Assessment of a Precast Prestressed Segmental Concrete Rail Transit Guideway Design

by

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B.S. Civil Engineering (1996) University of Puerto Rico

Submitted to the Department of Civil and Environmental Engineering in partial fulfillment of the requirements for the degree of

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Abstract

The four objectives of bridge design (safety, serviceability, economy, and aesthetics) are discussed, and their complex interrelation is analyzed using, as a background, a precast prestressed segmental concrete rail transit guideway being constructed as part of the Tren Urbano Project in Puerto Rico. Advantages and problems with this type of structure are discussed. Special attention is given to problems related to bearings and expansion joints. To observe the effects that a design decision would have on the four objectives, a bridge model is developed and analyzed using the program SAP2000 in which the continuity of the superstructure is given different configurations. The changes of the structural behavior with changes in continuity are evaluated under dead loads, live loads, earthquake loads, creep, shrinkage, and thermal effects. A life-cycle cost analysis is executed to observe the economical impact of changing continuity. Suggestions on how to evaluate a design decision while considering the four objectives are made.

Thesis Supervisor: Jerome J. Connor
Title: Professor of Civil and Environmental Engineering

To the memory of Eugene Francis.

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Chapter 1

Introduction

1.1 Bridge and Elevated Guideway Design

An elevated guideway is a structure that is similar to a bridge. The difference between these two structures has not been clearly established, but normally an elevated guideway is a longer structure that is not used for crossing over an specific obstacle, like a river or a canyon. Instead, it crosses a city, over previously built streets and other structures.

In any case, in engineering terms, both the bridge and the elevated guideway are the same structure. The structural engineering terms that apply to one also applies to the other. Therefore, when referring to all bridge-like structures including elevated guideways and viaducts, only the word bridge is used. The same terminology has, therefore, been used in this report as well.

Designing a bridge structure or an elevated guideway is a very complex task. Its fundamental objectives are <u>safety</u>, <u>serviceability</u>, <u>economy</u>, and <u>elegance</u> (Menn, 1990). The final design depends on the relative weight given to each of these objectives.

Progress in the field of structural analysis includes innovations in computer analysis and a greater understanding of the behavior of structures. This has caused an increase in the demand for structures that require less maintenance, make optimal use of space, and behave better when subjected to lateral loads such as earthquakes and wind loads. Therefore, it has become very important to have an understanding of what it means to accomplish the above mentioned objectives. It is also very important to learn the complex relationship that exists between the four objectives.

There are a great number of decisions that need to be made in order to make a bridge design that fulfills all the desired objectives. Probably, one of the first decisions to be taken is what type of bridge will be used. One of the options that a bridge designer has is the Precast Prestressed Segmental Concrete bridge. This type of construction is specially appropriate when one or more of the following conditions are met:

1. There is a need for longer spans than the ones that can be built using other simpler construction methods.
2. The bridge has to pass over an area where a critical requirement is for the construction of the superstructure to be done from the top of the piers. Examples of such requirements are when a bridge has to be built over a canyon or across an urban area, where traffic cannot be obstructed.
3. The bridge has a large number of repetitive spans.

In the concrete segmental construction of bridges, the bridge is composed of several elements called segments. These segments can be either precast or cast-in-place. They are held together and made to carry loads by post-tensioning. The advantages of this construction method (discussed in Chapter 2) have made segmental construction very popular today. Like any other type of bridge construction method, the relationship between the four design objectives is a very complex one that needs to be understood.

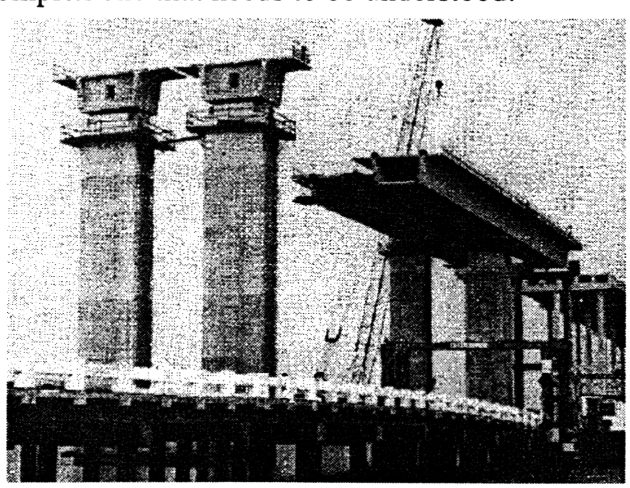

Figure 1.1: Construction of a precast segmental concrete bridge. (PTI/PCI, 1978)

1.2 The Tren Urbano Project

The Tren Urbano is a rapid transit system being designed and built for the metropolitan area of San Juan, Puerto Rico. The system includes elevated, underground and ground-level rail transit sections.

The first section of the Tren Urbano Project is known as the Bayamón contract. It is currently under construction and is scheduled to be finished by September of 1999. It consists of an elevated railway of 2.9 km length.

The elevated railway design of the Bayamón phase is composed mostly of two-span-continuous units, formed by precast box segments, seated in epoxy, and held together by prestressing steel cables. At the beginning and at the end of each unit, expansion joints and elastomeric bearings are used.

The design of the Bayamón phase will be covered in more detail in Chapter 4. Before covering the design in detail, it is important to know the past history of bridges in Puerto Rico, so that likely problems can be anticipated.

1.3 Bridges in Puerto Rico

When posed with the question: "What is the major source of problems with bridges in Puerto Rico?", most bridge experts would probably answer: "expansion joints".

This is not a problem that is unique to Puerto Rico. Expansion joints are a source of problems for bridges around the world. If there are problems with the sealing system of the joint, leaks can occur. Therefore, there can be corrosion in the reinforcement steel and in the bearing system, and stains noticeable on the columns. Protection against corrosion is very important in Puerto Rico, given that it is an island and has a lot of salt in the air. Without enough cover, reinforcement steel quickly corrodes.

Another problem typical in Puerto Rico is that the expansion joints themselves deteriorate. This deterioration can be felt by a rider in a moving vehicle as a bump. How uncomfortable and noisy the bump is will depend on how deteriorated is the expansion joint. The deterioration will also cause stresses that were not accounted for in the guideway design.

The causes of problems with expansion joints are: improper installation and poor inspection and maintenance. It is generally believed in Puerto Rico that if expansion joints were properly installed and given reasonable inspection and maintenance, there would be no problems with them on the island.

Usually, the highest loads for which bridges in Puerto Rico are designed are earthquake loads. Although there has not been an earthquake in Puerto Rico since 1918, the island is located in an earthquake prone zone. Recent events in California and Japan make designing for earthquakes a field that is increasing in importance and sophistication.

Another fact is that bridges in Puerto Rico are mostly built using simple spans. Very seldom do designers try to take advantage of using continuous units. They seem to be extremely cautious about changes in length due to changes in temperature, which is very strange because the temperature in the Puerto Rico does not goes through such drastic changes as it does in other parts of the world. It also appears that not many designers are willing to go through the more complicated procedure of designing for continuous spans.

Segmental bridge construction is practically new in Puerto Rico. Only one segmental concrete bridge has been constructed: the Caguana River Bridge in Utuado, a part of

Highway 10. There are no segmental elevated railways in Puerto Rico. In fact, there are no railway bridges in operation in Puerto Rico.

1.4 Research Objectives

The objectives of this research are:

a) Improve upon the current design philosophy for the elevated railway structure of the first phase of the Tren Urbano Project with the hope that these improvements are considered for adoption in the design of future sections of the project and in future designs similar to the one presented here. This objective will be achieved by analyzing how the behavior of the structure would be affected by changes in span continuity (simple span, two-span-continuous, three-span continuous, etc.). Alternative designs might have the following advantages over the current design:

1. Less vibrations.
2. Smaller deflections.
3. More stability.
4. Greater resistance and better behavior under lateral loads, particularly earthquakes.
5. Use of fewer expansion joints and bearings, which in turn would mean a structure that needs less maintenance.
6. Improved appearance.

b) Study the feasibility and economic impact of the alternative designs. Once an optimal design is found, it needs to be determined as to what is needed to be done to make the changes and what are the costs associated with these changes. The cost-effectiveness of making the changes, as compared to keeping the current design, will be studied by performing a life-cycle cost analysis. This will determine the plausibility of making the improvements to the design.

1.5 Organization

In Chapter 2, the precast prestressed segmental concrete construction method is introduced, presenting advantages and disadvantages that this method will have in comparison with other methods. A general overview on the process of design and construction of precast prestressed segmental concrete bridges and guideways is given.

In Chapter 3, the four design objectives (safety, serviceability, economy, and aesthetics) of bridge structures are discussed. Design issues that may cause the failure of not achieving the four objectives when constructing the precast prestressed segmental concrete bridges or guideways are presented.

In Chapter 4, the design of the first phase of the Tren Urbano is presented in detail. The method of construction is also explained. Possible problems that may arise with the structure are mentioned.

In Chapter 5, an analysis is performed in order to study the structural impact of increasing changing the continuity of a bridge structure similar to that of the Tren Urbano Project. An introduction to the computer program SAP2000 is given. The results obtained from the analysis using the program are discussed.

In Chapter 6, a Life-Cycle Cost analysis is performed on each of the design alternatives evaluated in Chapter 5.

Finally, in Chapter 7, the results of the analysis presented in Chapter 5 and 6 are analyzed to show how the four fundamental objectives of bridge design get affected with one design decision, in this case being increasing continuity of the superstructure. Final recommendations are given on how to improve the design philosophy in the design of bridges.

Chapter 2

Precast Prestressed Segmental Concrete Bridges and Guideways

2.1 Reinforced vs. Prestressed Concrete

Prestressed concrete is defined by the American Concrete Institute (ACI) as: "concrete in which there have been introduced internal stresses of such magnitude and distribution that the stresses resulting from given external loadings are counteracted to a desired degree. In reinforced-concrete members, the prestress is commonly introduced by tensioning the steel reinforcement."

It is known that concrete performs well under compression, but it is not as good under tension. When prestressed concrete was invented, it was with the idea of achieving a concrete that did not have any tensile stresses when external loads were applied. As the definition of the ACI says, the prestressing was done by tensioning the reinforcement steel of the concrete members.

In reinforced concrete, the tensile forces resulting from the bending moments are resisted by the reinforcing steel, which is put into tension by the bond created in the reinforcement process. Therefore it is assumed that the tensile strength of the concrete in reinforced concrete is negligible and disregarded. Once a reinforced member reaches its limit state at service load, the cracking and deflection that occur are practically irrecoverable.

In contrast, in prestressed concrete, the prestressing permits a relatively high controlled recovery of cracking and deflection. In the case the flexural tensile strength of concrete is exceeded, the prestressed member starts to act like a reinforced concrete element.

There are certain qualities of prestressed concrete that can make its use more cost-effective than the use of reinforced concrete. A prestressed member usually has a depth of about 65 to 80 percent of the depth of a reinforced concrete member with the same span and loading conditions (Nawy, 1996). This means that there is a saving in materials when prestressed concrete is used. Also, by having shallower depths, the prestressing members can be lighter than the reinforced members, which means that lighter foundations can be achieved with prestressed members. In addition, because cracking is reduced, less maintenance is needed and longer working life can be achieved using prestressed members, again meaning more savings.

By no means do the statements above imply that prestressed concrete its always more economical than reinforced concrete. In prestressing, a higher quality of materials is needed. Besides, performing the prestressing is a more complex task than simply reinforcing the concrete, therefore resulting in an additional cost. Which one is more economical between prestressed or reinforced concrete depends on the specifics of the structure being designed.

When a beam is being designed for a span longer than 30 meters (like the ones used in bridges and guideways), if reinforced concrete is used, the cross-section needed to resist the moments caused by the life loads will be very large. This increases the self-weight of the beam substantially, causing excessive cracking and deflections. Therefore, the use of prestressed concrete becomes necessary, unless arches are used.

2.2 Non-Segmental Prestressing vs. Segmental Prestressing

Currently, short-span bridges (up to 50 meters) in the United States are most commonly build using precast pretensioned concrete I-girders, taking advantage of all the benefits of using prestressed concrete (PCI, 1992). Still, although longer spans can be achieved with prestressed concrete than with reinforced concrete, sometimes there were requirements for even longer spans that could not be achieved with just prestressed concrete. Other materials, like steel, had to be used. Therefore, segmental construction got its start as an alternative concept for bridging longer spans.

Segmental bridges are those that are constructed from a number of short transverse segments. Usually, prestressing is applied to segmental bridge construction, allowing the construction of longer and/or thinner spans. Prestressing is done by using post-tensioning tendons in the longitudinal direction of the beams.

The most typical configuration in prestressed segmental bridge construction is the box girder. This is true because in most cases, box girders are the most efficient and economical design for a bridge (Podolny and Muller, 1982). In fact, the box girder has been used in almost every type of bridge construction including simple span, continuous spans, cantilevers, arches, stayed girders, and suspension. When using box girders, permanent post-tensioning can also be applied transversely to the top slab to increase the strength of the deck, and vertically to the webs to increase the shear capacity of the box girder.

The box girders used are mostly single cell and double cell. There have been cases where three-cell box girders has been used, but these segments need temporary stiffeners to prevent buckling during construction (Wium and Buyukozturk, 1984).

Since getting started as an alternative to bridging longer spans, segmental construction continued development because it offers other advantages. One of them is that horizontally curved alignments, humps or sag vertical curves, and transitioned cross slopes can easily be accommodated with segmental construction. Also, when compared to other types of construction, segmental prestressed construction can be done by a small work force performing repetitive tasks. Besides, with segmental construction it is possible to reduce significantly the interference with existing traffic, eliminating expensive detours.

Finally, it is normally believe that segmental construction protects the environment (Podolny and Muller, 1982). If an elevated guideway is constructed instead of constructing a highway, a road, or a railway using cut-and-fill type of construction, then an environmentally sensitive area would be less affected with the narrower path of the alignment construction. Then, if the guideway was constructed using segmental construction, longer spans would be possible and the construction of the superstructure could be done from top of the piers, thereby protecting the environment even more.

2.3 Precast vs. Cast-In-Place

As mentioned before, segmental bridges are constructed from a number of short transverse segments. The segments can be either:

1. Precast, meaning they are manufactured in a precast yard on or off-site, which then are taken to the bridge location where they are assembled; or

2. Cast-In-Place, meaning they are cast in their final position.

Choosing between any of the two will bring advantages and disadvantages.

The advantages of the using precast concrete over cast-in-place concrete in prestressed segmental bridge construction are:

1. The precast segments can be fabricated while the substructure is being built, which means a saving in construction time.

2. The fabrication of this segments is done by the repetitive use of industrialized manufacturing techniques. This provides the opportunity of achieving high quality and high strength concrete.

3. There is no need for falsework and everything can be accomplished from the top of the completed portions of the structure. This is very useful when traffic has to remain undisturbed under the bridge or for high-level crossings.

4. The effects of shrinkage and creep are substantially reduced. This happens because usually, the segments have already matured to full design strength by the time of erection and post-tensioning.

5. Precast construction is less sensitive to weather conditions than cast-in-place construction.

The advantages of using cast-in-place segments instead of precast segments in segmental construction are:

1. There is no required dimensional control of high degree during the manufacturing and erection of the segments. Connections of the ducts for tensioning cables between segments can be done more easily.

2. There is room for error in cast-in-place construction. If the actual camber does not agree with predicted camber, corrections can be made as the construction progresses by revising the alignment of the following segments.

3. Joints can be treated better for transfer of bending and shear stresses and for water tightness for protection of the tendons. Longitudinal reinforcing steel can be placed between the segments.

4. The size and weight of the cast-in-place segments is not limited to the transportation equipment, as it is for the precast segments.

2.4 Precast Prestressed Segmental Concrete Construction

All the advantages of precast concrete, prestressed concrete, and segmental construction come together in precast prestressed segmental concrete construction. This type of construction is most widely used in the construction of bridges. In this section, general aspects of this type of construction for bridges are presented.

2.4.1 Fabrication of the Precast Segments

The fabrication of the segments can be distinguished by the type of joint to be used between the segments themselves (PTI and PCI, 1978). This joint can either be a cast-in-place joint or a contact joint. These joints will be discussed in more detail in the Section 2.4.3.

When using cast-in-place joints (also known as wide joints or broad joints), the precision of line of segments depend more on how accurate is the casting of the joint during erection and less the accuracy of the segments (PTI and PCI, 1978). Segments using wide joints can be cast separately.

When using contact joints (also known as match-cast joints), the connecting surfaces between the segments fit each other very accurately, so only a thin layer of filling material is needed for the joint. Sometimes no filling material is needed at all. Segments using match-cast joints are cast by either long-line method or the short-line method.

In the long-line method (Figure 2.1), all the segments are cast on a long line. One or more formwork units move along the line. The formwork units are guided by a pre-adjusted soffit. A long line is easy to setup and to maintain control. After stripping the forms, it is not necessary to take away the segments immediately. Its main advantage is that similar corrections are usually needed for each span, so that only one base structure has to be built for all of the spans in the structure.

Unfortunately, the long-line method has some disadvantages. Substantial space may be required for a long line (usually about half the length of the longest span of the structure). The long line must be constructed on a firm foundation which will not settle under the weight of the cast segments. If the structure has a curvature, the long line must be designed to accommodate for it. Also, the equipment for casting and curing has to move from place to place since the forms are mobile.

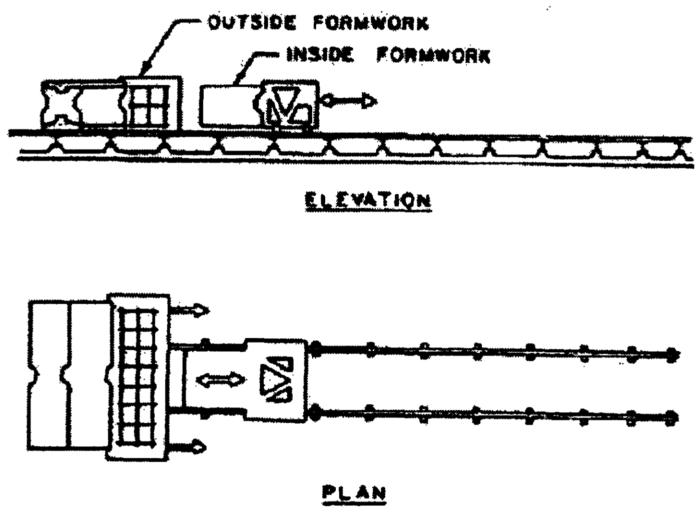

Figure 2.1: The long-line method. (Barker, 1980)

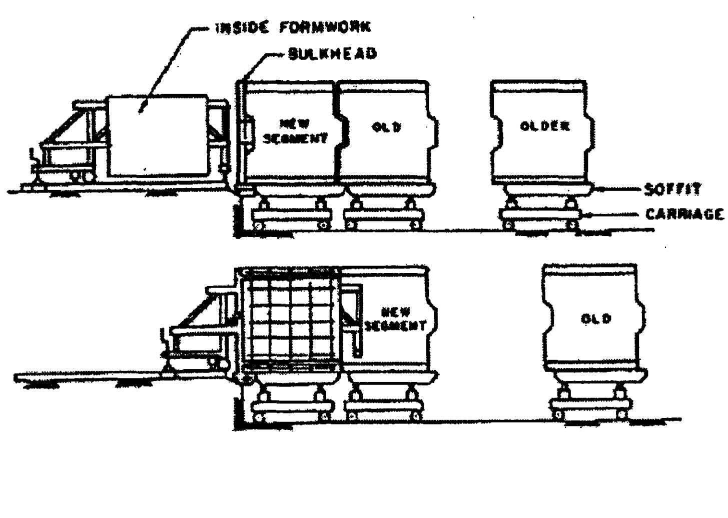

Figure 2.2: The short-line method. (Barker, 1980)

In the short-line method (Figure 2.2), the segments are cast at the same place in stationary forms and against a neighboring segment. After the segment is cast, the neighboring segment is taken away and the new segment takes its place, so that another segment can be cast. The space needed for the short-line method is about three times the length of a segment, small compared to the space needed for the long-line method (PTI and PCI, 1978). Curves and twisting of the structure can be obtained by adjusting the neighboring segment. In the short-line method, the elements can be cast in the vertical position to be later used in the horizontal position.

