$$

Corresponding to the equivalent strain distribution shown in Figure 4-8(d), the stress distribution of the equivalent model is shown in Figure 4-8(e). From the equivalent stress distribution, the ultimate moment can be found as follows:

$$\begin{aligned} M_u = & \frac{1}{2} \sigma_{cr(eq)} d_1 \left(\frac{2}{3} \right) d_1 (2) + \sigma_{cr(eq)} (d_2) \left(d_1 + \frac{d_2}{2} \right) (2) \\ & + (\sigma_{y(eq)} - \sigma_{cr(eq)}) (d_2) \left(\frac{1}{2} \right) \times \left[\left(\frac{2}{3} \right) (d_1 + d_2) (2) \right. \\ & \left. + \sigma_{y(eq)} (d_3) \left(\frac{h}{2} - \frac{d_3}{2} \right) (2) + \frac{1}{2} \Delta \sigma (d_3) \left(\frac{h}{2} - \frac{d_3}{3} \right) (2) \right] \end{aligned} \quad (4.32)$$

By equating the equivalent ultimate strain and having the equivalent cracking and yielding strain in the equivalent model, d_1 , d_2 and d_3 in Equation 4.32 are found by Equations 4.33 and 4.34.

$$d_2 = d - d_1 \quad (4.33)$$

$$d_3 = \frac{h}{2} - d \quad (4.34)$$

Equation 4.31 can then be solved to obtain the stress $\Delta\sigma$, which is the difference between the ultimate stress and the yielding stress. Consequently, the equivalent ultimate stress $\sigma_{y(eq)}$ can be determined by Equation 4.35.

$$\sigma_{u(eq)} = \sigma_{y(eq)} + \Delta\sigma \quad (4.35)$$

The derived equivalent model can be used to analyze the non-linear behavior of frame structures consisting of beams, columns and beam-column members. In analyzing columns and beam-columns structures, the same procedure as beam element is employed to find the equivalent stress-strain curve. However the influence of axial force on the inelastic behavior of the member has to be taken into consideration. The axial forces in column and beam-column members in a frame may be obtained from a linear finite element analysis. The resulting axial forces are then considered in the derivation of the equivalent stress-strain relationship by including the axial force in the equilibrium calculations of the internal forces in the reinforced concrete section.

In the following section two numerical examples are presented to demonstrate the analysis procedure using the equivalent model. The first example is a cantilever reinforced concrete beam section. The second example is an arch with axial force in the member.

4.4 The Example of Cantilever Beam

Figure 4-9 shows a cantilever beam subjected to a moment applied at the free end of the beam. The dimensions and material properties of the beam are shown in the figure. In the analysis, the beam is divided into ten finite elements along the span.

Figure 4-9 Cantilever Beam

An equivalent model section of non-linear material with the derived stress-strain curve is adopted to model the inelastic behavior of the cantilever reinforced concrete beam. To derive the equivalent stress-strain curve of the equivalent model, the cracking moment, the yielding moment and the ultimate moment of the reinforced concrete section are found. The curvatures at the cracking, yielding and ultimate stages are calculated using Equations 4.7 to 4.11, 4.14 to 4.22 and 4.28 to 4.30, respectively. Maintaining the same moment-curvature behavior, the corresponding equivalent strains for the equivalent model are determined using Equations 4.12, 4.23, and 4.31. The results of the equivalent stress-strain curve is tabulated in Table 4-1.

	Cracking	Yielding	Ultimate
ϕ	0.0002244	0.00381	0.0249
ϵ_{eq}	0.0000897	0.0015	0.010

Table 4-1 Cracking, Yielding and Ultimate Curvature and Strain of the Equivalent Model

After finding the curvatures and the equivalent strains of the equivalent model section, the equivalent cracking stress, the equivalent yielding stress and the equivalent ultimate stress of the equivalent model can be found by using equations 4.13, 4.24 to 4.27 and 4.32 to 4.35, respectively. The results are shown in Table 4-2.