The short-line method has a disadvantage. To obtain the desired structural configuration, the neighboring segment must be positioned with extreme accuracy.

The length of the precast segment will depend largely on what equipment is available for transporting, lifting and prestressing. When choosing what is going to be the size of the segments, it has to be decided between having segments with the same length or having segments with the constant weight (Wium and Buyukozturk, 1984). Designing for the same length has the advantage that less adjustments are needed for the formwork. Meanwhile, designing for constant weight means having the advantage of always transporting and lifting units at full capacity which could reduce the construction time.

2.4.2 Erection Methods

There are four methods used to assemble a segmental bridge: the balanced cantilever method, the progressive placing method, the incremental launching method, and the span-by-span method. The method used depends on the length of the bridge or guideway, the individual span lengths, the height of the superstructure along its length, access to the area below where the superstructure is going to be, and the available equipment.

In the balanced cantilever method, a segment is placed on top of a column and securely anchored so that it can resist large overturning moments. This segment can be either cast-in-place or precast. Then, other segments are put on each side of the pier, one at a time alternating from side to side, stressing tendons that go through the segments and over the pier. Segments on both sides of the pier balance each other. When the number of segments on each side of the pier is not equal, there is a large unbalanced moment at the column. The procedure is repeated until the cantilever segments from adjacent piers meet at midspan. The segments used can be either precast or cast-in-place.

The progressive placing method is very similar to the balanced cantilever method. The difference is that the construction starts at a pier and proceeds in only one direction. Therefore, in this method, a cantilever can be almost as long as the length of the span being constructed. The bending moments on the columns are substantial, often requiring the use of temporary intermediate supports.

In the incremental launching method, the construction of the superstructure is performed from the top of an abutment. A segment is cast on top of the abutment. The segment is cured and prestressed, and then moved forward, so that a new segment is cast against this segment. Then, prestress is applied to the segments, and both segments are moved forward. This procedure is repeated, and the superstructure spans from pier to pier until it is completed. Very often, a steel launching nose is attached to the deck of the first segment in order to reduce the free cantilever length of the first span. Temporary piers or temporary stays might be used in cases where the negative moments are too large.

In the span-by-span method, a supporting system is used to place all the segments of a given span during construction. This structure can be a truss fixed at the columns that supports the segments (in case of precast construction) or formwork (in case of cast-in-place construction) from below, or it can be a truss that holds the segments or formwork from above the deck. Since this method uses the truss, the length of the spans of the bridge to be constructed is limited to the length of the truss.

The method used to erect precast units for any of the previous methods will depend on the site conditions. They can be lifted and put in place by using a crane on the ground or a barge. They can be lifted using equipment on the deck of previously erected units, or, in the case of erection using the span-by-span method, they can be placed using a launching gantry.

The launching gantry can be supported by the completed piers, by the completed parts of the superstructure, or by both. They are very expensive and are only used economically in large projects. They are very useful when the area under the superstructure can not be used for the delivering of precast units or for operating erection equipment.

2.4.3 Joints

When placing the segments to create the superstructure, the segments can be placed together using contact joints or cast-in-place joints. One type of contact joint is the dry joint, which does not need any filling material between the segments, but the most popular is the epoxy joint. In the epoxy joint, the adjacent segments have matching surfaces, and a very thin layer (less than 1 millimeter) of epoxy coating is used between the segments.

Epoxy has many uses. It acts as a lubricant when placing the segments and works as a glue that helps the segments act as a single unit. Also, epoxy helps correct some of the irregularities between the mating surfaces of adjacent segments, and serves as waterproof material for the joints between segments.

Cast-in-place joints are made using mortar and concrete. They are wider than the epoxy joints. Mortar or unreinforced concrete has been used frequently for joints between segments of 24 to 100 millimeters. Reinforced concrete has been used for joints from 200 to 600 millimeters, where the reinforcing steel from adjacent segments is lapped or welded together (Degenkolb, 1977).

Mortar and concrete joints do not require the exact fitting that epoxy joints need, but they are not suited for cantilever construction. This type of joint is mostly suited for construction using falsework.

Today, almost all the joints used in segmental construction are contact joints. Cast-in-place joints are mostly used when the alignment needs to be corrected (Wium and Buyukozturk, 1984), and to make a a final adjustment in a span of contact joints.

2.4.4 Railroad Bridges and Guideways

Segmental prestressed construction is as suitable for rail transit and railroads as it is for highways. The procedure for designing an elevated railway is similar to that of a highway bridge. The difference lies in the larger design life and impact loads for railroad structures. Therefore, elevated railways are more stocky than highway bridges.

In the case of a railway bridge, the actual load applied to the structure is much closer to the design load than in the case of a highway bridge. Therefore, fatigue and durability of railway structures could be a problem and need careful consideration (Podolny and Muller, 1982). This is made even more important due to the fact that the maintenance and repair of railway structures could cause unacceptable disturbance in the train operation.

Usually, ballast is used across the railroad bridge so that the ties and rails are kept in proper alignment and grade separate from the bridge structure. Typically, for rail transit, the ballast is not used, and either the ties are fixed directly on top of the deck, or the track is set directly on pads and shims attached to the bridge deck, eliminating the use of ties and ballast. When using ties, they can be made of wood, steel, reinforced concrete, or prestressed concrete. Whichever system is used to fixed the track to the deck, it should have drains or sufficient slope so that water is not trapped.

Until about 30-40 years ago, all tracks, in all networks, were laid by leaving gaps between consecutive rails, and then jointing the rails with fishplates (Profillidis, 1995). The gaps and fishplates were used to permit elongations in the rail due to temperature changes. The use of fishplates has many problems (Profillidis, 1995):

1. It significantly reduces passenger comfort.
2. It causes considerable wheel and rail fatigue and wear.
3. A bumping noise is produced when the vehicle's wheels run on top of it.
4. It greatly increases maintenance expenses.

An alternative to jointed rails is the continuous-welded rail (CWR). A CWR is formed by putting together discrete pieces of rail, eliminating the use of fishplates. No thermal expansions along the length of the rail are permitted. This generates forces in the rail that it must be able to resist.

The cost of installing CWR is higher than that of installing a jointed rail. Nevertheless, the CWR provides an adequate return in capital for the initial investment by reducing maintenance cost, improving track stability, permitting the achievement of higher speed by the vehicles, reducing power consumption, slowing the development in track defects, reducing noise, and improving passenger comfort (Profillidis, 1995).

There is a tendency to use continuous-welded rails wherever possible (Fryba, 1996). However, with the introduction of the continuous-welded rail into railroad bridges and guideways, a new problem came up with regard to the interaction between the superstructure and the rail. New forces and stresses are developed with changes in temperatures. This problem is presented in more detail in Chapter3.

Chapter 3

Design Considerations for Precast Prestressed Segmental Concrete Bridges and Guideways

3.1 Introduction

Bridge design for any kind of bridge, including precast prestressed segmental concrete bridges, should always have the same fundamental objectives. In this chapter, these fundamental objectives of bridge design are presented, and what it means to achieve these objectives is discussed. Later in the same chapter, major issues concerning precast prestressed segmental concrete bridges are discussed, and how they affect the accomplishment of the fundamental objectives is analyzed.

3.2 Design Objectives

The fundamental objectives of bridge design are safety, serviceability, economy and elegance. A design can only be successful when all four goals have been achieved. The relative importance of each objective is going to be defined by the consequences of not achieving them (Menn, 1990). Things get further complicated by the fact that all four objectives are interrelated. For example, it is possible that making a structure more aesthetically pleasing may mean added costs that would make it less economical.

3.2.1 Safety

A bridge is safe when it is known that it is not going to collapse under the applied loads. The achievement of safety for a bridge structure is mostly believed to be scientific process (Menn, 1990). A bridge designer follows design codes, and a bridge is designed to resist ultimate loads. When an ultimate load is applied and the structure resists the load, then it is said that the applied and resisting loads are in equilibrium. If an applied load exceeds the value of an ultimate load, the structure is not in equilibrium and, therefore, collapses. If all the steps in the design and construction process are followed accordingly, and if the structure is inspected properly and given the necessary maintenance, the bridge is said to be safe, and the structure is not supposed to collapse as long as the values of the ultimate loads are not exceeded.

The fact is that the value of both, the loads to which the structure is subjected and the ultimate load which the structure can resist, are not known exactly. Both are functions of many different factors. Among other things, the loading will depend on the actual traffic patterns that use the bridge, the wind loads and earthquake loads to which the structure is subjected, the settlements taking place in the foundations, and the changes in temperature that occur in the structure. Likewise, the ultimate load the structure resists will depend, among other things, on the actual strength of materials, the workmanship given in the construction, the deterioration the structure suffers, and the inspection and maintenance given to the structure. That is why design codes have factors of safety included in their procedure. There are factors of safety that make the expected applied load larger, and factors of safety that make the expected structure resistance lower.

Design codes are not infallible. Bridges that have followed design codes have been known to collapse. Sometimes this is because the applied loads were larger than expected, even with the use of factors of safety. At other times it was because the structure was not constructed according to its design, or because it was not given the proper inspection and maintenance. At times design codes improperly simulate the structural action of bridges.

The factors of safety and equations found in design codes are trying to simulate what will happen to the structure in reality. Sometimes the simulations are not as close to reality as they need to be. Therefore, codes are revised and changed, trying to get closer to reality and helping in the design of safer structures.

In spite of all of the above, safety is not only achieved by following the design codes. The codes do not specify which material to use for construction, what method of construction to use, how to arrange the components of the structures, and many other issues that arise in the design phase. These issues, that have to be decided by the designers, are also going to play a role in determining how safe the structure will be. Therefore designers must use their creativity and common sense, as well as their personal and observed experiences with similar projects. It is good to know what has gone both, right and wrong, with similar projects so that this knowledge can be used in the design of a safer structure.

3.2.2 Serviceability

Serviceability is an issue very similar to safety. It is sometimes believed to be achieved completely through scientific procedures by following design codes (Menn, 1990). In reality, this will also depend on the creativity, common sense, knowledge, and experience of the designer.

A bridge is serviceable when it fulfills the need for which it was designed in a positive manner. The most important aspects of serviceability are *appearance*, *function*, and *durability* (Menn, 1990). Appearance refers to giving the impression that the structure is secure. The showing of cracks and large deflections can give the impression that a structure will fail if used, although it may not. Function refers to how the bridge works when used. The bridge should be safe for the users (and for any traffic that passes under it) during both normal and emergency conditions. It should not be uncomfortable, and it must meet the requirements of abutters and the aspirations of the community as a whole. Finally, durability refers to remaining in service or remaining fit for use for a long period of time.

There are aspects of serviceability that can be quantified such as deflection, vibration, cracking, run-off of rain water on the deck, and traffic volumes. These aspects can be fulfilled by following design codes and following proper construction procedures. Other aspects, that can not be quantified, need of the creativity and experience of the designer to be provided, for example, corrosion of the bearing system. The designer should think of protecting the system against corrosion, and provide for easy inspection and replacement of components in case of corrosion.

A bridge will deteriorate with time. The more serviceable will deteriorate slower. In any case, all bridges need inspection. Therefore, inspection is very important in maintaining a bridge serviceable, like proper design and construction are too.

3.2.3 Economy

Economy in bridges is about getting as much as possible out of money. It is not about spending the least possible amount of money in the construction of the bridge. The big picture has to be seen, and costs incurred in operation, maintenance, rehabilitation, and demolition of the bridge have to be considered too.

When a bridge is designed, a number of decisions have to be made. These decisions will impact the cost of construction, and other costs in the long run. They include the selection of materials, the type of construction to be used, how will the workers get organized, details of the structure, and the equipment to be used. Also, an appropriate strategy has to be established for construction, maintenance, rehabilitation, and ultimate disposal.

As mentioned earlier, every bridge deteriorates. The difference is how fast they deteriorate. This will depend on those very first decisions made for the construction phase. How it deteriorates will impact the operation of the bridge and the maintenance needed for the bridge to remain serviceable. Also, how the bridge deteriorates will determine if the bridge needs at some point in the future a major rehabilitation, or how can it be disposed of the structure in the case is deemed the bridge is not useful anymore. Operation, maintenance, rehabilitation and demolition represent costs, and they can be very substantial.

The important thing to remember here is that when aiming for cost-effectiveness in the design of a bridge, the future operation, maintenance, rehabilitation and demolition, as well as the present (or immediate future) construction costs must all be thought of. The best tool the designer has to determine the cost effectiveness of a bridge is the life-cycle cost, in which the total cost of the bridge is calculated for the life-span that the bridge is designed for.

3.2.4 Aesthetics

The issue of achieving aesthetics in bridges can, at first, appear to be very complicated, because beauty is believed to be very subjective. In reality, by checking the literature written by experts in aesthetics of bridges, it can be found that there is agreement on most of the major issues of elegance in bridges.

The experts agree that bridges must be adequate for their surroundings, whatever they are. As Ritner (1990) explained: "visual beauty is something that is not awkward, it is something that does not appear to be out of place". At the same time, the bridge structure can be perceived as an individual entity (Menn, 1990, 1991), therefore its beauty will also depend on the bridge's structural form.

Commonly believed ways of achieving an elegant bridge include: using efficiently the materials for a structure that is slender and transparent as opposed to heavy and massive; integrating harmonically all the structures components into one coherent, organized structure; making the structure symmetrical; designing for simplicity without unnecessary ornamentation; and, incorporating topographical features of the surroundings.

As for safety, serviceability, and economy, when designing for aesthetics, the future must be kept in mind. How the bridge will deteriorate has to be considered, along with what kind of maintenance must be given to keep its elegance has to be thought through.

In many cases, the first obstacle that has to be crossed in order to get an aesthetically pleasing bridge is to have the bridge designer understand that making a bridge more elegant does not always means having a more expensive structure. Sometimes money has been saved by changing a bridge design to make it more elegant. And in the cases where making a design more elegant means an increase in cost, the changes should not always be discarded just for the sake of saving money. The designers have to understand that the public does care about having an aesthetically pleasing bridge, approaching the decision of adding cost for elegance as they would do for adding a safety feature that would increase the cost of the bridge (Gottemoeller, 1990, 1991).

The majority of bridges are designed by engineers who have no formal education in aesthetics. That is why the engineer must work together with a person who does have the formal education: the architect (Lacroix, 1990). It is in public agencies on whom the responsibility of designing, building, maintaining, and as consequence, achieving elegance in bridges falls (Kruckemeyer, 1990). Therefore, they should get architects to be involved in the design of public bridges.

3.3 Major Issues in the Design of Precast Prestressed Segmental Concrete Bridges and Guideways

In 1994, the American Segmental Bridge Institute (ASBI) published in the PCI Journal the results of a survey in which the performance of 96 segmental concrete bridges of the United States and Canada was evaluated (Miller 1994). The major conclusions of this report were:

1. Segmental concrete construction performs well over time, with consistently high conditions ratings for bridges that have been in use for up to 30 years.
2. The performance of post-tensioned segmental concrete is similar to that of other prestressed concrete bridges.

Though it has been found that precast prestressed segmental construction is a very good method of construction, it is still not perfect. In this section are presented important design decisions and possible sources of problems related to precast prestressed segmental concrete bridge. Some of the decisions and problems are not exclusive to precast prestressed segmental construction. It will be shown how these decisions and problems will affect the achievement of the four before mentioned objectives of bridge design.

3.3.1 Continuity

One decision to be made in a bridge or elevated guideway design is what kind of span configuration should be used. Basically, a decision must be taken between using simple spans or a continuous system. Furthermore, if a continuous system is selected, then a continuous unit needs to be chosen (two span continuous, three span continuous, etc.). Selecting a continuous system over the use of simple spans will have an effect on a number of important aspects of the structure. Some of these effects are positive, and some are not.

In a simple span configuration, the maximum moment on each beam occurs at midspan, while the moments at both ends are zero. If the configuration is changed to continuous, the moment at midspan is reduced, while the maximum moments occur on the section of the unit that passes on top the support continuously. The moments at the end of the continuous unit are zero.

A problem with using a continuous design is that it is much more difficult than making the design of a bridge in which all its spans are simply supported. It is important that the bridge designer realizes that there are many possible advantages that can be taken out of designing a more continuous structure and that it is worth going through the trouble. Not exploring the possibility of using a continuous system could mean not achieving the four objectives of bridge design.

Safety

If a bridge design is changed from using simple spans to using continuous units while the size of the cross-section is not changed for the new design, the stiffness of the girders would be increased. Therefore, the continuous unit could resist heavier loads than the simply supported member.

Another advantage obtained by using continuity is that the over-all structural stability is improved through redundancy. Continuity increases safety margins by allowing the redistribution of stresses under overload conditions (Taly, 1998). Also improved is the response to dynamic loads such as earthquake and wind loads. This happens because increasing the continuity of the structure means having more columns connected to the girder, which is what causes the improvement of stability, and provides a better mechanism to resist horizontal loads.

Another way of improving resistance of lateral loads, is that by increasing continuity of a design, the maximum moments at mid-span are reduced, therefore allowing for a reduction in the size of the girder cross-section. This reduces the weight of the structure which, in turn, reduces the momentum of the structure, which reduces the lateral loads to be resisted (Taly, 1998).

Is not always better to increase continuity. As explained before, in the columns where the superstructure pass over continuously, there is a concurrence of moment and shear at the support sections, which reduces the moment strength at this locations. Also at these columns, new lateral forces and moments are formed. These forces are caused by the elastic shortening of the long-span beam under prestress, and by horizontal loading that could come from earthquakes, acceleration and breaking of vehicles, and temperature changes. Additionally, the secondary stresses, usually produced by shrinkage, creep, temperature variations, and settlement of the supports, are magnified when compared to a simply supported structure. All these possibilities need to be considered, and the supports may need more reinforcement, be made more flexible or made larger.

Another possible problem that could present itself if the continuity of a bridge design is increased is the reversal of moments. If live loads are much heavier than dead loads, and if partial loadings on the spans are considered, the beam can be subjected to serious reversal of moments. This problem could be addressed for by using partial prestressing (in which some tension is allowed in the concrete).

Serviceability

By changing a bridge design from using simple spans to using continuous units, if the size of the cross-section that would be used for the simple span design is kept, the stiffness of the girder would be increased. Therefore, the continuous units would have a smaller deflection than the simply supported.

The number of expansion joints and bearings used could be reduced by increasing the continuity of the bridge. This means that the structure would require less of the inspection and maintenance associated with expansion joints and bearings. Other problems associated with expansion joints, such as leakage, and corrosion of the reinforcement and the bearing support system, could be minimized. Also, by reducing the number of expansion joints, the number of bumps felt by riding over them is reduced, thus the ride comfort is improved.

The problem with an increase in continuity is that there are going to be greater elongations at the free-ends caused by an increase in temperature. Therefore, bigger and more complex expansion joints have to be provided. It is more difficult to provide sealant for these joints.

Economy

If a bridge design is changed from simple span to continuous, the size of the cross-section can be reduced due to the reduction of the maximum moments and stresses at the midspan. Having a smaller cross-section means a reduction in the cost of materials and, possibly, construction. Also, the structure would be lighter, which means that the columns and foundations can be lighter, which, in turn, means further reduction in the cost of materials and, possibly, construction. Alternatively, the span length could be increased with continuity which, in turn, could mean that the number of piers to be constructed could be reduced.

By increasing continuity, the number of anchorages needed to tension the steel at intermediate supports can be reduced when compared to the quantity used for simply supported members. This means further reduction in costs and labor.

The number of expansion joints and bearings is reduced by increasing the continuity of a bridge. This means that the structure would require less inspection and maintenance, and therefore, there would be a reduction in cost. Sometimes, depending on the development of new lateral stresses, the width and thickness of the piers can be reduced by reducing the number of bearings, which means further savings of materials. Other problems associated with expansion joints, such as leakage and corrosion, could be minimized, therefore the possibility of having to perform what could be a very costly bridge rehabilitation are reduced.

In continuous beams, various spans can be prestressed with the same continuous tendon by providing and undulating profile, resulting in the reduction of anchorages needed. This is not always possible, because there could be a non-acceptable reduction in the prestressing force when continuous tendons are used due to the friction between the prestressing steel and the duct through which the steel pass caused by the increase number of bends that the tendon has to go through.

Various disadvantages that come with continuity are: the concurrence of moment and shear at the support where the superstructure passes over continuously; excessive lateral forces and moments at this supports due to the elastic shortening of the long-span beam under prestress and by horizontal loading that could come from earthquakes, acceleration and breaking of vehicles, and temperature changes; and, the secondary stresses, usually produced by shrinkage, creep, temperature variations, and settlement of the supports, are magnified when compared to a simply supported structure. Therefore, these supports require more reinforcement, unless the reduction of the moment strength at this section is acceptable.

Aesthetics

As mentioned before, continuity allows the reduction of the cross-section of the beam. This permits having a slender, hence more elegant, structure than a bridge that uses simple spans.

Again, by increasing the continuity of a bridge, the number of expansion joints and bearings needed could be reduced. This causes the reduction of the possible leakages of the expansion joints that causes the staining of the substructure (Wasserman, 1991). Also, by reducing the number of bearings, the width and thickness of the piers can be reduced, which makes the structure more aesthetically pleasing (Menn, 1990).

3.3.2 Expansion Joints

Another important decision that has to be made when designing a bridge or a guideway is the selection of expansion joints to be used. The ones chosen must be fit to handle displacements of the structures such as those caused by the changes in temperature or seismic loads. According to David J. Lee(1994): "expansion joints should, more correctly, be known as movement joints since they cater for relative movement between bridge deck spans and abutments resulting from a number of causes not exclusively due to the temperature".

There are two ways in which expansion joints can be classified. The first way is by classifying them as either open gap or covered gap (Köster, 1969). The use of this classification has to do with the riding quality of the expansion joint when the joint is used in highways, as opposed to when it is used in rail transit, where the vehicle does not make contact with it. The classification has nothing to do with the sealing used. The other method of classification is based on the size of the movement the joint has to take into account. They can be classified as joints for small movements, for medium movements, or for large movements (Lee, 1992).

All over the world, bridges deteriorated due to faulty expansion joints. Usually these joints have defective or ineffective waterproofing, or poor drainage details.

Serviceability

Expansion joints are located in an extremely vulnerable position. They are subjected to the impact and vibration of traffic, and exposed to both the effects of the natural elements (water, dust, grit, ultra-violet rays, and ozone) and applied chemicals (salt solutions, cement alkalis, and petroleum derivatives).

The expansion joints must fulfill the following conditions (Lee, 1994):

1. accommodate horizontal and vertical movements of the structure;
2. withstand applied loadings;
3. be hardly noticeable when ridden on top of it, therefore not being a hazard to any kind of road user;
4. resist corrosion and withstand attack from grit and chemicals;
5. require little maintenance;
6. permit easy inspection, maintenance, repair or replacement.
7. prevent the penetration of water, silt, and girt, or provide for their removal.

The lifetime of the expansion joint will depend on the chosen model, the type of structure which it serves, the quality of the workmanship given at installation, and the traffic it is subjected to (Chabert and Ambrosino, 1996). Problems have been registered with expansion joints for the seven conditions mentioned above. When expansion joints are not performing well (either because of improper design or improper installation), they can be a nightmare for the serviceability of a bridge.

If an expansion fails while the joint is used in a highway bridge, it becomes quite noticeable to the riders in vehicles. Also, if the waterproofing fails, damage can be caused to the reinforcing steel of the beams end that the joint is supposed to protect, accelerating the deterioration of the bridge. The elastomeric bearings can also get affected with the corrosion of its reinforcing steel. The same can happen to the piers. The fact that the expansion joint fails means that it is going to need replacement, creating additional maintenance for the bridge. Also, the deterioration of the joint causes new forces and stresses in the bridge that were not accounted for in the design.

Therefore, it is important to give special attention to expansion joints. When carefully designed and detailed, properly installed, and given reasonable maintenance, expansion joints should be trouble-free for many years.

Economy

There is a large selection of expansion joints in the market. It is important to choose one that gives good performance and trouble-free life for at least as long as that of the surfacing. They should be properly installed, and given the necessary inspection and maintenance. Otherwise, the expansion joint could start producing problems that could be costly to fix. If they don't work properly: they could permit leakage that could cause the corrosion of reinforcement and of the bearing system; and they would have to be replaced with new expansion joints.

Aesthetics

The major aesthetic problem that expansion joints present is leakage. Those that permit the passage of water borne chemicals, solids, and pollutants stain and discolor the structure which can significantly alter and detract from the elegance of the structure (Watson, 1996).

3.3.3 Bearings

Another important decision that has to be made when designing a bridge or a guideway is the selection of the bearings to be used. The bearings chosen must be fit to handle displacements of the structures such as those caused by the changes in temperature or seismic loads.

Bearings can be classified broadly into three groups: mechanical bearings, elastomeric bearings, and a combination of both. Mechanical bearings are usually regarded as rigid, while elastomerics are elastically yielding.

In mechanical bearings, movements and rotations are accommodated by rolling, rocking or sliding action, usually of metal parts. Much use is made of low friction plastics in the sliding surfaces. In elastomeric bearings, movements and rotations are accommodated by compressing or shearing layers of rubber-like materials. They can be long strips, plain pads or laminated, that is, several rubber-like layers interleaved with and bonded to thin sheets of steel.

There exist intermediate types of bearings. For example, a bearing where an elastomer is used as the rotation medium, but horizontal movement capacity is provided mechanically.

Serviceability

Bearings are used to take care of all kinds of movements that a bridge could face. Sources of movement in bridges are:

1. temperature and humidity changes;
2. creep, shrinkage and fatigue effects;
3. axial and flexural strains arising from dead loading, live loading, prestressing, etc.;
4. dynamic load effects;
5. overload;
6. tilt, settlement or movement of ground;
7. mining subsistence;
8. seismic disturbance;
9. moving parts of structures;
10. erection procedures.

Generally, bridge bearings fulfill the following functions (Lee, 1994):

1. They transfer forces from one part of the bridge to another, typically from the superstructure to the substructure.
2. They allow movements of one part of the bridge in relation to another.
3. They allow free movement in some directions but not in others.

Bearings should normally be designed to last as long as the bridge in which they are placed. The problem with metallic bearings is that can suffer corrosion and subsequent seizing and collapse. Meanwhile, improper maintenance of the non-metallic materials may reduce the serviceability life of the bearing.

It is important to provide for the inspection and the replacement of bearings in whole or in part. Provisions should be made for the installation of jacks necessary for the removal of bearings or any part thereof, insertion of shims or any other operation requiring lifting of the bridge deck from the bearings. This operations should be completed with minimal difficulty.

Bearings should be detailed without crevices and recesses that can trap moisture and dirt. The materials used in their manufacture and the method adopted for protection against corrosion should be such as to ensure that the bearings function properly throughout their life. Dissimilar materials that can give rise to corrosion currents should not be used. Special precautions should be taken when a bearing is used in aggressive conditions, such as a marine environment.

Bearings must be put in places where they are able to operate as intended. Any secondary effect produced by eccentric loading or by movements not along a major axis of the bearing should be taken into account in the design of the bearings.

Economy

In particular for bearings, a decision must be made between bearings of low cost that need a lot of maintenance and have a short life-span, or more expensive bearings that need less maintenance and a have a longer life. The selection should be made carefully, probably with the help of calculating life-cycle costs, which should include costs of installation, inspection, maintenance, and replacement of the bearing.

The lifespan of a bridge can depend on the time of life of the bearings used. Spending more money on bearings can result in an overall cheaper structure (due to reduction in bending moments) with a longer time of life.

Aesthetics

Bearings are usually designed only with structural performance in mind, forgetting about their aesthetic impact (Watson, 1996). The area of the location of bearings are a source of aesthetic problems. When using bearings, the width and thickness of the piers needs to be increased which makes the structure less aesthetically pleasing (Menn, 1990). Also, the use of bearings can provide enough space between the top of the pier and the bottom of the beam for birds to make their nests, whose defecation could make a bridge less elegant and cause deterioration due to the acid and humid nature of the droppings.

3.3.4 Fabrication of Precast Segments

There are two methods of fabricating precast segments: with cast-in-place joints and with contact joints. These methods are discussed in more detail in Section 2.4.1.

Economy

There are certain problems that can occur during the fabrication of precast segments. These problems are economically related because, if these problems were to occur, the segment or segments would need to be fabricated again, creating an additional construction cost. All of them can be avoided if proper construction procedures are followed.

Since precast prestressed segmental concrete bridges are designed to very close tolerances, it is important to ensure that the concrete reaches the specified strength at an early age in order to be able to transfer prestressing forces to the concrete. Therefore, a low water-cement ratio is used for the concrete, but this reduces its workability, and makes it difficult to compact. Proper quality control is therefore needed to reduce the variability in the concrete strength. Doing so will allow the reduction of the target strength while producing a concrete with sufficient strength, but with better workability (Wium and Buyukozturk, 1984).

Sometimes steam curing is used in the fabrication of the precast segments. If the steam curing is applied too early, it could cause the cracking of the segments. It has been suggested that steam should only be applied after four hours have passed since the placing of the concrete.

Problems can occur when an element being cast has a different temperature than the elements that is being cast against. This is due to the heat generated by the curing concrete of the segment being cast. It could cause the elements not to fit properly anymore after the concrete is cured. Therefore, ample heat should be applied so that both elements have the same temperature, and the joints can match perfectly after the segments cool off (Wium and Buyukozturk, 1984).

Segments should not be placed on top of poor bearing surfaces before being erected. This way, warping and permanent deformations are avoided (Wium and Buyukozturk, 1984).

All the problems mentioned above are economy related, because if they occur it will mean that a segment or various segments will need to be fabricated again.

Other problem that does not necessary means the fabrication of a new segment is the complexity of the section, which makes it hard to compact the concrete during casting. This can cause the problem of honeycombing, which can be very costly to fix, and thus is economy related (Casey, 1982).

3.3.5 Joints

There are two kinds of joints used with precast prestressed segmental concrete construction: cast-in-place joints and contact joints. Joints are discussed in more in detail in Section 2.4.3.

Serviceability

Joints between segments should be carefully designed and constructed. They must transmit normal and shear stresses, but they should also be designed to require as little maintenance as possible. Joints must prevent the entrance of water into the prestressing ducts, therefore protecting the tendons from corrosion.

In some precast prestressed segmental concrete bridges, contact joints have failed because the epoxy was not handled properly during the mixing and application procedure, particularly in rain and cold weather. The negative effects of this problem can be largely reduced by designing shear keys in the webs (Podolny and Muller, 1982).

Also when using epoxy joints, tendons should not be threaded until after the epoxy has reached its design strength. Otherwise, the high stresses caused by the already placed tendons can cause the epoxy to bleed and block the ducts where tendons have not been placed yet.

Economy

If the joints are not properly designed and constructed, the rehabilitation of the bridge can be very costly. This is especially true if the joint allows the water to get into the ducts, permitting the corrosion of the tendons.

The fabrication of cast-in-place joints requires a long curing period of time. Therefore, additional equipment might be required to hold the new segment in place while the joint achieves its working strength (Wium and Buyukozturk, 1984). Still, they might be necessary to correct the alignment of the bridge.

Aesthetics

Joints can be an aesthetic problem because they can make the structure divided, instead of the preferably continuous appearance. This can be especially true for the case when cast-in-place joints are used, and the concrete between the joints and the segments is of different color. In the case when contact joints are used, if there is oozing epoxy between the segments, the discontinuity is accentuated, besides having non pleasing stains.

3.3.6 Ducts, Prestressing Tendons, and Anchors

Steel tendons are normally installed in precast prestressed segmental concrete bridges by being pulled or pushed through voids in the concrete formed by ferrous metal tubing known as ducts. Commonly, galvanized metal tubing is used (PTI/PCI, 1978).

All ducts must have sufficient grouting inlets, vent pipes and drains, to permit proper grouting and to avoid accumulation of water inside the ducts. Inlets and/or vent pipes shall be located at all high points of the tendon profile (PTI/PCI, 1978).

Post-tensioning tendons are used to apply both temporary and permanent prestressing forces to the precast segmental concrete bridges. Permanent post-tensioning is required for the primary longitudinal reinforcement, and sometimes is used in the top slab to provide transverse post-tensioning, and in the webs for vertical post-tensioning. At other times, permanent post-tensioning is used to provide a permanent connection between the superstructure and the piers (PTI/PCI, 1978).

Temporary post-tensioning is used during cantilever construction. Permanent post-tensioning is not applied only after a segment has been erected at each end of the cantilever. Therefore, during the placing of the first segment at one end, the element is attached to the cantilever using temporary post-tensioning (PTI/PCI, 1978).

The anchorages for permanent longitudinal tendons may be located at either the webs in the face of the segment, or in special web stiffeners cast into the segment for the purpose of providing a location for anchorage that does not interferes with the erection process (PTI/PCI, 1978).

Serviceability

Using too many ducts for post-tensioning tendons in thin concrete plates can cause the spalling off of concrete due to insufficient bearing. Also, it is important that the ducts are not damaged or misplaced during construction because these could make more difficult the threading of tendons, and they could cause the loss of friction forces (Wium and Buyukozturk, 1984). In some cases, laminar cracking in the deck slab or in the bottom flange have occurred, due to the wobble and improper alignment of the ducts at the joints (Podolny and Muller, 1982).

In a number of precast prestressed segmental concrete bridges, the concrete has cracked or spalled off in the area of the prestressing tendons and anchors (Wium and Buyukozturk, 1984). Tensile cracks have been found behind tendons anchorages, particularly for high-capacity continuity tendons in the bottom flange of the box section (Podolny and Muller, 1982).

Usually, after the tendons are stressed, they are grouted to improve the bond between the tendons and the ducts. This grout could enter adjacent ducts, blocking it before the adjacent tendon is placed. Therefore, grout should be placed only after all adjacent tendons have been placed and post-tensioned enough so that the safety of the bridge during construction does not depend on whether the ducts have been grouted or not. At tendons anchors, all voids left in the webs must be fill with grout, to prevent the formation of ice there.

Economy

There have been cases in which water has entered the ducts during construction, where it has frozen. This is a particular problem for the ducts anchored in the deck slab, like the vertical prestressing tendons or draped continuity tendons (Podolny and Muller, 1982). This can slow the construction process, which in turn can mean an additional construction cost.

3.3.7 Erection Methods

There are four methods used to erect the segments in precast prestressed segmental construction: the balanced cantilever method, the progressive placing method, the incremental launching method, and the span-by-span method. These methods are discussed in Section 2.4.2.

Economy

Usually the contractor has the task of selecting which of the four methods of construction should be used. This selection will significantly impact the cost of the project. It will determine the equipment needed for construction and the time it will take to complete the project, plus it will also impact adjacent traffic to the construction.

Segmental bridge design vary from one to the other. Usually, a lot of the equipment, like gantries, trusses and lifting devices, have to be designed and built for each project, increasing the time and construction of a project (Otter, 1982).

3.3.8 Instabilities

During segmental construction, partially completed spans will create large forces and moments on the substructure, especially during cantilever construction. Sometimes these are going to be the largest loads the substructure will ever experience. It is very important to account for these loads in the design of the substructure.

Safety

It is important that all partially constructed parts of the bridge structure remain stable during construction (Wium and Buyukozturk, 1984). Bridges constructed using a cantilever method are more prone to having stability problems.

Emphasis has to be given to the strength of small local details such as temporary prestressing tendons, temporary jacks and bearings, and spacer blocks used prior to the installation of expansion joints (Wium and Buyukozturk, 1984).

3.3.9 Geometric Control

Geometric control refers to making sure that the completed bridge has the correct elevations on the deck level. Geometry control needs to be accounted for from conceptual design through production and final erection methods. If this control is not provided, the project will be unsatisfactory (Bender and Janssen, 1982).

Serviceability

Bridges constructed using cantilever methods are more susceptible to having changes in the elevation on the deck level due to creep, shrinkage, elastic deformation, and changes in loads. Therefore, elements have to be cast with additional camber to obtain the desire elevation (Wium and Buyukozturk, 1984).

3.3.10 Time Dependent Effects

Creep and shrinkage are deformations that occur to the concrete through time. Creep is the property of continuing to deform over considerable lengths of time at constant stress loads, like post-tensioning. Shrinkage is caused by the evaporation of a large part of free water in concrete (meaning water that was not needed for hydration). This effects cause the shortening of the superstructure, therefore new stresses and forces are developed in both the superstructure and substructure.

Safety

Time dependent effects in concrete can cause serious problems in precast prestressed segmental concrete bridges. Typically, these bridges are much larger and have expansion joints farther apart than the conventional concrete bridges do, which make the total shortening more significant. In some precast prestressed segmental concrete bridges, the creep and shrinkage deformations were not correctly calculated, probably because of their large size and due to their sensitivity to changes in the material properties (Wium and Buyukozturk, 1984).

Serviceability

Creep and shrinkage can cause transverse cracking, opening of joints, and deflections in the structure. To avoid the problem of cracking and opening of joints, transverse post-tensioning may be needed. Deflections can be critical when joining two cantilever section. If the two sections are not at the same level, large secondary moments can be developed by joining them. The camber of the segments has to be adjusted to avoid this problem.

3.3.11 Superstructure/Rail Interaction

In Section 2.4.4, the tendency of using continuous-welded rail (CWR) was discussed. Using CWR in bridges introduces forces not very well understood into both the superstructure and the rail due to changes in temperature. In Chapter 5, a model is analyzed to study the superstructure/rail interaction.

Safety

The use of CWR introduces several new forces and modifies the distribution of other superstructure design forces during temperature changes. Variations in temperature develop significant interaction forces between the rail and the superstructure that need to be considered in the design of both, the superstructure and the substructure. Usually these forces are longitudinal, but they can be transversely horizontal if the bridge is curved.

Likewise, the superstructure will influence the stresses and displacement of the CWR, especially above the movable bridge bearings. Therefore, the use of the CWR has to be evaluated, using as limiting factor the expansion length of the bridge (Fryba, 1996).

The maximum expansion length of the bridge will be determined by four conditions: strength, gap in the case of rail structure, mutual rail and bridge displacement, and stability. The strength and gap conditions determine the maximum expansion length if ballast is used, while the mutual displacement condition determines in the case ballast is not used. The stability condition is complied with as long as a maximum stress in the rail is not exceeded. The four conditions, in turn, will depend on the fastening of rails and its maintenance, on the rail cross-section area, on the presence of ballast, on the material of the bridge, and on the maximum difference of temperature of the rails and the bridge from the fixing temperature (Fryba, 1996).

Serviceability and Economy

If the interaction between the rail and the bridge is not studied carefully, it could affect the serviceability and the economy of the guideway. The superstructure has to be designed to resist the stresses and forces of this interaction, which could mean an increase in construction costs. In the case rail failure occurs, the rail system would be interrupted, and fixing this problem would mean additional maintenance costs.

Chapter 4

Tren Urbano Rail Transit Guideway Design

4.1 The Bayamón Contract

The construction of the Tren Urbano Project has been divided into seven contracts. Some of these segments, and other possible future ones, are to consist of an elevated railway, including the first section of the project. The first section is known as the Bayamón contract.

Since the Bayamón contract is the first section and is currently under construction, more information is available than for any other elevated railway segment. Therefore, the design of this contract's guideway is described here. Still, most of the information given here for the Bayamón contract applies to the other elevated railway sections.

The reader must understand, that a guideway for a rapid transit system is a complex structure that has a lot of details to take into account, and that covering all these details here is not possible, nor is it useful to what this research is investigating. What it is presented in this chapter is a general overview of what to expect from the Bayamón contract. The information given here applies to most of the contract, but there are going to be exceptions that are not going to be discussed here. For example, it will be said that elastomeric bearings will be used, but there might be a pier in the contract that uses a pot bearing.