Stress	Cracking	Yielding	Ultimate
σ (MPa)	2.2109	14.21	23.27

Table 4-2 Cracking, Yielding and Ultimate Stress of the Equivalent Model

The resulting equivalent stress-strain curve for the reinforced concrete section of the cantilever beam is plotted in Figure 4-10.

Figure 4-10 Equivalent Stress-Strain Curve

In this example, a concentrated moment equal to 1600 kN-m is applied at the end point of the cantilever beam. The cantilever beam is analyzed using a finite element analysis program. The resulting stress, strain and displacement at the end point of cantilever is summarized in Table 4-3.

	σ (MPa)	ϵ	Displacement (cm)
End point of the cantilever beam	13.18	0.0014	19.00

Table 4-3 Resulting Stress, Strain and Displacement in the Cantilever Beam

The resulting stress and strain is shown in Figure 4-10.

4.5 Arch Beam

Figure 4-11(a) shows an arch beam subjected to a concentrated force at the mid-span of the beam. The dimensions and material properties of the beam are shown in the figure. In the finite element model, the beam is divided into ten elements along the span, as shown in Figure 4.11 (b). A linear finite element analysis is first carried out to find the axial force in the arch. In the analysis of the beam-column section, the equilibrium of the resultant forces is expressed by Equation 4.36

$$C_c + C_s = T_s + P \quad (4.36)$$

Figure 4-11(a) Arch Beam Subjected to Axial Force

Figure 4.11 (b) Finite Element Mesh

A linear analysis of the arch shows that the axial force in the arch is equal to 160 kN. Using Equations 4.7 to 4.12, 4.14 to 4.23, and 4.28 to 4.31 the curvatures and the equivalent strains at cracking, yielding and ultimate states can be determined. The results are tabulated in Table 4-4.

	Cracking	Yielding	Ultimate
ϕ	0.0002244	0.00384	0.0249
ϵ_{eq}	0.0000897	0.0015	0.010

Table 4-4 Cracking, Yielding and Ultimate Curvatures and Strains of the Equivalent Model.

After finding the equivalent strain in the section, the equivalent stresses in the section are found using the method derived earlier. The equivalent cracking stress, the equivalent yielding stress and the equivalent ultimate stress of the equivalent model are determined

by Equation 4.13, 4.24 to 4.27, and 4.32 to 4.35, respectively. The results are shown in Table 4-5.

	Cracking	Yielding	Ultimate
σ (MPa)	2.2109	14.21	23.27

Table 4-5 Cracking, Yielding and Ultimate Stress of the Equivalent Model

After finding the equivalent stresses and strains in the equivalent section, a non-linear finite element analysis is performed using the equivalent stress-strain curve. The resulting stress, strain and displacement for the mid-span is summarized in Table 4-6.

	σ (MPa)	ϵ	Displacement (cm)
Mid-span of the arch	4.28	0.00033	1.65

Table 4-6 Resulting Stress, Strain and Displacement in the Arch Beam

Chapter 5 Plaza Bridge Modeling and Results

5.1 Plaza Bridge Modeling

To find out the behavior of the Plaza Bridge under temperature and truck loading, two-dimensional linear and non-linear and three-dimensional linear analysis are carried out in this study. Beam elements are used in the two-dimensional analysis of the bridge. The arch bridge is divided into 76 segments. Each segment is modeled by 5 straight beam elements. The two-dimensional analysis model of the bridge has a total of 380 elements. The equivalent model derived in Chapter 4 is employed to determine the nonlinear inelastic behavior of the Plaza Bridge. In the three-dimensional analysis model, Shell layer elements are used to model the arch structure.

5.2 Material Properties

At the time of the construction of the Plaza Bridge, the standard concrete mix practice of 1:3:6 (proportions of volume of cement, sand and coarse aggregates) was followed in the construction of the mass foundations, piers and abutments. The structural arch barrels were constructed of concrete with a mix ratio of 1:2:4.

The results of coring concrete tests (Dillon 1980) show an average strength of 30 MPa for the arch concrete and 19.5 MPa for all substructures concrete. For the linear analysis in this study, the average strength of 30 MPa is assumed for the concrete.