4.2 Design

The elevated railway of the Bayamón contract of the Tren Urbano Project is currently under construction using the segmental construction method. The use of this method is very appropriate in this and in future phases of the Tren Urbano because the San Juan Metropolitan Area is densely populated. A lot of existing traffic needs to remain undisturbed, and the segmental construction method provides a way of minimizing disturbances. Also, the Project can take advantage of the repetitiveness of segmental construction, which can accelerate the erection of a structure of such length as that of the Bayamón contract (2.9 km).

The substructure of the guideway consists of columns with circular cross-sections, and of two kinds of foundations: piles and spread footings. The type of foundation used for a specific column will depend on the available space, on the nature of the soil, and on the forces and moments developed in the columns.

The superstructure is made using precast concrete box girder segments. In the majority of the Bayamón contract, the units formed with the segments are two-span-continuous, with each span having a length of about 36 meters. The units are seated together in epoxy and held together using post-tensioning steel. The size of the cross-section of the segments will depend on if the guideway is single track or dual track. In both cases, the shape of the cross-section of each segment will depend on the location of the segment in the span.

On top of the columns where the superstructure is not continuous, a pier cap with an inverted-T shape is cast in place (Figure 4.1). Two elastomeric bearings are located on each of the flanges of the inverted-T. On top of each pair of bearings rests part of the precast concrete segment known as the joint segment. This segment is a non-prismatic member. It has two cross-section, of which one rests on top of the bearing, while the other is on the air (Figure 4.1). On Appendix F Figures F.1 and F.2, these two cross-sections are shown.

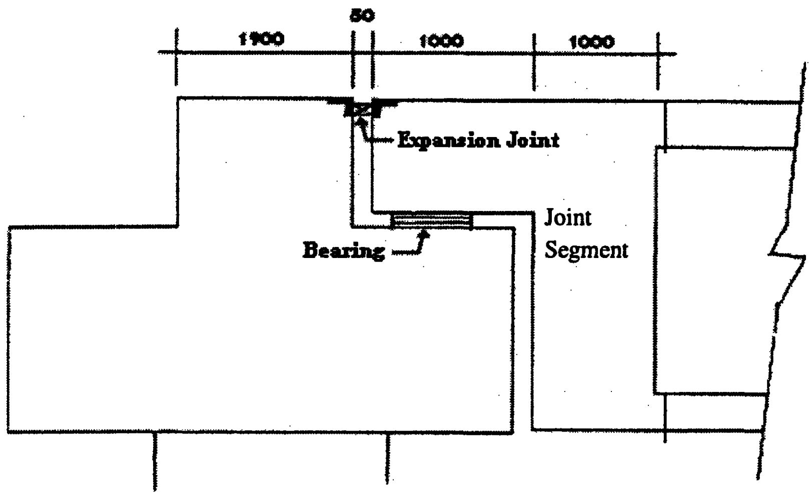

Figure 4.1: Elevation of column with pier cap.

On top of the columns where the superstructure is continuous, a segment is cast in place forming a monolithic structure with the column. This segment is known as the pier segment (Figure F.3).

The methods used for construction are the span-by-span method and the balanced cantilever method. Which method of construction is used depends on where the span is going to be constructed. This will determine construction conditions like span length, limited clearance or construction over traffic.

When a span is constructed using the span-by-span method, eleven segments are used between each pier segment and each joint segment. These segments are known as typical segments. The typical segment cross-section is shown in Appendix F (Figure F.4).

When the balanced cantilever method is used for the assembly of a span, the number of typical segments between the joint segment and the pier segment varies. Besides typical segments, cast-in-place joints that could be longer than 1 meter are used.

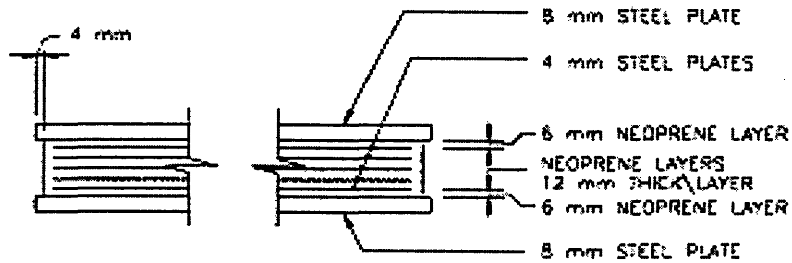

Figure 4.2: Reinforced elastomeric bearing. (Tren Urbano and GMAEC, 1996)

The bearings used consist of various slabs of elastomer bonded to metal plates in sandwich form (Figure 4.2). The elastomer is neoprene and the metal is steel. The bearings have outer steel plates at the top and at the bottom. The bottom outer plate is attached to the flange of the inverted-T, while the top plate is attached to the joint segment. The translational movement is accommodated by shear in the elastomer. Rotational movement is accommodated by the variation in the compressive strain across the elastomer.

The elastomer should have sufficient shearing flexibility to avoid transmitting high horizontal loads and sufficient rotational capacity to avoid transmission of significant moments to the supports. The vertical stiffness should be such that significant changes in height under loads are avoided (Lee, 1994).

Between the top of the joint segment and the top of the pier cap, expansion joints are to be located (Figure 4.1). At the time this is being written, the designers of the Bayamón contract appeared to have chosen a certain type of expansion joint that is very similar to the joints used for other bridges in Puerto Rico (Figure 4.3). It consists of edge-protection angles connected to anchor bars that are embedded in concrete. In the gap between the angles, an elastomeric material is placed. This material has a cross-section that allows free expansion and contraction in the gap between the angles, and the same time it does not permit that any material (water, dirt, or others) falls through the gap.

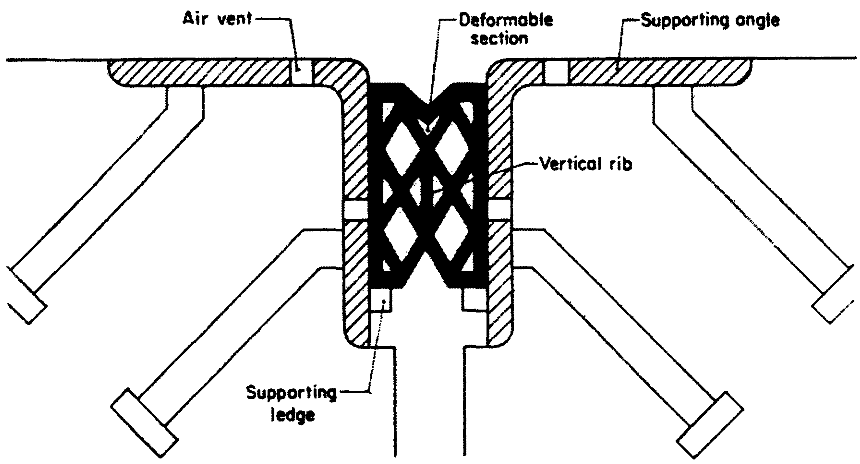

Figure 4.3: Expansion joint. (Long, 1974)

Finally, the rails that will be used are CWRs attached to the superstructure and adjusted for accuracy using concrete pads. The pads are cast in place around steel rings that are anchored to the top of the precast segments. The CWRs are fixed to the ties with the use of fasteners. A rubber-like material is used between the ties and the rail in order to reduce friction when the rail experiences thermal expansion and contractions, and to reduce the transmission of vibration and noise produced by the vehicle wheels on top of the rails.

4.3 Methods of Erection

Two methods of erection are going to be used for the construction of the guideway: the span-by-span method and the balanced cantilever method. For both cases, the use of an assembly truss could be necessary.

The span-by-span method can be used whenever the span length can be covered with the assembly truss. As described in the Tren Urbano Fixed Facilities Category II Drawings (Tren Urbano and GMAEC, 1996), the erection of a segment with the span-by-span method is done in three phases (Figure 4.4). They are:

1. Segment Placement
   a. After the previous span has been completed, the truss has been advanced and placed between two piers to construct the next span.

   b. The precast segments are delivered by truck and loaded onto the assembly truss with the use of a ground based crane.

   c. After all the segments have been loaded on the truss, the grade and alignment of each segment is adjusted, leaving a gap between each segment for the application of epoxy.

2. Epoxy Joining and Stressing of Longitudinal Tendons
   a. In a typical span, there are eleven segments between the joint segment and the pier segment, segments one through three are named section 1, segments five through seven are named section 2, and segments nine through eleven are named section 3. Segments four and eight are where the deviation ribs are located. The deviation ribs are used to change the trajectory of the post-tensioning tendons through the span. For each of the sections 1,2, and 3, epoxy is applied between the segments, and the segments are stressed together with temporary post-tensioning bars.

   b. Polyethylene ducts are positioned between the expansion joint segment, the pier segment, and the deviation ribs.

   c. Longitudinal post-tensioning tendons are installed.

   d. Concrete joints are placed in closure joints.

   e. Selected tendons are stressed in both webs simultaneously to approximately 10% of the final stressing force.

   f. Closure joints are poured.
   g. After closure joints have reached a minimum compressive strength, the tendons are stressed simultaneously to their final force.

   h. The completed span is lowered onto the bearings.

3. Truss Advancement
   a. A truss support bracket (used to support the assembly truss) is positioned at the next pier.

b. The assembly truss is advanced to the next pier.

c. Phases 1 and 2 are repeated to assemble a new span.

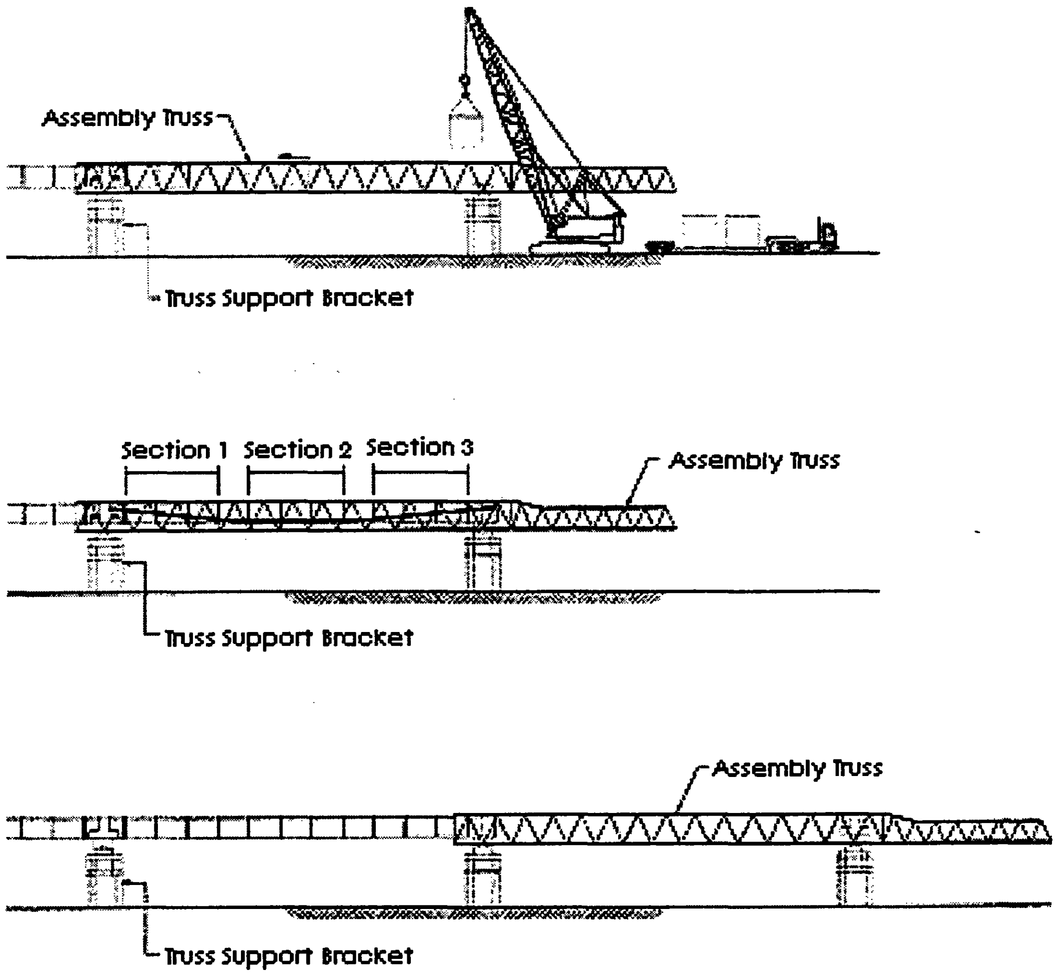

Figure 4.4: Span-by-span method. (Tren Urbano and GMAEC, 1996)

When the balanced cantilever method is used (Figure 4.5), it is mostly because a span can not be constructed with the assembly truss. Following is an example of how the balanced cantilever method can be used in the erection of a span A, located between piers A and B, which is next to a span B between piers B and C. Span A is too long to be constructed with the span-by-span method. The procedure is presented in five phases:

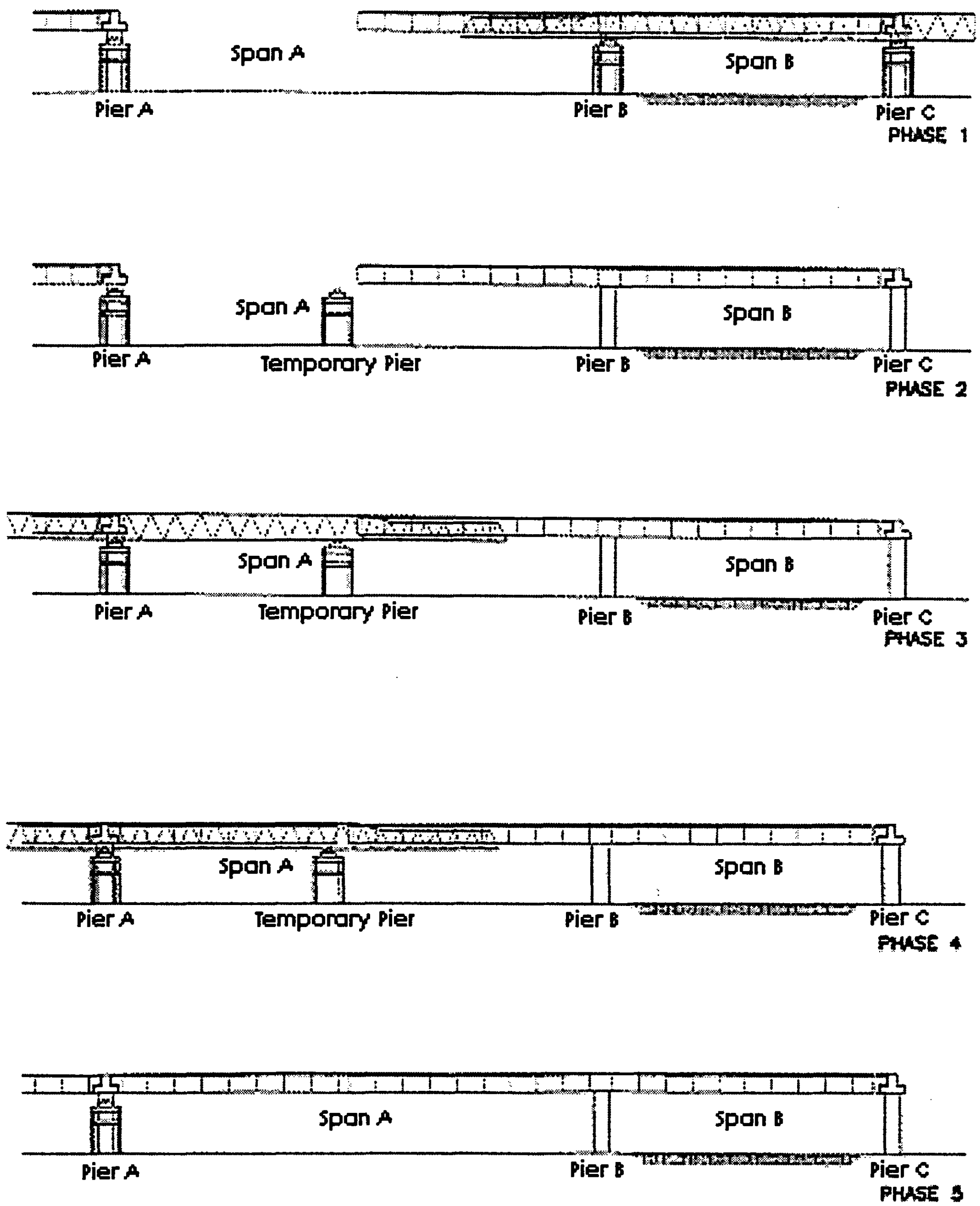

Figure 4.5: Balanced cantilever method. (Tren Urbano and GMAEC, 1996)

1. Using the assembly truss to support the segments for span B, a balanced cantilever is constructed at pier B toward pier A.

2. A temporary pier is constructed for span A.

3. The assembly truss is placed between pier A and the temporary pier.
4. The remaining portion of span A is constructed with the span-by-span method.
5. Continuity post-tensioning is stressed, cast-in-place joint are made, and the temporary pier is removed.

4.4 Loads and Forces

According to the Tren Urbano Design Criteria (United States Department of Transportation, 1996), in the design of the Tren Urbano structures, the following loads and forces shall be considered:

1. Dead loads (DL):

Dead loads include the actual weight of the structure and other permanently installed fixtures such as track-work, acoustical barrier, and emergency walkway. Also needed here to be considered are the loads to which the structure is subjected during its erection.

2. Live loads (LL), including rail vehicle (RV), and maintenance rail crane car (RC):

A train in the Tren Urbano will consist of 1, 2, or 3 pairs of rail vehicles. One rail vehicle has four axles (Figure 4.6). The weight of each axle shall be taken as 146 kN (this includes the weight of the vehicle and its passengers). It has to be taken into consideration the possibility of one train pushing or pulling a failed train. The guideway has to be designed for any train combination that produces the most critical condition.

The guideway has to be designed for one rail crane car on one track at a time. It has four axles, each weighing 134 kN.

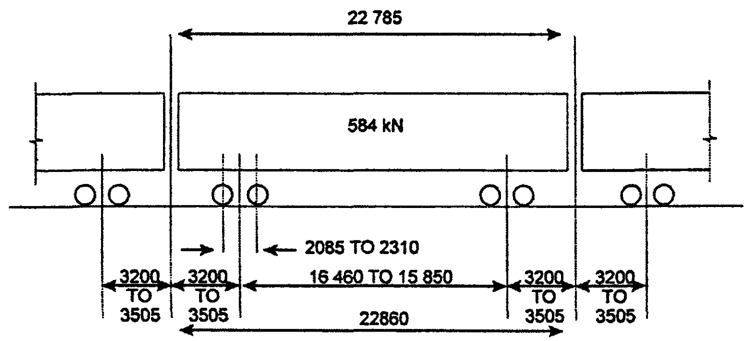

Figure 4.6: Rail vehicle. (USDOT, et al., 1996)

3. Derailment loads (DR):

These loads are produced in the case the derailing of a train occurs. It includes vertical derailment loads, and horizontal derailment loads.

Vertical derailment loads shall be produced by fully loaded vehicles placed with their longitudinal axes parallel to the track. The lateral vehicle excursion shall vary from 100 to 900 millimeters maximum for tangent track and for curved track with radii greater than 1.5 kilometers. A vertical impact factor of 100 percent of vehicle weight shall be applied in computing the equivalent static derailment load. When designing a dual track guideway, only one train on the track shall be considered to have derailed, while the other track is loaded with a stationary train.

For cross-sections having clearance between vehicle and barrier wall of 150 millimeters to 900 millimeters, with maximum vehicle speed of 88 kilometers per hour, the horizontal derailment force shall be taken as 40 percent of a single fully loaded vehicle acting 600 millimeters above the top of the rail and normal to the barrier wall for a distance of 3 meters along the wall.

4. Earthquake force (EQ):

The guideway has to be designed in accordance with Division I-A, Seismic Design of the 15th Edition of the AASHTO Standard Specification for Highway Bridges. The following parameters have to be used:

Acceleration Coefficient	A = 0.2
Importance Factor	IF = 1.25

5. Impact (I), including vertical impact (VI) and horizontal impact (IH):

Impact loads are statically equivalent dynamic loads resulting from the vertical and horizontal acceleration of the life load. Vertical impact is taken as 30 percent of the rail vehicle or of the rail crane car. The horizontal impact is taken as 10 percent.

6. Centrifugal force (CF):

These are the forces that are produced by the train on the superstructure when the superstructure is curved. The centrifugal force shall be taken as a horizontal radial force equal to the percentage C of the live load. C is calculated with the formula:

$$C = \frac{S^2}{1.27R}$$

where:

S = design speed in kilometers per hour,

R = the radius of the curve of the track centerline in meters.

7. Longitudinal force (LF):

These forces are produced by the acceleration and the deceleration of the train. They are applied as uniformly distributed load over the length of the train in a horizontal plane at the top of the low rail. It is calculated as 28 percent of the live load for decelerating trains, and 14 percent of the live load for accelerating trains. It must be taken into consideration possible combinations of acceleration and deceleration where the guideway has more than one track.

8. Hunting force (HF):

This force is caused by the lateral interaction of the vehicle and the guideway due to the oscillation of the vehicle back and forth between rails. It is applied at the top of the rail head, at the lead axle of the train only, and its magnitude its taken as 12.5 percent of the live load.

9. Rolling force (RF):

It is taken as 10 percent of the live load per track, with a force applied upwards in one track and downwards on the other track.

10. Earth pressure (E):

This load is applied to structures that retain earth. Consideration shall be given for side pressure due to earth abutting against the retaining structure and load surcharges resting on abutting earth, as according to the AASHTO Standard Specification for Highway Bridges.

11. Hydrostatic pressure and buoyancy (B):

For this loads, the effects of hydrostatic pressure and buoyancy are considered whenever the presence of groundwater is indicated. Possible changes in groundwater elevation shall be taken into consideration.

12. Wind load on structure (W):

The guideway has to be designed to withstand wind loads of uniform pressure acting on the superstructure and the substructure. For the superstructure, if H is taken as the elevation of the underside of the main girder above the mean retarding surface, then:

H = 0 to 12 meters	Transverse wind pressure	$= 3.6\ kN/m^2$
	Longitudinal wind pressure	$= 0.9\ kN/m^2$
H = 12 to 18 meters	Transverse wind pressure	$= 4.0\ kN/m^2$
	Longitudinal wind pressure	$= 1.0\ kN/m^2$
H = 18 to 30 meters	Transverse wind pressure	$= 4.4\ kN/m^2$
	Longitudinal wind pressure	$= 1.1\ kN/m^2$

The substructure shall be designed to withstand the preceding loads applied to the superstructure as they are transmitted to the substructure, including the bearings. Also, an upward linear load shall be applied at the windward quarter point of the transverse width of the superstructure. Its intensity will be equal to $1.4\ kN/m^2$ of the exposed plan area of the deck and the walkway.

13. Wind load on live load (WL):

The guideway has to be designed for the effects caused by the wind load acting on the life load while the train is operating on top of the guideway. If H is taken as the elevation of the underside of the main girder above the mean retarding surface, then the loads acting on the train are taken as:

H = 0 to 12 meters	Transverse wind pressure	$= 2.4\ kN/m^2$
	Longitudinal wind pressure	$= 0.6\ kN/m^2$
H = 12 to 18 meters	Transverse wind pressure	$= 2.7\ kN/m^2$
	Longitudinal wind pressure	$= 0.7\ kN/m^2$
H = 18 to 30 meters	Transverse wind pressure	$= 2.9\ kN/m^2$
	Longitudinal wind pressure	$= 0.7\ kN/m^2$

For the case of the guideway being dual track, the loads shall be increased 30 percent when both tracks are loaded, in order to account for the shielding effect of vehicle-on-vehicle as the two trains run alongside each other.

14. Stream flow pressure and flooding (SF):

These loads take into consideration the loads applied to the structures in the case of flooding.Anticipated flood elevations shall be determined by a study of official flood records. These loads have to be applied to the guideway as applicable.

15. Creep and Shrinkage forces (R+S):

The stresses and loads caused by the creep and shrinkage of the concrete have to be considered in accordance with the AASHTO code.

16. Thermal force (T):

The stresses and deformations produced by changes in temperatures have to be considered in the design. The changes in temperature have to be taken above or below 26 ^oC. For concrete, it has to be considered a temperature rise of 11 ^oC, a temperature fall of 11 ^oC, and a coefficient of thermal expansion of 0.000011 per ^oC. In addition, a differential temperature between the top and the bottom of the box girder has to be considered. The top part of the box girder will be assumed warmer than the bottom. The differential is taken as 0 ^oC at the bottom and as either 10 ^oC or 5 ^oC at the top depending on if the thermal load is only to the dead load, or added to both the dead and live load, respectively.

For the direct fixation track, the controlled setting temperature will be between 27 ^oC and 32 ^oC, and it will be considered a temperature rise of 28 ^oC maximum, a temperature fall of 17 ^oC maximum, and a coefficient of thermal expansion of 0.000012 per ^oC. Provisions shall be made for transverse and longitudinal forces due to temperature variations in the rail. These loads shall be applied in a horizontal plane at the top of the low rail as follows:

- Transverse Force

$$T = \frac{205kN}{R}$$

where R is the radius of curvature of the guideway in meters.

- Longitudinal Force

$$T = 0.65 \times P \times L$$

where P is the longitudinal restraint force per meter (30 kN/m), and L is the effective length for longitudinal force calculation in meters. For curved track, L is measured along the curve. In this case, T can not be taken larger than 890 kN.

Wherever a continuous welded rail is terminated, provisions shall be made to fully restrain its end. This restrain shall be taken as 734 kN based on a 28 ^{o}C.

17. Forces due to rail fracture (RB):

For the direct fixation track, horizontal forces resulting from rail break shall be taken into consideration. Only the break of one rail at a time shall be considered.

18. Differential settlement (DS):

The possibility of differential settlements occurring in a foundation has to be taken into consideration, and the guideway has to be designed for the stresses and displacement that such settlement will produce. The foundations have to be designed in such a way that for girder spans up to 45 meters do not have differential settlements larger than 6 millimeters.

19. Collision loads (CL):

Piers or other guideway support elements that are situated less than 3 meters from the edge of an adjacent street or highway shall be designed to withstand a horizontal static force of 1 MN, unless it is protected by suitable barriers. This force shall be applied at an angle of 10 degrees from the direction of the road traffic, and at a height of 1.2 meters above ground level.

4.5 Possible Sources of Problems

As mentioned before, expansion joints have always been a source of problems in bridges, not only in Puerto Rico, but also in the rest of the world. To know what to expect we turn to Beneda and McKenna, whom in 1997 wrote about the aerial structure of the Metropolitan Atlanta Rapid Transit Authority (MARTA), a structure very similar to Tren Urbano:

"Almost everyone involved in the design, construction and maintenance of bridges knows that there is no perfect expansion joint seal. Failure of these joint seals dumps water directly onto the bearings, anchor bolts and substructure below where it can eventually cause critical problems. Replacement of the joint seals is very expensive, especially for transit structures. Costs are escalated because the work must be done in the trackway".

The joints being considered for the Bayamón contract have consistently failed in highway bridges in Puerto Rico. The major reason for failure is that at the time of installation, air voids form in the concrete under the edge-protection angles. With air under the angles, the angles give away, and the elastomeric material between the angles gets taken out of place. Another common reason is that the elastomeric seal is not placed correctly, and it gets hit by the vehicle wheels.

Bearings usually are not a source of problems unless expansion joints do not present any problem. In the case expansion joints fail, the metal support of the bearing could corrode, and then replacement would be needed.

Beneda and McKenna (1997) reported having mixed results with elastomeric bearings on MARTA. Many of the original bearings are still in use, but others have been replaced. From the writing it is deduced that the damage to bearings could have been prevented with proper design, manufacturing, and construction procedures.

Problems with expansion joints and bearings could be reduced by increasing the continuity of the guideway. The more continuous the guideway is, the less the number of expansion joints and bearings is. Still, reducing these sources of problems is not as easy as increasing continuity. By increasing continuity, other problems get magnified, such as loads produced by the creep and shrinkage of the concrete. Yet, the most interesting problem that could come up with increasing the continuity of the superstructure is the thermal forces that would develop in both the girder and the CWR, and how these forces would interact.

Chapter 5

Structural Analysis

5.1 Improvements Through Continuity

In Chapter 3, the complex relationship between the four objectives in bridge design was presented. It was also presented how these objectives affect certain design decisions of continuity, expansion joints and bearings. The accomplishment of the four objectives gets more complicated when it is realized that the three before mentioned design decisions are themselves interrelated. When the continuity of a bridge or guideway is increased, the number of expansion joints and bearings needed is generally reduced. Therefore, increasing or decreasing continuity of a bridge or guideway will have a complex impact that needs to be considered.

To study the impact of changing continuity in the Tren Urbano superstructure, two models of a bridge were developed. One model was used to analyze the effects of the dead loads, live loads, and seismic loads. The other models were used to analyze creep, shrinkage, and thermal effects.

The dimensions of the models are those of a typical single track section of the guideway for the Tren Urbano Project. Typical means that there is no actual section of the Project that has the exact arrangement and dimensions that the models have, but they do have what would be found on the average. It is made out of six spans, each of 36 meters long. These six spans are made of two-span-continuous units, for a total of three units

Besides these models for the two-span-continuous structure, three other pairs of models were developed. In each of pair, the continuity was changed, but the number of spans and the span length remained unchanged. These other three models developed are:

- a design in which the spans are simply supported,
- a design made out of three-span-continuous, and
- a design made out of two units, a four span-continuous with a two span continuous. This structure will be refer to from now on as the four-span-continuous structure.

Each of the eight models developed were analyzed using the program SAP2000, a finite element and structural analysis computer program.

For each pair of models, the results were combined. With these final results, the structural behavior of the four arrangements is predicted, providing a basis for comparison to define how the structural behavior gets affected with changes in continuity.

5.2 SAP2000

SAP2000 is the latest version of the series of SAP programs developed by Computers and Structures, Inc. (CSI). This series of programs is for use in the analysis and design of structures. The previous version of this series, SAP90, was used in the design process of the Bayamón contract by the consortium headed by company ICA, that includes the companies Euro Estudios and Stone & Webster Engineering Corporation. This consortium is the design/build contractor for the Bayamón contract.

Using SAP2000, a structure can be analyzed and designed by creating a model using the program's graphical user interface. The model can include the structure's properties, such as material properties, connections, and restraints. Loads can be applied to the model and the program provides results of the analysis including displacements, stresses, and reactions in the structure.

The use SAP2000 in this research, instead of SAP90, is a matter of the newer version being available to the author. SAP2000 is also more user friendly and has other modifications that SAP90 does not have (CSI, 1997). Still, these modifications are of no importance to this research, because the models used to analyze the dead loads, live loads, and seismic loads for this research are mostly based on the modeling done by ICA using SAP90. Using the same model in SAP2000 and SAP90 will still provide the same results.

5.3 Description of Models

The model analyzed in this research is based a precast prestressed segmental concrete rail transit bridge supported on spread footings with a single track. It consist of six spans with abutments at the end. Four different span configurations were studied for the bridge. For each span configuration, two models were used.

5.3.1 Model for Analysis of Dead Loads, Live Loads, and Seismic Loads

The first model (Figure 5.1) developed for each span configuration is mostly based on the model used by the consortium headed by ICA in the design of the Bayamón contract. This first model was used to study the effects in the structure due to dead loads (including prestressing), live loads, and earthquake loads.

In the Tren Urbano Fixed Facilities Category I Contract Drawings (Tren Urbano and GMAEC, 1996), it is shown that the guideway for the project will be mostly composed of two-span-continuous units, and that each span will normally be of 36 meters in length. Therefore, the reinforced concrete columns in all models were spaced 36 meters apart. There where two columns sizes taken: a column with diameter of 1.83 meters and a height of 5.9 meters, and another with diameter of 2.135 meters and a height of 6 meters. The column with the smaller dimensions is used to model the expansion joint pier. It has a slightly shorter height to allow the placement of elastomeric bearings, while keeping the superstructure at constant height.

These columns dimensions were taken from the model developed by the consortium involving ICA, because this columns have been proven to support the loads sustained during the construction of the guideway by the span-by-span erection method. These should not change, no matter what span configuration is built. Therefore, the loads can be ignored from the analysis.

Each column is modeled with one frame element. Isotropic material properties are given, meaning that the material properties are independent of the direction of loading or of the orientation of the material. Isotropic behavior is usually assumed for steel and concrete (CSI, 1997). The columns were given the properties of concrete with a strength of $f'_c = 28$ MPa (4000 psi). These properties are a density of 0.6 kg/m^3, unit weight of 5.866 kN/m^3, a modulus of elasticity of 23.42×10^6 kN/m^2, and a Poisson's ratio of 0.25.

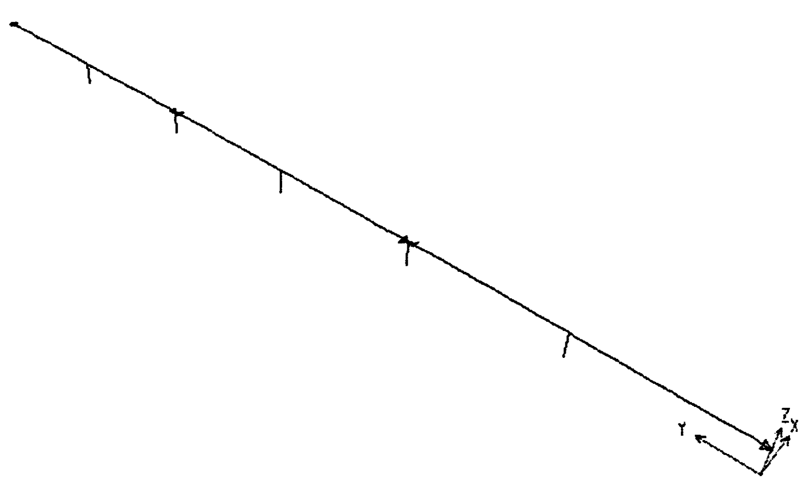

Figure 5.1: Model of two-span-continuous design for dead loads, live loads and earthquake loads.

The bottom node of each column was restricted of all movements and rotations. This is done assuming that the interaction between the spread footing and the soil behaves very closely to them being rigidly attach.

On top of the columns in which the guideway is not continuous, frame elements are used to represent rigid links and bearings (Figure5.2). Rigid links are members used to simulate the connection between the beams and the bearings, and between the bearings and the columns. Naming these elements rigid links means that they are given such properties that their stiffness is very high, therefore the elements do not suffer any deformation. Having rigid links permit the modeling of two bearings for one end of a beam unit.

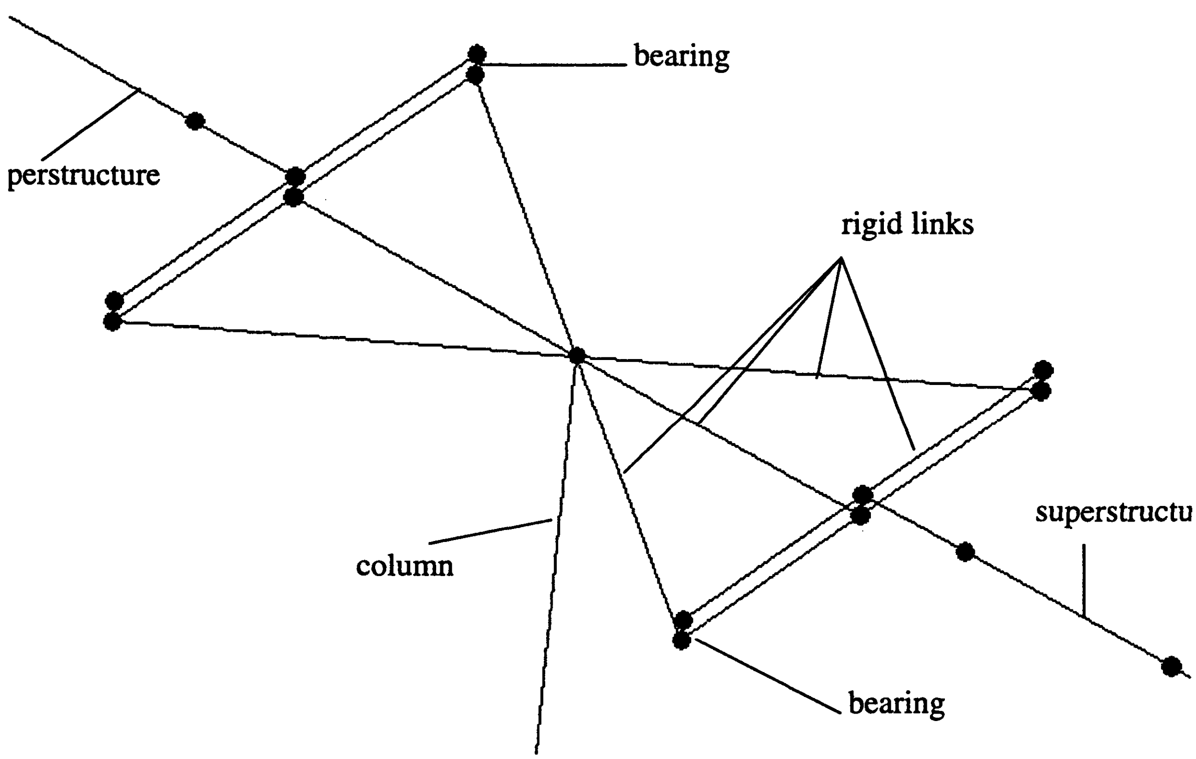

Figure 5.2: Expansion column model.

The elastomeric bearings are modeled as frame elements arranged vertically, with a length of 10 centimeters. This length of the frame element is what would be the bearing thickness. The cross-section size given to this element are the bearing's dimensions of width and length. All bearings have the same shear modulus of 1400 kPa, but each bearing's elastic modulus will depend on its dimensions. Details on how the bearing properties were obtained are presented in Appendix C.

To model the abutments, the same kind of rigid link configuration with bearings that was used for expansion piers was used. The joints at each end of the structure (that is part of this configuration) is restricted for all movements, same as it was assumed for the spread footings.

Four types of frame elements were used to model the precast segments. The types One and Two are used to model the joint segment, each type represents one of the joint segment's two different cross sections. The type Three is used to model the typical segments. One Type Three element represents one typical segment. Type Four is used to model the pier segment. Two Type four elements are used to model one pier segment. This is to allow the rigid connection of the pier segment and the column to occur at the mid-length of the segment. Each type was given the corresponding properties of the cross-section they represent.

Consecutive elements representing consecutive precast segments share one joint. Eleven typical elements were placed in each of the model six spans. The number of elements used to represent pier segments and expansion segments depend on the span configuration of the structure. All frame elements representing precast segments where given isotropic material properties corresponding to concrete of strength of f'_c = 35 MPa (5000 psi). These properties are a density of 2.4 kg/m^3, a unit weight of 23.544 kN/m^3, a modulus of elasticity of 27.67x10^6 kN/m^2, a Poisson's ratio of 0.30.

5.3.2 Model for Analysis of Creep, Shrinkage, and Thermal Effects

The second model is a two-dimensional (Figure 5.3). It was use to model the effects of creep, shrinkage, and changes in temperature.

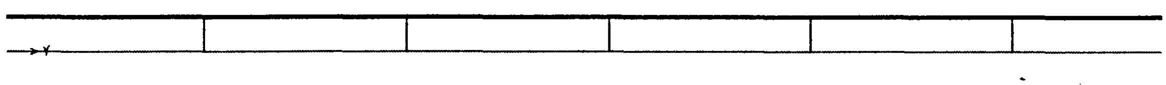

Figure 5.3: Model of two-span-continuous design for creep, shrinkage, and temperature effects.