According to OHBDC design code the coefficient of linear thermal expansion for steel is taken as $12 \times 10^{-6} / ^\circ\text{C}$. In the absence of more precise information, the coefficient of linear thermal expansion for normal concrete is taken as $12 \times 10^{-6} / ^\circ\text{C}$.

Distributed dead loads calculated from the weights of the concrete arch barrels with a mass density of 2400 kg/m^3 and the spandrel granular fill with a mass density of 2200 kg/m^3 are applied to the analysis model.

In addition, vehicle live loads are included in the analysis, which are taken from the standard OHBDC design code truck with five axels and a total gross weight of 720 kN.

5.3 Temperature Behavior

To illustrate the effect of temperature loading on the behavior of the bridge, the variations of the stress and strain under increasing temperature load are plotted at the strain gauge locations of the monitoring system and also at other selected points of the bridge. The bridge is analyzed for the maximum summer temperature loading and the minimum winter temperature recorded by the monitoring system. In addition, the bridge subjected to the maximum and minimum code design temperature as specified in the OHBDC design code. Two-dimensional linear and nonlinear and three-dimensional linear analyses of the bridge are carried out.

5.3.1 Maximum Summer Temperature Behavior

Two-dimensional linear and non-linear analyses of the behavior of the bridge along the monitoring line two has been carried out. The maximum temperature recorded by the monitoring system in 2001 and the maximum code design temperature according to the OHBDC design code are used as input loading in the analysis. The maximum recorded summer temperature occurred on June 27th, 2001 at 13:00 p.m. Before carrying out the non-linear analysis, a linear analysis is performed to find the axial forces in the arch member sections. The resulting forces are used to derive the equivalent stress-strain curves as discussed in Chapter 4. The derived stress-strain relationships are used in the equivalent model for the non-linear analysis of the inelastic behavior of the bridge. The cracking and yielding stresses and strains, calculated at selected joints, are shown in

Table 5-1. The selected joints in Table 5-1 are shown in Figure 5-1. The calculated moments, stresses and strains along the bridge arch from the non-linear analysis are shown in Figures 5-2 to 5-6. The results show that the stress at almost all the points are below cracking and the arch bridge has a linear behavior.

	Strain-Cr	Stress-Cr	M (KN.m)	$\epsilon(\text{eq})(\text{yield})$	Stress (y)	M (U)	$\epsilon(\text{eq})\text{ultimate}$	Stress(U)
Guage1	0.0000846	2.085	3.20E+03	1.38E-03	9.15E+00	6.38E+03	1.03E-02	1.59E+01
Guage2	0.0000919	2.266	1.33E+03	1.68E-03	1.82E+01	2.18E+03	8.11E-03	2.28E+01
Node101	0.0000831	2.05	3.94E+03	1.34E-03	7.58E+00	7.66E+03	9.02E-03	1.30E+01
Node140	0.0000859	2.116	2.70E+03	1.42E-03	1.05E+01	5.41E+03	1.13E-02	1.79E+01
Guage3	0.0000837	2.064	3.60E+03	1.37E-03	8.29E+00	7.13E+03	1.14E-02	1.44E+01
Guage4	0.0000919	2.266	1.32E+03	1.67E-03	1.81E+01	2.39E+03	9.21E-03	2.61E+01
Node241	0.0000862	2.126	2.57E+03	1.42E-03	1.09E+01	5.12E+03	1.25E-02	1.83E+01
Guage5	0.0000847	2.087	3.11E+03	1.32E-03	9.04E+00	6.35E+03	1.02E-02	1.62E+01
Guage6	0.0000919	2.266	1.34E+03	1.67E-03	1.84E+01	2.38E+03	9.02E-03	2.57E+01
Guage7	0.0000837	2.063	3.66E+03	1.35E-03	8.38E+00	7.23E+03	1.06E-02	1.45E+01