The modeling of the columns is done in the same way that it was done for the first model. A change made from one model to the other is that only one rigid link is used to connect expansion joint piers with the elastomeric bearing, but each rigid link is modeled with ten frame elements, to model the rail superstructure/interaction, as it will be explained later in this section. The joint of each rigid link at the each end of this structure is restricted from all movements, to account for the abutments. Also, only one frame element is used to model a pair of bearings. The cross sectional properties of the bearing models were adjusted to do so.

A critical subject in the modeling, in order to perform a successful analysis of the effects of creep, shrinkage, and temperature changes, is representing the superstructure/rail interaction. Fryba (1996) suggested that the connection between the CWRs and the superstructure is idealized by a system of infinitely close horizontal springs. Fryba offers typical values to be taken for the stiffness per unit length of the horizontal spring (Figure 5.4). For prestressed concrete bridge directly fixed to the structure (without ballast), a stiffness per unit length of 8 N mm^{-2} is suggested.

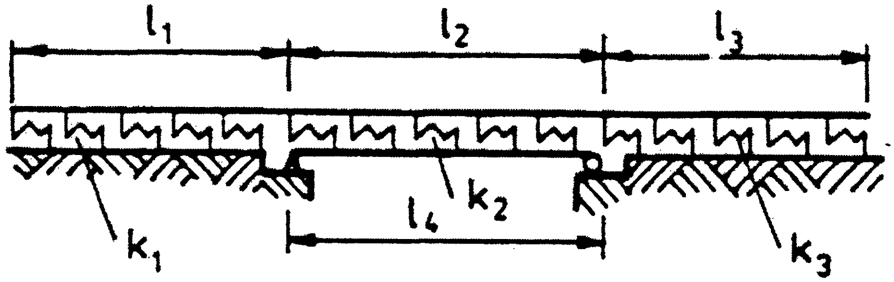

Figure 5.4: Idealization of the rail/superstructure interaction. (Fryba, 1996)

First, to model the superstructure/rail interaction for this research, the beams were modeled with the same four types of frame elements used in the first model, but this time, ten elements were used where only one element was used in the first model. Also, another property was added to the frame elements representing the segments: a coefficient of thermal expansion of 11x10^{-6} per Celsius degree.

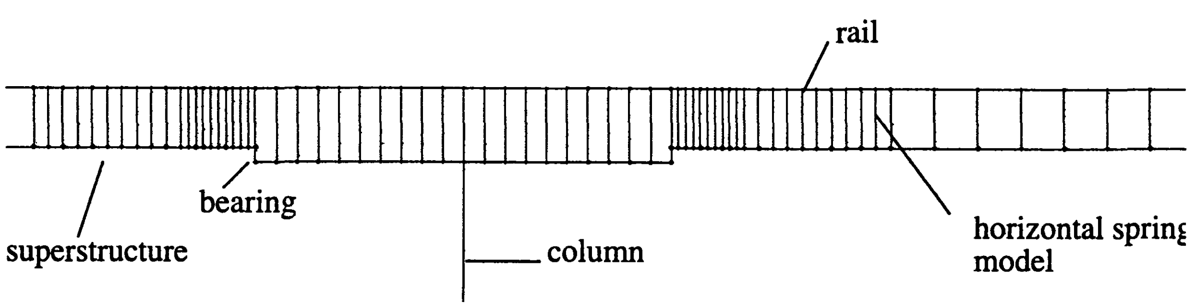

Figure 5.5: Modeling of rail/superstructure interaction using SAP2000.

Second, on top of each frame element that represents either the superstructure or a rigid link, a corresponding new frame element of the same length was place parallel at an arbitrary chosen distance of 40 cm from the superstructure, and a distance of 50 cm of the rigid links (which are located 10 cm below the superstructure). These new frame elements represent the rails. The area and the moment of inertia of each of this elements is that of the double of a typical 115 RE as specified by the American Railway Engineering Association (AREA). Also, typical properties for the rails were given: a modulus of Elasticity of 2.1×10^5 MPa, a Poisson's ratio of 0.30, and a coefficient of thermal expansion of 12×10^{-6} / ^{o}C. Both ends of the rail were restricted of all displacements.

Finally, the infinitely horizontal springs were modeled with vertical frame elements (Figure 5.5), connecting each of the joints of the rigid links and the superstructure to a corresponding joint of the rail directly on top. In order for them to act as a horizontal spring, a large area was given to each the vertical elements, so that they act as bending beams. Then the properties of the vertical element were fixed so that the beam acts as a horizontal spring of stiffness per unit length of 8 N mm^{-2}. This is done by equating:

$$k \times s = \frac{12 \times EI}{h^3}$$

where:

k = the stiffness per unit length of the horizontal spring;

s = distance between vertical elements;

E = modulus of elasticity for the vertical elements;

I = moment of inertia of the vertical element;

h = height of the vertical element.

The value of k is given, and the values of s and h are defined by the geometry of this model. An arbitrary value of E of 8 N mm^{-2} was chosen, and it was solved for I, assigning this value to the vertical element this result.

5.4 Modeling of Loads

As mentioned before, for each span arrangement, two models were developed using SAP2000. For the first model, the loads included in this analysis are dead loads (including prestressing), live loads (specifically, loads caused by the train, including vertical impact), and earthquake loads. Loads caused by creep, shrinkage, and temperature changes are analyzed in the second model. The results of the analysis of each corresponding pair of models are added to determine how a span arrangement will work.

Other loads could be part of the analysis (like wind loads), but they are not going to be considered here. This is done because by experience, the load combination that will determine the design of the elements when using either, Service Load Design or Load Factor Design, of a bridge with such a small height as this one, located in a Seismic Zone 3 is:

$$\text{Group VIII} = DL + LL + IV + B + SF + E + EQ$$

For the case analyzed here, B, SF and E are equal to zero. (Refer to Section 4.4 for load definitions). EQ has to be analyzed in both the transversal and longitudinal direction of the structure. The one that produces the most critical loading is used in the design procedure. Also, its written in the Tren Urbano Design Criteria (United States Department of Transportation, et al., 1996): "The permanent effect of creep (R) and shrinkage (R) shall be added to all service load and load factor design group loading combinations with a load factor of 1.0".

5.4.1 Dead Loads

The dead loads include the weight of the beams, columns, pier caps for the columns with bearings on top, the acoustical barrier, and the emergency walkway. The weight of the beams and columns were assigned with the material properties for its frame elements. The weight of the pier caps is represented as a concentrated load of 421.44 kN acting downward on top of columns with pier caps. The weight of the acoustical barrier and the emergency walkway is represented as a distributed load on top of the precast segments of 39.127 kN/m, as calculated by the consortium headed by ICA.

Also included in the dead loads are the prestressing loads. These loads are difficult to model, so assumptions were taken to simplify the analysis.

The major assumption made was that the each span is prestressed with just one tendon placed inside the beam with a parabolic profile and tensioned at the center of gravity of the beam cross-section. In reality, at least eight tendons are used to prestress one span. This simplification allows the modeling of the prestress with equivalent loads.

With the simplification, a prestressing force P is chosen. The effects of this load are model with equivalent loads. Theses are two downward loads at each end of the span, a horizontal load acting in compression at each end of the span, and a distributed load acting upward (Figure 5.6). The magnitude of the downward load is P $\sin\theta$ and the magnitude of the horizontal load is $P \cos\theta$, where θ is the angle from the horizontal plane at which the prestressing load acts. This angle was estimated from Tren Urbano drawings (Tren Urbano and et al., 1986) as being 5.86 degrees.

To calculate the distributed load, the following equation is used:

$$Pe = \frac{wl^2}{8}$$

where:

P = the prestressing force;

e = the eccentricity of the tendon from the center of gravity of the beam cross-section at the center of the beam.;

l = the beam length;

w = the equivalent distributed load produced by the prestressing.

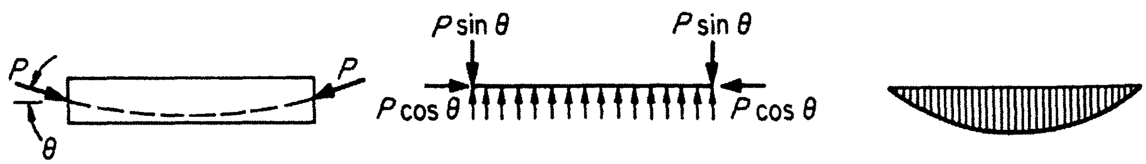

Figure 5.6: Equivalent loads for prestressing. (Nilson and Winter, 1987)

In the above equation, P is given, l is known, and e is estimated from drawings to be 1.07 meters. The equation is solved for w and this result is included in the loading of the model.

5.4.2 Live Loads

The effects of the live load is represented by performing a Bridge Analysis with the program SAP2000. First, the track of the train is modeled by specifying a lane on top of the precast segments, and on top of the rigid links aligned with the models of the segments.

Second, three kinds of Vehicles were defined. The first is made out of one pair of Rail Vehicles (RV), the second is made with two pairs of RV's, and the third one is composed of three pairs. Each RV has four axles, weighing each 146 kN. The separation between axes were taken from the Tren Urbano Design Criteria (United States Department of Transportation, et al., 1996), and are shown in Figure 4.6. The axle load given is taken by the program as a maximum value, meaning that SAP2000 will change this value, trying to find the most critical response of the structure. This is done to take into account for cases in which the train is not fully loaded (CSI, 1997). The Rail Crane Car (RC) is not included in this analysis because it has a lower weight than the Rail Vehicle.

Third, a Vehicle Class including the three modeled vehicles was defined, and finally, a Moving Load case was also defined. SAP2000 performs the analysis by taking all the assignments in a Moving Load case, trying every possible permutation of loading. The program gives as a result the maximum and minimum response quantities for the permutations of a Moving Load case as permitted by vehicle and lane assignments (CSI, 1997).

To include the effects of the vertical impact, as scale factor of 1.30 was assigned to the Moving Load case. This adds 30% to the live load, the specified load for the vertical impact.

5.4.3 Earthquake Loads

To perform the seismic analysis, the program SAP2000 uses the masses specified in the different elements of the model. By specifying density for the columns and precast segments, the program calculates lumped masses for the joints in the model. To account for the emergency walkway and the noise barrier, a distributed mass of 399 kg/m was added to the precast elements. To account for the piercaps, a mass of 429.6 kg was assigned to the top node of each column with a pier cap.

The response spectrum used for this analysis was calculated according to AASHTO (1996), using an acceleration coefficient of A = 0.2, a soil type II, and a structure importance factor of 1 (Figure 5.7). The response spectrum was inputted in SAP2000. The analysis is performed twice, first applying seismic forces in the transversal direction of the bridge, and then in the longitudinal direction.

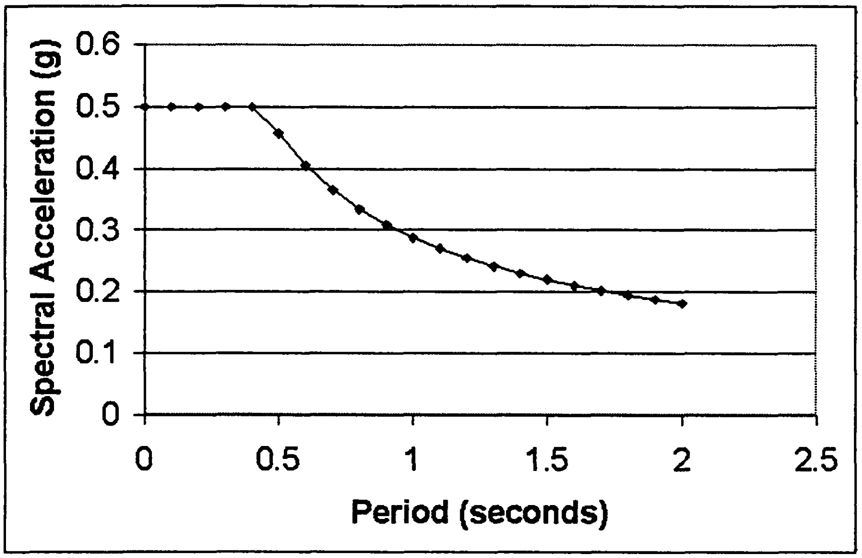

Figure 5.7: Response spectrum. (Euro Estudios, 1997)

The program performs a modal analysis after a 5 percent damping was assigned to each mode. SAP2000 solves for each mode performing an eigenvector analysis.

5.4.4 Creep and Shrinkage

Creep and shrinkage are going to produce a shortening of the beams in the structure. The Precast/Prestressed Concrete Institute (1992) offers a procedure to approximate the strain shortening due to these time dependent effects. Knowing that the design temperature change for Tren Urbano is 22°C, and that the average ambient relative humidity is 78%, calculations were performed using the PCI procedures, for different spans configurations. For all calculations, the total strain due to creep and shrinkage fell between $4.5\text{x}10^{-4}$ and $5\text{x}10^{-4}$.

Using SAP2000, strains can not be included in an analysis directly. Indirectly, these effects can be accounted for by representing them as an equivalent temperature drop. This is done by equating:

$$\alpha \Delta T L = \varepsilon L$$

where:

α = coefficient of thermal expansion

ΔT = temperature change

L = length of the beam

ε = strain due to creep and shrinkage.

Taking a strain of 5×10^{-4}, and the thermal coefficient of concrete, it is solve for the change in temperature. A result of 45°C was obtained and used for all models.

5.4.5 Temperature Effects

The first step in modeling temperature loads was assigning a reference temperature to the frame elements representing the precast segments and the CWRs. This temperature, as specified in the Tren Urbano Design Criteria, is 26 °C for the precast segments, and between 27°C and 32°C (it does not matter what temperature is chosen for the steel).

Then, a temperature drop of 11°C is added to the equivalent temperature fall calculated for the precast segments to account for creep and shrinkage. For the steel, a drop of 17°C is assigned. Also, a gradient of 10°C is assigned to the concrete segments to account for the top slab getting warmer than the bottom slab. A temperature drop was assigned in this analysis, because it will make the effects of creep and shrinkage more critical than a temperature rise.

5.5 Analysis Process

After the input for SAP2000 have been prepared with the model and the loading, the program was run. With the results provided by the program, the maximum shear deformation of the bearings due to the most critical combination of loads (dead load, live load, vertical impact, creep, shrinkage, temperature effects, and either a longitudinal or a transversal earthquake) was checked. AASHTO limits this deformation to one half the bearing thickness. Therefore it was checked that the bearings would not deform more that 5 cm. Sizes of the bearings were adjusted to comply with this specification.

After a satisfactory bearing size has been found, the program was run again. After the results were obtained, the stresses occurring in the precast segments were checked. It is desirable that the concrete is always in compression, so the prestressing force was adjusted trying to reduce to a minimum the tensile forces in the superstructure. When an optimum in prestressing forces was found, the reinforcing steel (with yield strength of $f_y = 60$ ksi) necessary to account for tensile forces in the superstructure was calculated.

Then, the stresses occurring in the rail were checked. This stresses were obtained from the analysis of the model used for creep, shrinkage, and thermal effects, and by performing a simple calculation to determine the effects of the live load on the rail. More details on these calculations are presented in Appendix G.

Later, the loads acting on the elements representing the columns were used to find the reinforcement that is going to be needed for the columns. Details of the design of reinforcement for the columns are presented in Appendix D.

Finally, the loads at the bottom of the columns were used to design spread footings. It was assumed that the soil on which the foundation is built is leveled all through the length of the bridge, for purpose of calculating the depth of the foundation. Typical soil condition for situations where spread footings are used in Puerto Rico were assumed. In Appendix E, the design procedure used for spread footings is presented.

5.6 Results

In this section, the results that are presented were obtained after combining the effects of the two models developed for each of the four analyzed structures.

For the three- and four-span-continuous designs, it was necessary to decrease the area of the columns that are fixed to the superstructure, in order to increase their flexibility. This was very useful due to the large forces due to creep, shrinkage, and temperature, as it will be seen in this section. The diameter used for all columns in these designs is 1.83 meters

In Table 5.1, the width and length dimensions selected are show. It is interesting to see, that the largest dimensions are required by the simple span structure, as expected, but it was surprising that the smaller dimensions were needed by the two-span-continuous design. It was expected that the four-span-continuous structure required the smaller dimensions.

As mentioned before, the four-span-continuous structure is the combination of a four-span-continuous unit with a two-span-continuous unit. In this structure, a smaller bearing size of 0.55m x 0.60m is needed for the two-span-continuous unit. It was not listed in Table 1 because the reason this configuration was developed is of to study the behavior of the four-span-continuous unit in this structure.

After the bearings dimension were adequately selected, the structures were analyzed, and the stresses in the superstructure were checked. In the Tren Urbano Project, compression stresses are not allowed to exceed 0.4 f'_c. When the results of the four-span-continuous structure were revised, the compression stresses exceeded this maximum allowable in certain sections of the superstructure. Therefore a change was needed in the design of this structure. The solution used in this research was increasing the strength of the concrete. The strength of the concrete was increased from 35 MPa (5000 psi) to 48 MPa (7000 psi). This probably was not the best solution. A better alternative would have been to increase the superficial area of the cross-section, therefore increasing the stress capacity of the superstructure. Difficulty and constraints in times did not permit the implementation of this solution in this research.

Table 5.1: Bearings dimensions.

Structure	L (m)	B (m)
simple span	1.2	1.2
two-span-continuous	.75	.75
three-span-continuous	.85	.85
four-span-continuous	1.05	1.05

When the creep, shrinkage, and thermal effects were ignored, there were no significant changes in the stresses acting on the superstructure. Including these effects in the analysis is of extreme importance, because the magnitudes of the stresses they produce increase drastically with increasing continuity.

In the three-span-continuous and in the four-span-continuous structure, large loads due to creep, shrinkage, and thermal effects (including the interaction of superstructure with the structure) where found in the spans that have both fixed ends. The maximum loads in the superstructure due to creep, shrinkage and temperature effects are shown in Table 5.2. Notice the drastic increase in loading that occurs from the two-span-continuous to the three-span-continuous structure.

Table 5.2: Maximum loads in the superstructure due to creep, shrinkage and temperature.

Structure	Axial Load (kN)	Moment (kN m)
simple span	371	3773
two-span-continuous	271	4198
three-span-continuous	4,211	9,618
four-span-continuous	10,346	20,518

Meanwhile, it can be seen in Table 5.3 that the stresses in the rail practically remain unaffected with changes in the continuity of the superstructure. This maximum values shown in here were found in the areas of the rail that are on of expansion piers, where the end of two beams meet and expand in different directions.

Table 5.3: Maximum stresses in the rail.

Structure	Compression (kPa)	Tension (kPa)
simple span	219,126	102,888
two-span-continuous	219,256	101,369
three-span-continuous	217,990	102,022
four-span-continuous	218,242	100,537

The prestressing loads P that were applied to the superstructure are shown in Table 5.4. The simple span structure needs a high prestressing load to reduce the deflections of the beams at midspan. The continuity in the two-span-continuous structure permits the reduction in prestressing load. In the three-span continuous structure, spans 2 and 5 have fix ends, as do spans 2 and 3 in the four-span-continuous structure. Creep, shrinkage and thermal effects required that these spans had their prestressing load increased, although the other spans of these structures could maintained the lower prestressing loads of the two-span-continuous structure.

Table 5.4: Applied prestressing load (kN).

Structure	Span 1	Span 2	Span 3	Span 4	Span 5	Span 6
simple span	160	160	160	160	160	160
two-span-continuous	130	130	130	130	130	130
three-span-continuous	130	160	130	130	160	130
four-span-continuous	130	160	160	130	130	130

A measure of the reinforcement needed in the columns for each structure is presented in Table 5.5. It can be seen that the required reinforcement increases drastically for piers 1, 2, 4, and 5 in the three-span-continuous structure, and in piers 1 and 3 in the four-span-continuous structure. These piers are rigidly connected to the superstructure. Again, this drastic increases are due to the large loads produced by creep, shrinkage, and temperature changes.

Table 5.5: Gross reinforcing ratio of columns.