Table 5-1 Moments, Stresses and Strains for Selected Joints

Figure 5-3 Non-linear Top Stress under Maximum Summer Temperature

Figure 5-4 Non-linear Bottom Stress under Maximum Summer Temperature

Figure 5-5 Non-linear Top Strain under Maximum Summer Temperature

Figure 5-6 Non-linear Bottom Strain under Maximum Summer Temperature

5.3.2 Maximum Code Design Temperature Behavior

According to the OHBDC design code the maximum effective temperature for a concrete system bridge should be taken 10°C above the maximum daily mean temp. For Ottawa the maximum daily mean temperature is 30°C . Therefore the bridge is analyzed for a maximum code design temperature of 40°C air temperature. A 2D non-linear analysis is carried out for this maximum temperature. The results are shown in Figures 5-7 to 5-11. The results show that even for the maximum code design temperature, the behavior of the bridge remains linear.

Figure 5-7 Non-linear Moment for Maximum Code Design Temperature

Figure 5-8 Non-linear Top Stress for Maximum Code Design Temperature

Figure 5-9 Non-linear Bottom Stress for Maximum Code Design Temperature

Figure 5-10 Non-linear Top Strain for Maximum Code Design Temperature

Figure 5-11 Non-linear Bottom Strain for Maximum Code Design Temperature

5.3.3 Minimum Winter Temperature Behavior

Linear and non-linear 2D temperature analyses of the behavior of the bridge subjected to the maximum recorded winter temperature have been carried out. The minimum recorded winter temperature occurred on Dec. 26th, 2000 at 05:00 p.m. Before carrying out the non-linear analysis, a linear analysis is performed to find the axial forces in the arch member sections. The resulting axial forces are used to derive the equivalent stress-strain curves as discussed in Chapter 4. The derived stress-strain relationships are used in the equivalent model for the non-linear analysis of the inelastic behavior of the bridge. As stated before, the finite element analysis model of the Plaza Bridge is divided into 76 sections. For each section, the cracking and yielding stresses and strains are found as explained in Chapter 4. The cracking and yielding stress and strains at selected joints are shown in Table 5-2.

	Strain-Cr	Stress-Cr	M (KN.m)	$\epsilon(\text{eq})(\text{yield})$	Stress (y)
Guage1	0.0000846	2.085	3770	1.34E-03	9.E+00
Guage2	0.0000919	2.266	1290	1.60E-03	1.7E+01
Node101	0.0000831	2.05	3630	1.28E-03	7.00E+00
Node140	0.0000859	2.116	1810	1.33E-03	1.1E+01
Guage3	0.0000837	2.064	3270	1.33E-03	8.00E+00
Guage4	0.0000919	2.266	1130	1.60E-03	1.7E+01
Node241	0.0000862	2.126	1850	1.41E-03	1.1E+01
Guage5	0.0000847	2.087	3120	1.33E-03	8.00E+00
Guage6	0.0000919	2.266	1320	1.57E-03	1.6E+01
Guage7	0.0000837	2.063	3540	1.28E-03	7.00E+00

Table 5-2 Moments, Stresses and Strains for Selected Joints for Minimum Winter Temperature

The results obtained from non-linear analysis are shown in Figures 5-12 to 5-16, followed by comparison of the results obtained from linear and non-linear analyses in Figures 5-17

to 5-19. The cracking and yielding capacities of the bridge are also shown in the same figures.

Figure 5-12 Non-linear Moment under Minimum Winter Temperature

Figure 5-13 Non-linear Top Stress under Minimum Winter Temperature

Figure 5-14 Non-linear Bottom Stress under Minimum Winter Temperature

Figure 5-15 Non-linear Top Strain under Minimum Winter Temperature

Figure 5-16 Non-linear Bottom Strain under Minimum Winter Temperature

Figure 5-17 Linear and Non-linear Moment under Minimum Winter Temperature

Figure 5-18 Linear and Non-linear Top Stress under Minimum Winter Temperature

Figure 5-19 Linear and Non-linear Bottom Stress under Minimum Winter Temperature

The incremental moments, stresses and strains are shown in Figures 5-20 to 5-24 at locations of the strain gauges of the monitoring system and other selected points.

Figure 5-20 Moment under Minimum Winter Temperature

Figure 5-21 Top Stress under Minimum Winter Temperature

Figure 5-22 Bottom Stress under Minimum Winter Temperature

Figure 5-23 Top Strain under Minimum Winter Temperature

Figure 5-24 Bottom Strain under Minimum Winter Temperature

As shown by the results presented in Table 5-2 each section has its own cracking and yielding stress and strain, so that each section reaches cracking or yielding at a different load level. From Figure 5-24, it can be observed that at almost all sections, the strains are beyond the corresponding cracking strains at the maximum load, resulting that the bridge behaves non-linearly. However the strains are still significantly smaller than the yielding strains. Consequently, the results of linear and non-linear analysis are not very much different from each other. In order to find the minimum temperature that the bridge can safely withstand, a nonlinear analysis is carried out for the minimum design temperature in the code.

5.3.4 Minimum Code Design Temperature Behavior

According to the OHBDC design code, the minimum effective temperature for a concrete system bridge should be taken 5 °C below the minimum daily mean temp. For Ottawa the minimum daily mean temperature is -35 °C. Therefore the bridge is analyzed a minimum code design temperature of -40°C air temperature. The non-linear results are shown in

Figures 5-25 to 5-29, followed by comparison of the results obtained between linear and non-linear analyses in Figures 5-30 to 5-33.

Figure 5-25 Non-linear Moment for Minimum Code Design Temperature

Figure 5-26 Non-linear Top Stress for Minimum Code Design Temperature

Figure 5-27 Non-linear Bottom Stress for Minimum Code Design Temperature

Figure 5-28 Non-linear Top Strain for Minimum Code Design Temperature

Figure 5-29 Non-linear Bottom Strain for Minimum Code Design Temperature

Figure 5-30 Linear and Non-linear Moment for Minimum Code Design Temperature

Figure 5-31 Linear and Non-linear Top Stress for Minimum Code Design Temperature

Figure 5-32 Linear and Non-linear Bottom Stress for Minimum Code Design Temperature

The results obtained for low winter temperature are non-linear, however they are still far below the yielding level. It is clear that the behavior of the bridge is linear under summer temperature loading conditions, where the behavior may exceed the elastic limit into the non-linear range under winter temperature loading. Therefore in order to find more information about the low temperature performance of the bridge, additional analyses are carried out by including the traffic load in the following section.

5.3.5 Minimum Code Design Temperature and Truck Load Behavior

The purpose of the analysis here is to determine the most critical low temperature loading condition which the bridge may be subjected to so as to give more detailed insight on the low temperature performance of the bridge. In order to find the most critical loading condition, the location of the truck is varied $1/4$, $1/2$, and $3/4$ of each span. The results are shown in Figures 5-34 to 5-38. Figure 5-33 shows the various positions of the truck considered in the analysis. For example, the legend T-258 in Figures 5-34 to 5-38 indicates there are three trucks on the bridge. The centre of gravity of one truck is at point 2, which is located at the mid span of the first span; the centre of gravity of the second truck is located above point 5 at the mid span of the second span, and the third truck is located above point 8 at the mid span of the third arch. To apply the truckload on the bridge, the centre of gravity of the truck is placed at the desired point. Then the forces of the five axles of the truck considering their distances from the centre of gravity are applied on the bridge. If the axle is located in a spot of the arch where there is no node, it is distributed proportionally between the two nearest adjacent nodes.

Figure 5-33 Truck Locations

Figure 5-34 Non-linear Moment for Minimum Temperature and Different Truck Positions

Figure 5-35 Non-linear Top Stress for Minimum Temperature and Different Truck Positions

Figure 5-36 Non-linear Bottom Stress for Minimum Temperature and Different Truck Positions

Figure 5-37 Non-linear Top Strain for Minimum Temperature and Different Truck Positions

Figure 5-38 Non-linear Bottom Strain for Minimum Temperature and Different Truck Positions

By comparing the results of the above figures, it is observed that the case T5 leads to the highest stresses and strains. The moments, stresses and strains at selected points of the bridge under the increasing load of the case T5 are shown in Figures 5-39 to 5-43, followed by comparison the linear and non-linear results in Figures 5-44 to 5-46.