Structure	Pier 1	Pier 2	Pier 3	Pier 4	Pier 5
simple span	0.01	0.01	0.01	0.01	0.01
two-span-continuous	0.01	0.01	0.01	0.01	0.01
three-span-continuous	0.05	0.05	0.01	0.05	0.05
four-span-continuous	0.08	0.01	0.08	0.01	0.01

Of course, the higher loads in the columns produce higher loads in the foundations. In Table 5.6 it can be seen that the required dimensions are larger for the foundation that support the columns were there was a drastic increase in reinforcement.

Table 5.6: Spread footing dimensions of width and length in meters.

Structure	Pier 1	Pier 2	Pier 3	Pier 4	Pier 5
simple span	4x6	4x7	4x7	4x7	4x6
two-span-continuous	6x7	4x4	6x7	4x4	6x7
three-span-continuous	10x5	11x5	3x6	11x5	10x5
four-span-continuous	13x7	11x5	15x5	3x5	6x7

Chapter 6

Life-Cycle Costing

6.1 Bridge Economy

A common error made when estimating the cost of a bridge is to include only the costs of construction, primarily the costs of materials and labor. This error leads to the design of a bridge with minimum construction costs that fulfills only immediate safety and serviceability requirements. Usually this is the type of bridge that will need a lot of inspection, maintenance, and, probably, a major rehabilitation, in order for it to keep fulfilling the safety and serviceability requirements.

The cost-effectiveness of a bridge can not based only on the construction cost. The economy of a bridge is best evaluated using the life-cycle cost (Menn, 1990). Life-cycle cost analysis is defined by the The American Institute of Architects (1977) as any technique which allows assessment of a given solution or choice among alternative solutions on the basis of considering all relevant economic consequences over a given period of time (or life-cycle). In the case of a bridge or guideway, the life-cycle cost should at least include the costs of construction and operation. Other costs can be included in the analysis, such as costs of major rehabilitation, and demolition. Brito and Branco's (1994) life-cycle model (which they called global cost analysis) includes the benefits a bridge provides in offering a better service for traffic.

Life-cycle cost analysis can be performed during different stages of the construction process, including planning and design, construction, and operation. It is in the design and planning phase of a project where the life-cycle cost analysis has the greatest potential to produce savings (Arditi and Messiha, 1996). When the project gets into the construction and operation phase, it is more difficult to make a change in the bridge that will bring savings. In addition, these changes at this point are more costly.

The cost of operation includes costs incurred in inspection, maintenance, and rehabilitation. When calculating life-cycle cost, operating costs are usually represented as an equivalent annual expenditure. Usually, as explained before, annual operating expenditures will be higher for bridges that have been designed solely to minimize construction costs.

Although it seems advantageous to perform a life-cycle cost analysis for the construction of bridges, there are many bridge designers who do not implement this method. Ignoring life-cycle cost is very common among bridge designers in Puerto Rico, and for that matter, designers of the Tren Urbano Project. The reasons for this could be (Arditi and Messiha, 1996):

1. The lack of standards or formal guidelines for the performance of a life-cycle cost analysis.
2. The lack of reliable past data.
3. The disability to determine future costs and factors.
4. Focusing on present costs, and lack of concern for future cost implications.

The second reason is especially true for the Tren Urbano Project, since nothing like this project has ever been built in Puerto Rico. Still, similar projects have been built in the United States and in other countries, and many of the private design and construction companies involved have also worked on similar projects.

6.2 Effects of Increasing Continuity on the Life-Cycle Cost of a Bridge

As it was seen in Chapter 5, increasing continuity of a bridge design can cause major changes in the design. Foundations sizes can change, beam cross-sections sizes could be changed, the amount of reinforcing and prestressing steel could be changed.

The reader must not make the mistake of generalizing the results of Chapter 5 to all other projects. For example, in this research it was found that the sizes of the spread footings increased with the increase of bridge continuity. The main reasons for this increase were the high loads induced by creep, shrinkage and temperature effects. Foundation sizes do not necessary have to increased in other kind of bridges. For example, it is possible that the problem of creep and shrinkage could have been avoided by not connecting rigidly the columns with the superstructure. In any case, the increase in continuity of a bridge or a guideway will most certainly cause a change in the consumption of materials, which means a change in the construction costs.

One thing is certain: when continuity is increased, the number of expansion joints decreases. No expansion joint last forever. Therefore, they need to be inspected and replaced when damaged. Failure to do so may result in damages to the bridge structure that could be very costly. It can be safely concluded that with increase in continuity, there is a reduction in operation costs.

Also, with an increase in continuity, the number of bearings is reduced, depending on whether the continuous beam units are integral with the columns. It is usually said that if designed properly, bearings can last the lifetime of the bridge. Still, bearings have failed in many cases. Therefore, bearings need to be inspected, and sometimes replaced. If bearings can be reduced through an increase in continuity, operation costs can be further lowered.

6.3 Life-Cycle Cost Analysis

In this section, a simplified life-cycle cost analysis is performed for each of the structures analyzed in Chapter 5. The costs used in the analysis were taken from a list of costs prepared by the Bridge Department of the Massachusetts Highway Department in order to estimate the construction costs of a future project. These costs are presented in Table 6.1.

Table 6.1: Unit costs (Source: Massachusetts Highway Department)

Material	Unit Price
Prestressed Concrete Box Beam- f'_c=35 MPa	\$452.13 per surface square foot
Prestressed Concrete Box Beam - f'_c=48 MPa	\$484.42 per surface square foot
Prestressing Steel - ASTM, Grade 270	\$38,137.26 per cubic meter
Reinforcing Steel - f_y=60 ksi	\$19,068.63 per cubic meter
Elastomeric Bearings	\$61,032.68 per cubic meter
Expansion Joints	\$1,049.92 per linear meter
Concrete - f'_c=28 MPa	\$459.34 per cubic meter
Concrete -f'_c=35 MPa	\$548.83 per cubic meter
Concrete -f'_c=35 MPa	\$548.83 per cubic meter

In the calculation of construction costs for this analysis, labor costs and other material costs were not included in the analysis. These costs were ignored because they would be roughly the same for all four designs. Therefore they will not affect a comparison of costs between the structures.

First, the cost of the precast segments was calculated. This was obtained by calculating the total surface area of the top of the superstructure. This cost is only different to the four-span-continuous structure because a higher strength concrete was used.

Second, the cost of prestressing steel was estimated. The tendons to be used, as specified by the Tren Urbano Design Criteria (United States Department of Transportation, et al., 1996), are ASTM A416, Grade 270. Their maximum allowable stress at anchorage is 1300 MPa. The area of the tendons required for each structure was obtained by dividing the prestressing load by the allowable stress. Then, the tendon length needed for each span was approximated and multiplied by the area to obtain the volume of prestressing steel.

Third, the reinforcing steel area required for the tensile stresses in the precast segments was considered. It was assumed that the reinforcing bars were going to be placed through the length of the precast segments, therefore allowing the calculation of the total steel volume.

Fourth, knowing the bearings dimensions, the volume of each bearing was calculated. Considering the number of bearings used in each design, the total bearing volume was calculated for each structure.

Fifth, the number of expansion joints needed for each structure was calculated. By multiplying this number by the top width of the cross-section of the segments (5.7 meters), the total expansion joint length was obtained.

Sixth, the total cost of the columns was obtained by computing the required steel volume of each column and the required concrete volume. By multiplying the required gross steel ratio (obtained in the analysis presented in Chapter 5) by the gross area of the column, and then multiplying the result by the height of the column, the steel volume was estimated. Then, subtracting this volume off the gross column volume, an estimate of the concrete volume was obtained. For all columns, concrete with $f'_c = 28$ MPa was used, except for the four-span-continuous structure where concrete with $f'_c = 35$ MPa was used.

Finally, costs of the foundations are calculated using the results of the analysis presented in Chapter 5. With the obtained dimensions, the volume of excavation and the volume of the concrete needed for the foundation were calculated. For all foundations, concrete of strength $f'_c = 28$ MPa was used.

In the calculation of operation costs, two assumption were made: all expansion joints of each structure would need to be replaced every ten years, and half of the bearings of each structure would be replaced at half the life of the project. The life of Tren Urbano Project is taken as 50 years. Therefore, it is expected that bearings will be replaced at years 10, 20, 30, and 45 of the project, while half of the bearings will be replaced at year 25.

Each of the maintenance costs were changed into present costs with the equation:

$$P = F(1+i)^{-n}$$

where:

P = present cost;

F = future cost;

i = annual interest rate;

n = year when the future cost takes place.

For the interest rate i, effects of inflation can be considered. Different values of i were used in the analysis to observe its effect in the maintenance cost.

Finally, the construction costs and the maintenance costs were added to obtain the total cost of each structure considered.

6.4 Results

Table 6.2 lists the detailed construction costs of each six-span bridge analyzed. It shows how the different construction costs get impacted by changing the continuity of a structure. Of interest is the increase in costs of columns and footings with increasing continuity, and the decrease of expansion joints with an increase in continuity.

Table 6.2: Detailed construction costs for each analyzed bridge design.

Structure	Precast Segments	Steel Tendons	Rebars	Bearings	Expansion Joints	Columns	Footings
simple span	\$556,662	\$938	\$5,573	\$210,928	\$71,815	\$48,735	\$76,210
two-span-continuous	\$556,662	\$794	\$55,724	\$41,197	\$35,907	\$59,969	\$116,678
three-span-continuous	\$556,662	\$849	\$43,567	\$35,276	\$23,938	\$97,748	\$214,832
four-span-continuous	\$596,418	\$849	\$111,907	\$34,972	\$23,938	\$98,647	\$260,158

In Table 6.3, the total construction costs are listed for each of the analyzed bridge designs. The two-span-continuous structure is the design with the lowest construction cost total.

Table 6.3: Total construction costs for each analyzed bridge design.

Structure	Construction Costs
simple span	\$970,862
two-span-continuous	\$866,931
three-span-continuous	\$972,964
four-span-continuous	\$1,126,887

The operating costs at different interest rates are shown in Figure 6.1 for each design. It can be seen that an increase in continuity produces a saving in operating costs.

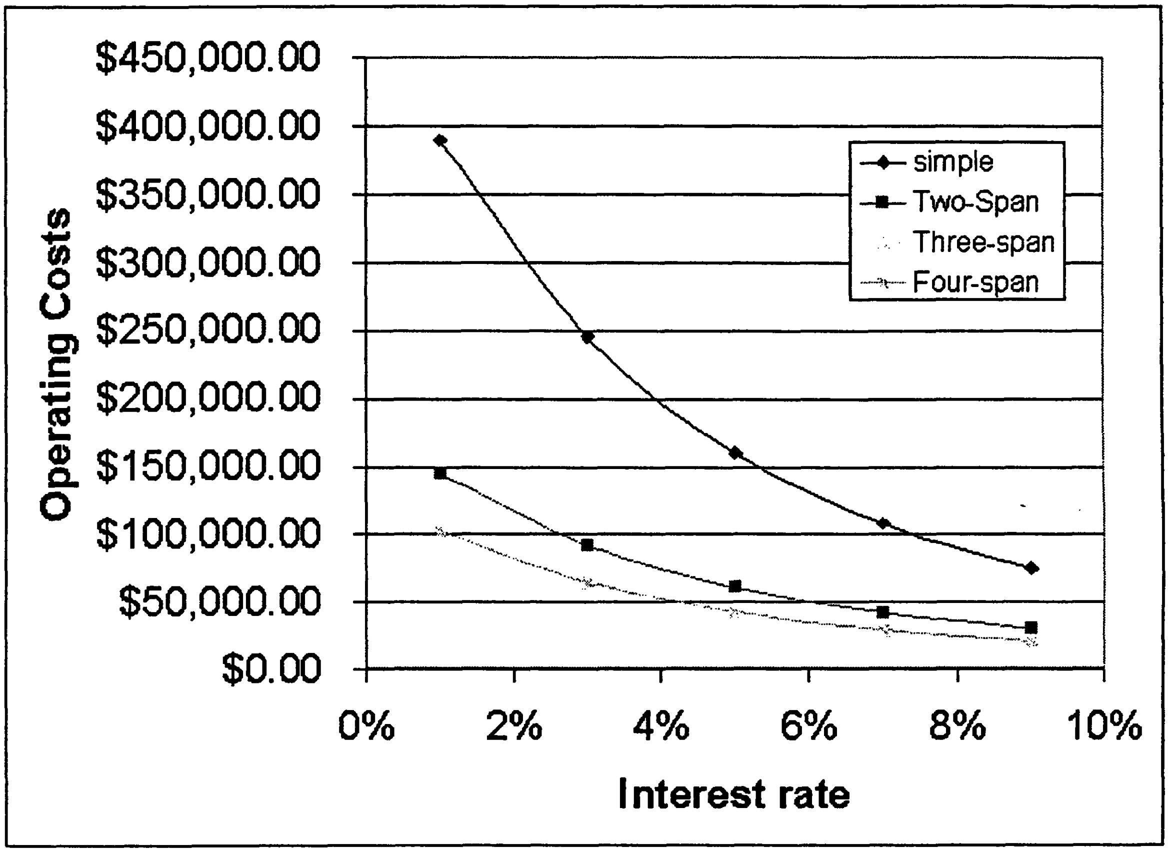

Figure 6.1: Sensitivity analysis for operating costs.

In Figure 6.2, the results of the life-cycle cost analysis are presented. The effects of using different values of *i* in the analysis can be appreciated. At all values of *i*, the two-span-continuous structure remains the most economic structure, while the three-span-continuous structure remains the second most economical design. Although it was the design with the second lowest construction cost, the simple span structure has a higher total cost than the four-span-continuous structure when the value of *i* is less than 4%.

Strictly speaking in economic terms, the two-span-continuous structure turns out to be the best design. It is the structure that uses both the advantages of continuity, and minimizes the effects of creep, shrinkage, and expansion joints.

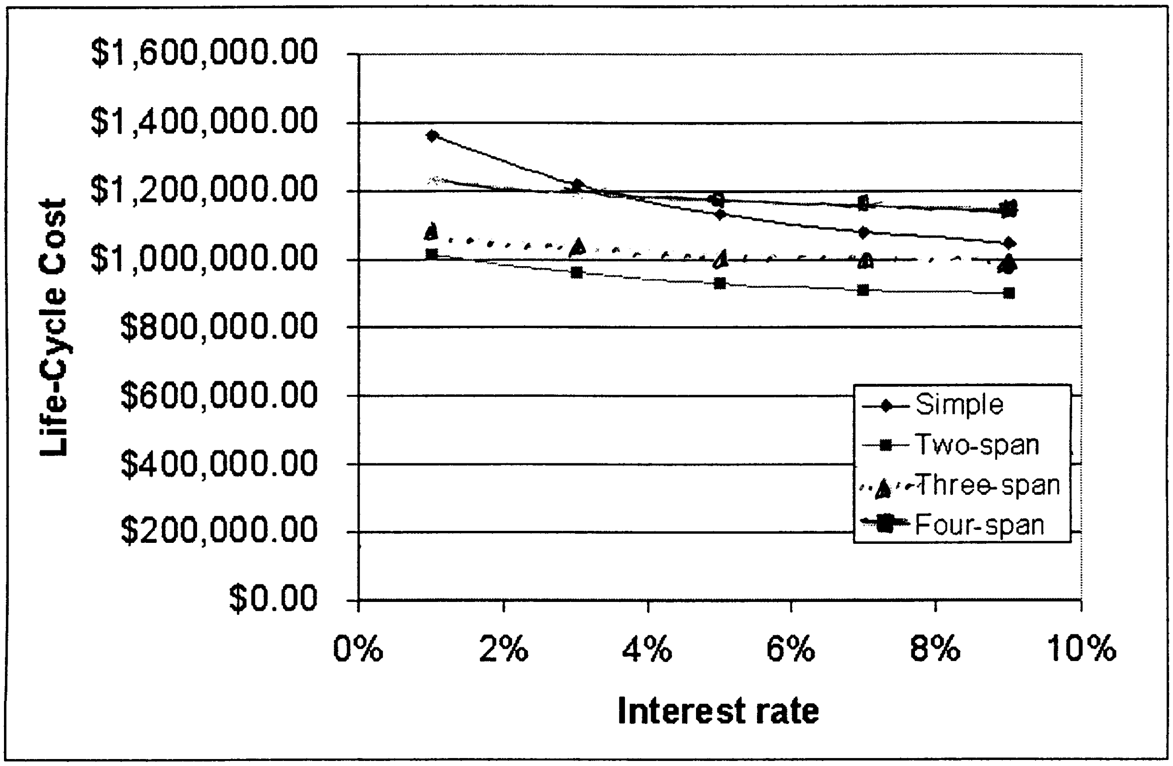

Figure 6.2: Sensitivity analysis for life-cycle cost.

Chapter 7

Increasing Continuity for Tren Urbano

7.1 Introduction

It has been shown in Chapter 5, that continuity of the guideway design of the Bayamón contract of Tren Urbano can be increased to at least three spans, without changing the strength of the concrete used and the yield strength of the reinforcing steel. Also shown is that a four-span-continuous unit could be constructed if a higher strength concrete is used, or different alternatives are considered, like increasing the area of the cross-section of the precast segments. A different kind of foundation could be considered, such as piles.

Changing the continuity in the Tren Urbano Project would have some impact on the achievement of the four objectives of bridge design. In this chapter, these effects and their interrelation are presented.

7.2 Safety

Any design must be safe, no matter what span continuity is adopted. It cannot be allowed to fail under design loads. Yet, things go wrong and designs can fail. In the event of there being a failure, the more continuous the structure is, the safer it will be because of its redundancy.

When using simple spans, if a failure were to occur in the superstructure, there would be a large deflection at midspan, and the columns would not be able to offer much resistance to reduce this deflection, and there could be a collapse in the structure. Meanwhile, if the superstructure were continuous, the columns would be able to offer more resistance against there being larger deflections. The more continuous the structure is, the more columns that would be able to act in the resistance against deflections.

Another advantage of redundancy is more protection if a column were to fail. In the case that simple spans were used, if a column were to collapse, the spans that it supports would collapse too. In the case of having continuous units, if a column were to collapse, there is a better chance that the beam or beams that the column supports will not collapse, because other columns continue to support the continuous unit.

One must not conclude from the discussion presented above that continuous units are safer for Tren Urbano. It was found in the analysis presented in Chapter 5 that with the increase in continuity, there are going to be increased stresses in the precast units. Again, this is due to there being columns monolithic with the continuous units, which in turn causes the development of stresses due to creep, shrinkage, temperature changes, and rail/ superstructure interaction. Therefore, stresses in the superstructure get closer to the allowable stresses with increasing continuity.

7.3 Serviceability

Increasing continuity would definitely be an improvement to the serviceability of Tren Urbano. The first obvious reason for this is that the number of expansion joints would be reduced.

Expansion joints have always been a source of problems for bridges. No expansion joint lasts forever. Therefore they need to be inspected periodically, because an expansion joint failure, when undetected, can cause damages to the concrete and reinforcing steel of the beams and pier cap, and to the bearings and their supports.

Imagine having a rail transit guideway of 30 spans (29 columns plus two abutments) with a similar design to that of the Bayamón contract. If simple spans were used, the guideway would have 60 expansion joints (two per columns plus one per abutment). All these expansion joints need to be inspected and at some point be replaced. All 60 expansion joints could damage the structure if they fail. Instead, if two-span-continuous units are used, the number of expansion joints would be reduced to 30, and if three-span-continuous units were used, the number of expansion joints would be further reduced to 20, and so on...

Therefore, the increase of continuity would decrease the number of expansion joints. This would decrease the required inspection of the structure. Since there would be fewer expansion joints to be repaired or replaced, the required maintenance is also reduced. Also, there is less possibility of the bridge structure suffering any major damage due to leakage.

The second advantage of increasing continuity in the Tren Urbano Project is that the number of elastomeric bearings can be reduced. Bearings are not a great source of problems as the expansion joints are, but they still need to be inspected, and sometimes replaced. It is important that bearings remain fit to handle the displacements of the bridge structure, such as those caused by creep, shrinkage, temperature and seismic loads. When this is not done, there is a great risk that the design loads will exceed the allowable stresses and a localized failure will occur.