Figure 5-39 Increasing Moment for Minimum Temperature and One Truck

Figure 5-40 Increasing Top Stress for Minimum Temperature and One Truck

Figure 5-41 Increasing Bottom Stress for Minimum Temperature and One Truck

Figure 5-42 Increasing Top Strain for Minimum Temperature and One Truck

Figure 5-43 Increasing Bottom Strain for Minimum Temperature and One Truck

Figure 5-44 Linear and Non-linear Moment for One Truck and Minimum Code Temperature

Figure 5-45 Linear and Non-linear Top Stress for One Truck and Minimum Code Temperature

Figure 5-46 Linear and Non-linear Bottom Stress for One Truck and Minimum Code Temperature

Based on the results presented in Figures 5-42 and 5-43 and Table 5-3, it is noted that non-linear behavior is observed with the strains exceeding the cracking level. However the strains are still below the yield strain with some close to yielding. Further analyses are carried out by considering two trucks.

	Strain-Cr	Stress-Cr	My(KN.m)	yeilding strain	yeilding stress	Mu(KN.m)	ultimate strain	ultimate stress
Guage1	8.46E-05	2.09	3066	1.36E-03	8.71	6257	1.07E-02	15.69
Guage2	9.19E-05	2.27	1234	1.69E-03	16.75	2205	9.28E-03	24.05
Node101	8.31E-05	2.05	3892	1.32E-03	7.43	7632	9.13E-03	13.14
Node140	8.59E-05	2.12	2962	1.47E-03	11.57	5652	1.05E-02	18.40
Gauge3	8.37E-05	2.06	3814	1.36E-03	8.83	7398	1.08E-02	14.83
Guage4	9.19E-05	2.27	1326	1.67E-03	18.22	2386	9.14E-03	26.01
Node241	8.62E-05	2.13	2793	1.47E-03	11.90	5358	1.18E-02	18.96
Guage5	8.47E-05	2.09	3122	1.35E-03	9.07	6289	1.03E-02	15.96
Guage6	9.19E-05	2.27	1269	1.65E-03	17.36	2390	9.55E-03	26.44
Guage7	8.37E-05	2.06	3598	1.34E-03	8.23	7172	1.08E-02	14.40

Table 5-3 Moments, Stresses and Strains for Selected Joints for Minimum Code Temperature and Truck Loading

A non-linear analysis of the most critical truck location is carried out with two trucks at the same position. The results are shown in Figures 5-47 to 5-51, followed by the results at selected points of the bridge under the increasing load, as shown in Figures 5-52 to 5-56. Linear and non-linear analysis results are presented in Figures 5-57 to 5-59.

Figure 5-47 Non-linear Moment for minimum Temperature for One and Two Trucks

Figure 5-48 Non-linear Top Stress for minimum Temperature for One and Two Trucks

Figure 5-49 Non-linear Bottom Stress for minimum Temperature for One and Two Trucks

Figure 5-50 Non-linear Top Strain for minimum Temperature for One and Two Trucks

Figure 5-51 Non-linear Bottom Strain for minimum Temperature for One and Two Trucks

Figure 5-52 Incremental Moment for Minimum Temperature and Two Trucks

Figure 5-53 Incremental Top Stress for Minimum Temperature and Two Trucks

Figure 5-54 Incremental Bottom Stress for Minimum Temperature and Two Trucks

Figure 5-55 Incremental Top Strain for Minimum Temperature and Two Trucks

Figure 5-56 Incremental Bottom Strain for Minimum Temperature and Two Trucks

Figure 5-57 Linear and Non-linear Moments for Two Trucks and Minimum Code Temperature

Figure 5-58 Linear and Non-linear Top Stress for Two Trucks and Minimum Code Temperature

Figure 5-59 Linear and Non-linear Bottom Stress for Two Trucks and Minimum Temperature

The analysis results show that all sections have reached cracking but still below yielding, except at one point which reaches yielding. To obtain more detailed information about the behavior of the bridge, a more detailed evaluation and investigation of the stress and strain behavior at the middle span of the Plaza Bridge under the temperature and truck loading condition is carried out in the following section.

5.4 Stress and Strain Behavior of the Bridge under Temperature and Truck Loading

In order to verify the resulting stress and strain behavior of the bridge as determined by the equivalent model, the behaviour of the arch under combined truck loading and temperature loading at the mid-span of the Plaza Bridge are analyzed in more detail here. The bridge span is first subjected to the truckload and dead load, and then the temperature loading is added. The arch bridge is modelled by dividing into 22 straight segments. The dimensions and material properties of the midpoint section of the arch are listed in Table 5-4:

H(m)	B(m)	Density (kg/m ³)	Coefficient of expansion
0.649	1.00	2400	12E-6

Table 5-4 Properties of the Section

The equivalent stress-strain curve for the equivalent model representing the behavior of the mid-span section is shown in Figure 5-60.

Figure 5-60 Equivalent Stress-Strain Curve

A non-linear 2D analysis is performed for each loading condition, as stated before. The resulting stress and strain at the top and bottom of the section in the mid-span of the bridge under these loading conditions are shown in Figure 5-61.

Figure 5-61 Resulting Stress and Strain and Equivalent Stress-Strain Curve

Figure 5-61 shows the equivalent stress-strain curve and the resulting stress and strain under the truck load and temperature loading. The results show that the stress and strain due to truck loading can be identified on the equivalent stress-strain curve. The stress and strain behavior resulting from the additional temperature loading is described by the equivalent stress-strain curve shifted in the horizontal strain axis.

In order to have a more clear understanding of the behavior of the arch under temperature loading, the arch is analyzed under different reference temperatures. The resulting stress and strain at the top of the section for some selected points on the curve under three reference temperatures are shown in Figure 5-62. A shift of strain equal to $\alpha \Delta t$ is

observed for each reference temperature, where α is the coefficient of expansion and Δt is equal to the difference of the current temperature and the reference temperature.

Figure 5-62 Stress – Strain Curve for Various Reference Temperature

The input data for the arch is as follows:

$$\alpha = 12 \times 10^{-6} \text{ } ^\circ\text{C}^{-1}$$

$$T_{ref} = 15 \text{ } ^\circ\text{C}$$

$$T_{ref} = 0 \text{ } ^\circ\text{C}$$

$$T_{ref} = -30 \text{ } ^\circ\text{C}$$

where;

α = Coefficient of expansion

T = Curve Temperature

T_{ref} = Reference Temperature

So for $T_{ref} = 15 \text{ } ^\circ\text{C}$

$$\alpha \Delta t = 12 \times 10^{-6} (-15.57 - 15) = -0.00036684$$

For $T_{\text{ref}} = 0^\circ\text{C}$

$$\alpha \Delta t = 12 \times 10^{-6} (-15.57 - 0) = -0.00018684$$

For $T_{\text{ref}} = -30^\circ\text{C}$

$$\alpha \Delta t = 12 \times 10^{-6} (-15.57 - (-30)) = 0.00017316$$

5.5 Three-Dimensional Analysis

To find out the behavior of the Plaza Bridge under temperature and truck loading, a three-dimensional linear analysis is performed in addition to the two-dimensional analysis. The three-dimensional analysis is also performed to compare the two-dimensional and three-dimensional results in order to find out the accuracy of the analysis models.

The three-dimensional finite element model consists of layered shell elements. The model has 9 layers of shell elements, including the steel reinforcement as one of the layers with the same volume as the actual reinforcing bars.

The bridge finite element mesh is shown in Figure 5-63 with the strain gauges locations also shown in the figure. To perform the temperature analysis of the bridge, the temperature data at the top and bottom of the arch have been found by using thermocouple monitoring data at locations shown in Figure 5-64. Using these temperature data, a contour of the temperature distribution is obtained over the bridge surfaces. The

resulting temperature contour distribution is applied to the bridge surface in the form of an average temperature at the middle surface of the arch and a temperature gradient.

The results for the maximum temperature in the summer and minimum temperature in the winter are shown in Figures 5-65 and 5-66. The results are compared between two - dimensional and three - dimensional analysis results.

Figure 5-63 Three – Dimensional Modelling of the Bridge

Figure 5-64 Thermocouples locations

Figure 5-65 Moment under Maximum Summer Temperature

Figure 5-66 Moment under Minimum Winter Temperature

The analysis results show that the equivalent model results obtained from 2D linear and 3D linear analysis of the recorded monitoring temperature correlate well with each other and show the same trend.

Chapter 6 Summary and Conclusions

An investigation has been undertaken to study the response characteristics and the load carrying capacities due to temperature changes and truck traffic of an arch bridge over the entire loading range up to failure. The arch bridge structure studied is the Plaza Bridge, an old reinforced concrete arch bridge in the city of Ottawa which has recently been rehabilitated. During the rehabilitation project, a monitoring system has been installed on the bridge as a part of a health monitoring system of bridge systems. The monitoring data are used to obtain information on the performance of the bridge due to thermal effects and live loads.

Detailed parametric studies have been conducted to determine the influence of the temperature and truck loading on the bridge. Linear and non-linear finite element analyses have been carried out for the bridge. A simplified equivalent model, which accounts for the non-linear material behavior of reinforcing steel and concrete, has been developed in the present study. The developed model has been verified by numerical examples. The developed model can be used for non-linear frame analysis of reinforced concrete structures with beam-column members.

A special procedure is formulated to evaluate the effect of air temperature changes on the deformation behavior of the bridge. The measured temperature data from the monitoring

system are used in the performance assessment study to predict the performance of the bridge under extreme temperature conditions.

The performance of the bridge under combined extreme temperature and truck loading is investigated. In order to find the most critical loading condition, the number of trucks and the position of the trucks across the bridge are varied in the analysis. Based on the analysis results, the findings are summarized as follows

1. The finite element method is an accurate and powerful tool for the determination of the response behavior and performance of reinforced concrete arch bridges up to maximum load condition at ultimate state.
2. A new simplified equivalent model, which accounts for the non-linear material behavior of reinforcing steel and concrete, has been developed in this study. This model can be used for non-linear analysis of reinforced concrete frame structures with beam-column members. It is simple to implement and can give high accuracy on the failure behavior of reinforced concrete structures up to the ultimate failure state.
3. The equivalent model is developed based on specifying an equivalent stress-strain relation for the section of reinforced concrete structural member, which gives the same moment-curvature behavior as the original reinforced concrete section. In this way, the equivalent structural member behaves the same as the reinforced concrete section in non-linear analysis.

4. The equivalent model developed in this study has been demonstrated to give accurate results and reliable to give accurate predictions on the behavior and performance of reinforced concrete frame structures up to failure.
5. The analysis of the Plaza Bridge under the maximum recorded temperature from the monitoring system and maximum code design temperature according to OHBDC design code shows that the bridge responses remain linear and deformation of most sections are before cracking.
6. The analysis of the bridge for minimum recorded temperature from monitoring system shows that most sections of the arch beam members exceed the elastic limit of cracking of concrete into the non-linear range of behavior.
7. The analysis of the bridge subjected to the minimum code design temperature specified in the OHBDC design code shows that the arch members are expected to crack under the low temperature loading. However, it is pointed out that cracking in the reinforced concrete structure is not unexpected and since the deformations are still significantly below the yielding level, the performance of the bridge is therefore not seriously compromised.
8. Temperature analysis of the bridge shows that the low temperature behavior in the winter is more critical than its temperature behavior in the summer.
9. According to the analysis of combined minimum code design temperature loading and live load from truck traffic, the most critical position of the truck is found to

be at the mid-span of the central canal arch. At this position with two trucks side-by-side, the strains of some reinforcing steel bars reach yielding.

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