Returning to the example of the 30 span guideway mentioned above, if simple spans were used, 120 bearings would be needed. If two-span-continuous units were used, the number of bearings would be reduced to 60, the number of bearings would be further reduced to 40 if three-span-continuous units are used, and so on... Therefore, increasing continuity reduces both the time spent on maintenance of bearings, and the possibility of there being bearings that need replacement (assuming that the number of bearings that needs to be replaced in a bridge is directly proportional to the number of bearings the bridge possesses).

By increasing continuity and reducing the number of expansion joints and bearings, another aspect of serviceability gets affected. The interruption of the rail transit system due to the replacement of bearings and expansion joints is minimized. It is anticipated that Tren Urbano will operate 20 hours a day, providing little opportunity to replace bearings and expansions joints without interrupting service.

Still, there is one more improvement obtained with an increase in continuity: the more continuous the structure, the smoother the ride. Every time there is a break in the continuity of the structure, there is also a break in the continuity of the deflections of the structure. These breaks make the ride less smooth.

7.4 Economy

It was shown in Chapter 6 that two things are likely to happen if the continuity is increased in Tren Urbano. First, that the construction costs will be higher, and second, that the operation costs of maintenance and inspection will be lower.

As shown by the analysis in Chapter 5, increasing continuity would require more reinforcing steel for the columns. Also, the size of the spread foundations will increase, and with it, the size of the excavation. In addition, more reinforcing and prestressing steel would be needed for the continuous units. Even though the costs expansion joints and bearings decreases, the increase in continuity translates into an increase in construction costs.

Still, an increase in continuity would mean a reduction of operating costs. Increasing continuity in Tren Urbano would reduce the number of expansion joints and bearings in the guideway. As mentioned in the previous section this would reduce substantially the necessary inspection and maintenance of the structure, (including major rehabilitation due to the failure of expansion joints), and the costs associated with them.

Still, it was found in this research that the structure with the lowest life-cycle cost is the two-span-continuous design that Tren Urbano is using on most of its guideway. This result is tentative, since a lot of assumptions had to be made because reliable data was unavailable. Not included in the analysis, because they were unavailable, was the cost of inspection, which is expected to be more beneficial for an increase in continuity.

The effects of increasing the continuity of the Tren Urbano guideway in the total (or life-cycle) cost is something that needs to be determined more carefully using data on the economy of Puerto Rico. The life-cycle cost depends on such values as interest rates and inflation. These must be determined accurately in order to perform a correct analysis. It also depends on keeping accurate data on all the costs that can be expected to be incurred during the life of the bridge construction. Costs incurred after the bridge have been constructed also need to be predicted with accuracy.

7.5 Aesthetics

Increasing continuity would certainly improve the aesthetics of the guideway. First, the number of expansion joints would be reduced. As mentioned before, no expansion joint last forever. And when expansion joints fail, leakage starts to occur. With leakage, the passage of water borne chemicals, solids, and pollutants that stain and discolor the structure.

This affect negatively the elegance of the structure. With an increase in continuity, leakage is reduced, and therefore elegance is improved.

Another way aesthetics are improved as continuity is increased is that the number of expansion piers is reduced. The gap between the beam and the pier, where the bearings and expansion joints are located, make the structure seem awkward at this locations. All components of a bridge structure should be harmoniously integrated into one coherent, organic entity (Menn, 1990). The connection between the expansion pier and the beams does not fulfill this requirement by making the structure seem discontinuous.

Also, increasing continuity in Tren Urbano would reduce the number of bearings in the structure. Sometimes, birds use the gap provided by the bearings between the pier cap and the beam for nesting. Birds droppings would certainly be source of ugliness in a bridge structure. Increasing continuity would reduce the number of pier caps where birds could nest, improving the chance of keeping a cleaner structure.

Notice that none of the sources of problems in aesthetics mentioned above are completely eliminated with increasing continuity in the Tren Urbano Project, unless the superstructure is made completely continuous. To do this, it would be necessary to explore other design possibilities that were not tried in this research, such as changing the cross-section of the precast segments, or not attaching rigidly the columns to the superstructure.

7.6 Conclusions

It is important for bridge designers to always keep in mind the four objectives of bridge design. They should give each objective the necessary attention, and they should not commit the error that many designers have committed before of neglecting some objectives.

They should try to understand the complex relationship between the objectives. One way of understanding this relationship would be with the development of graphs as the one shown in Figure 7.1.

Figure 7.1 shows the effects that increasing bridge continuity in Tren Urbano would have on the achievement of the four objectives of bridge design. This graph can only be applied to other similar structures like the one analyzed here. Meaning, this particular graph only applies to a precast prestressed segmental concrete rail transit guideway, supported on spread footings, with similar temperature changes, seismological conditions, and skills for construction and maintenance as Puerto Rico. Changing any of these aspects could change the final graph.

In the graph, "safety 1" identifies the changes in safety as affected by continuity through redundancy, as explained in Section 7.2. Likewise, "safety 2" refers to the changes on safety due to the variations in stresses in the precast segmental units, also explained in Section 7.2. Due to the nature of the Tren Urbano Project, it was necessary to use these two lines to represent safety. In other graphs for different conditions that the ones of Figure 7.1, it is possible that safety could be replaced with just one line, or it may need additional lines.

Representing the economy of the bridge in Figure 7.1 are both, the "initial (or construction) costs" of the project, with the "operation costs". The graph would be more helpful if instead, the two lines were replaced with one line for "life-cycle cost". A line that could be done with accurate information of the economy of Puerto Rico. Notice that for this graph, a reduction in cost means a better rating.

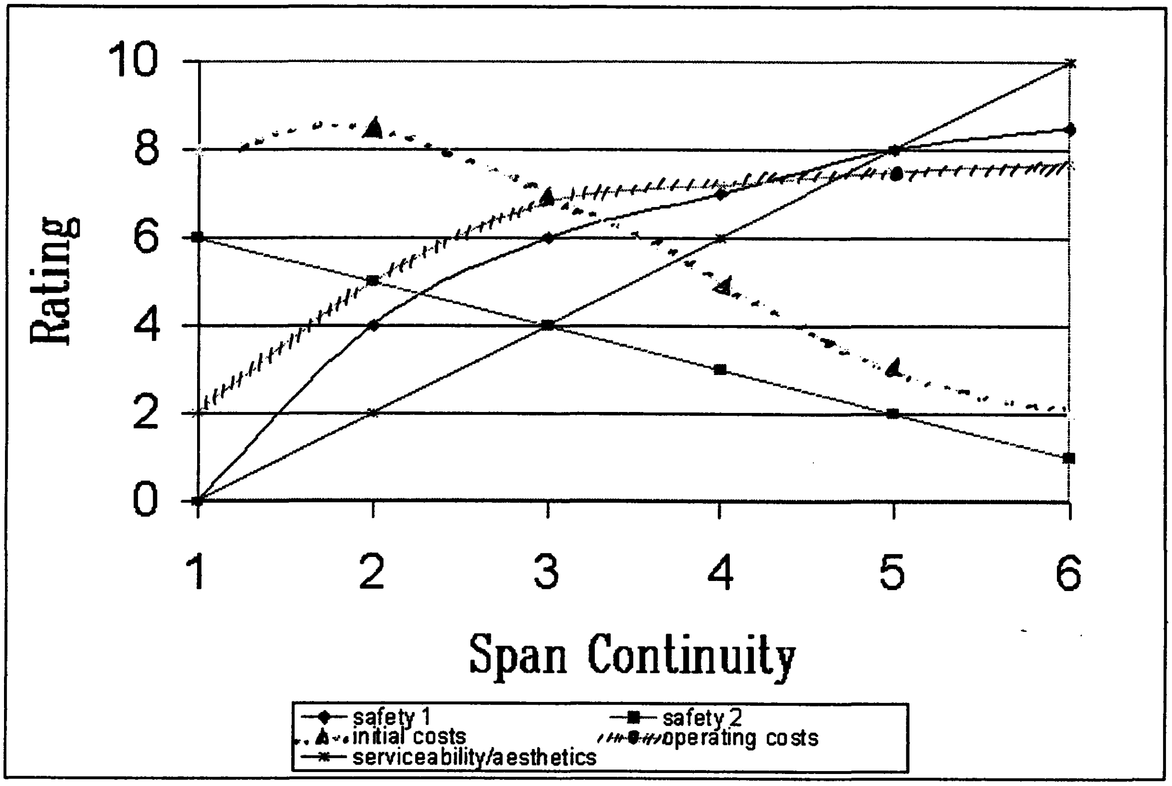

Figure 7.1: Interrelationship of the four objectives based on issues of continuity for a precast prestressed segmental rail transit bridge structure supported on spread footings

Finally, in the graph of Figure 7.1, the objectives of serviceability and aesthetics share one line. This occurs because both of these objectives are directly related to the number of expansion joints.

Figure 7.1 shows that when trying to improve an aspect of a bridge design, it is inevitable that other aspects will get affected. It is important to find a point in the graph where all lines of the objectives are maximized. There are necessary tradeoffs in the decisions that can be made as a result of the knowledge gained by this graph.

Similar graphs to the one shown in Figure 7.1 could be developed for different alternatives that were not analyzed in this research. For example a graph could show the effects of increasing continuity of the superstructure using bearings on top of the columns, as opposed to the continuous units being integral with the columns.

Another important thing shown by this research is that the life-cycle cost analysis can be a powerful tool for design. It does not make sense to spend money on products of bad quality. The life-cycle costing can help predict the quality of a bridge by predicting all the money that is going to be spent on it.

This research has shown that it would be wise for the Tren Urbano Project to start collecting data from the construction of the guideway sections that are already underway. Also, the performance under operation of these guideways should be observed. The collected data should be used to perform life-cycle costing for future phases of Tren Urbano.

Appendix A

Input Files for Dead Loads, Live Loads, and Earthquake Loads

A.1 Simple Span Design

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SIMPLE SPAN

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704 J=702,703   SEC=LINK
705 J=703,704   SEC=LINK
706 J=702,705   SEC=BEARING
707 J=704,707   SEC=BEARING
708 J=705,706   SEC=LINK
709 J=706,707   SEC=LINK

LOAD
NAME=DEAD
TYPE=GRAV ELEM=FRAME
ADD=* UZ=-1
TYPE=FORC
ADD=2004,6004,1000 UZ=-421.440
ADD=101,701,600   UZ=-421.440
TYPE=DIST
ADD=1,15,1      UZ=160
ADD=16,30,1     UZ=160
ADD=31,45,1     UZ=160
ADD=46,60,1     UZ=160
ADD=61,75,1     UZ=160
ADD=76,90,1     UZ=160
TYPE=FORC
ADD=1,81,16     UY=20496  UZ=-2092
ADD=16,96,16    UY=-20496 UZ=-2092

MODES
TYPE=EIGEN N=50

FUNCTION
NAME=ACCSPEC NPL=1
0   .5
0.1 .5
0.2 .5
0.3 .5
0.4 .5
0.5 .4572
```

```
0.6 .4048
0.7 .3653
0.8 .3342
0.9 .3090
1.0 .2880
1.1 .2703
1.2 .2550
1.3 .2418
1.4 .2301
1.5 .2198
1.6 .2105
1.7 .2022
1.8 .1946
1.9 .1877
2.0 .1814

SPEC
NAME=TRANS  ANG=0 DAMP=0.05
ACC=U1 FUNC=ACCSPEC SF=9.81
NAME=LONGIT ANG=0 DAMP=0.05
ACC=U2 FUNC=ACCSPEC SF=9.81

LANE
NAME=TRACK
PATH=102
PATH=1,15,1
PATH=202,211,9
PATH=16,30,1
PATH=302,311,9
PATH=31,45,1
PATH=402,411,9
PATH=46,60,1
PATH=502,511,9
PATH=61,75,1
PATH=602,611,9
PATH=76,90,1
PATH=702

VEHICLE
NAME=1PAIR   TYPE=GEN
P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
D=4.09  P=146
D=2.085 P=146
```

D=13.54 P=146
D=2.085 P=146
NAME=2PAIR TYPE=GEN
P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
D=4.09 P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
D=4.09 P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
D=4.09 P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
NAME=3PAIR TYPE=GEN
P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
D=4.09 P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
D=4.09 P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
D=4.09 P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
D=4.09 P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
D=4.09 P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146

VEHICLE CLASS

```
NAME=ALL
VEHI=1PAIR
VEHI=2PAIR
VEHI=3PAIR

BRIDGE RESPONSE
ELEM=JOINT  TYPE=DISP,REAC,SPRING
ADD=1,87,1
ADD=2003,6003,1000
ADD=2004,6004,1000
ELEM=FRAME
ADD=1,80,1
ADD=2003,6003,1000

MOVING LOAD
NAME=TRAIN  RF=1
CLASS=ALL   LANE=TRACK   SF=1.3

COMBO
NAME=ONE    TYPE=ADD
LOAD=DEAD
MOVE=TRAIN
NAME=TWO    TYPE=ADD
LOAD=DEAD
SPEC=TRANS    SF=1.25
MOVE=TRAIN
NAME=THREE  TYPE=ADD
LOAD=DEAD
SPEC=LONGIT   SF=1.25
MOVE=TRAIN

END
```

A.2 Two-Span-Continuous Design

```
TWO-SPAN-CONTINUOUS

SYSTEM
LENGTH=M  FORCE=KN

JOINT
2003 X=0   Y=36    Z=0
2004 X=0   Y=36    Z=6
3003 X=0   Y=72    Z=0
3004 X=0   Y=72    Z=5.9
4003 X=0   Y=108   Z=0
```

4004 X=0 Y=108 Z=6
5003 X=0 Y=144 Z=0
5004 X=0 Y=144 Z=5.9
6003 X=0 Y=180 Z=0
6004 X=0 Y=180 Z=6
101 X=0 Y=0 Z=5.9
102 X=1.1 Y=1.4 Z=5.9
103 X=0 Y=1.4 Z=5.9
104 X=-1.1 Y=1.4 Z=5.9
105 X=1.1 Y=1.4 Z=6
106 X=0 Y=1.4 Z=6
107 X=-1.1 Y=1.4 Z=6
301 X=0 Y=72 Z=5.9
302 X=1.1 Y=70.6 Z=5.9
303 X=0 Y=70.6 Z=5.9
304 X=-1.1 Y=70.6 Z=5.9
305 X=1.1 Y=70.6 Z=6
306 X=0 Y=70.6 Z=6
307 X=-1.1 Y=70.6 Z=6
308 X=0 Y=72 Z=5.9
309 X=1.1 Y=73.4 Z=5.9
310 X=0 Y=73.4 Z=5.9
311 X=-1.1 Y=73.4 Z=5.9
312 X=1.1 Y=73.4 Z=6
313 X=0 Y=73.4 Z=6
314 X=-1.1 Y=73.4 Z=6
501 X=0 Y=144 Z=5.9
502 X=1.1 Y=142.6 Z=5.9
503 X=0 Y=142.6 Z=5.9
504 X=-1.1 Y=142.6 Z=5.9
505 X=1.1 Y=142.6 Z=6
506 X=0 Y=142.6 Z=6
507 X=-1.1 Y=142.6 Z=6
508 X=0 Y=144 Z=5.9
509 X=1.1 Y=145.4 Z=5.9
510 X=0 Y=145.4 Z=5.9
511 X=-1.1 Y=145.4 Z=5.9
512 X=1.1 Y=145.4 Z=6
513 X=0 Y=145.4 Z=6
514 X=-1.1 Y=145.4 Z=6
701 X=0 Y=216 Z=5.9
702 X=1.1 Y=214.6 Z=5.9
703 X=0 Y=214.6 Z=5.9
704 X=-1.1 Y=214.6 Z=5.9
705 X=1.1 Y=214.6 Z=6
706 X=0 Y=214.6 Z=6

```
707  X=-1.1 Y=214.6  Z=6
1   X=0   Y=1.4   Z=6
2   X=0   Y=1.9   Z=6
3   X=0   Y=2.9   Z=6
14  X=0   Y=35    Z=6
LGEN=3,14,1
15  X=0   Y=36    Z=6
16  X=0   Y=37    Z=6
27  X=0   Y=69.1  Z=6
LGEN=16,27,1
28  X=0   Y=70.1  Z=6
29  X=0   Y=70.6  Z=6
30  X=0   Y=73.4  Z=6
31  X=0   Y=73.9  Z=6
32  X=0   Y=74.9  Z=6
43  X=0   Y=107   Z=6
LGEN=32,43,1
44  X=0   Y=108   Z=6
45  X=0   Y=109   Z=6
56  X=0   Y=141.1 Z=6
LGEN=45,56,1
57  X=0   Y=142.1 Z=6
58  X=0   Y=142.6 Z=6
59  X=0   Y=145.4 Z=6
60  X=0   Y=145.9 Z=6
61  X=0   Y=146.9 Z=6
72  X=0   Y=179   Z=6
LGEN=61,72,1
73  X=0   Y=180   Z=6
74  X=0   Y=181   Z=6
85  X=0   Y=213.1 Z=6
LGEN=74,85,1
86  X=0   Y=214.1 Z=6
87  X=0   Y=214.6 Z=6

RESTRAINT
ADD=2003,6003,1000 DOF=ALL
ADD=101,701,600    DOF=ALL

WELD
NAME=ALL  TOL=0.000001
ADD=*

MASS
ADD=3004,5004,2000  UX=42.96 UY=42.96 UZ=42.96
ADD=101,701,600     UX=42.96 UY=42.96 UZ=42.96
```

```
MATERIAL
NAME=BEAM   TYPE=ISO  M=2.4  W=23.544
E=27670000  U=0.30   A=0.000011
NAME=COLUMN  TYPE=ISO  M=2.4  W=23.544
E=23420000  U=0.25
NAME=TBEAM   TYPE=ISO
E=27670000  U=0.30
NAME=ELASTO  TYPE=ORTHO
E=1110242,1110242,1110242    G=1400,1400,14000

FRAME SECTION
NAME=ONE     TYPE=PRISM  MAT=BEAM   A=4.2374 J=1.054  I=.43567,7.1614
AS=2.862,3.2669 MPL=3.99 WPL=39.127
NAME=TWO     TYPE=PRISM  MAT=BEAM   A=6.6136 J=3.883  I=2.8134,9.1271
AS=2.900,3.480  MPL=3.99 WPL=39.127
NAME=THREE   TYPE=PRISM  MAT=BEAM   A=3.0664 J=2.976  I=1.8551,6.8861
AS=1.175,1.460  MPL=3.99 WPL=39.127
NAME=FOUR    TYPE=PRISM  MAT=BEAM   A=6.9040 J=4.076  I=2.9480,9.1760
AS=3.183,3.850  MPL=3.99 WPL=39.127
NAME=COLFIX  TYPE=PRISM  MAT=COLUMN SH=P    T=2.135
NAME=COLEXP  TYPE=PRISM  MAT=COLUMN SH=P    T=1.83
NAME=LINK    TYPE=PRISM  MAT=TBEAM  A=1000   J=1000   I=1000,1000
AS=1000,1000
NAME=BEARING  TYPE=PRISM  MAT=ELASTO SH=R    T=0.75,0.75

FRAME
1  J=1,2   SEC=ONE
2  J=2,3   SEC=TWO
3  J=3,4   SEC=THREE
GEN=3,13,1
14 J=14,15 SEC=FOUR
15 J=15,16 SEC=FOUR
16 J=16,17 SEC=THREE
GEN=16,26,1
27 J=27,28 SEC=TWO
28 J=28,29 SEC=ONE
29 J=30,31 SEC=ONE
30 J=31,32 SEC=TWO
31 J=32,33 SEC=THREE
GEN=31,41,1
42 J=43,44 SEC=FOUR
43 J=44,45 SEC=FOUR
44 J=45,46 SEC=THREE
GEN=44,54,1
55 J=56,57 SEC=TWO
56 J=57,58 SEC=ONE
```

```
57 J=59,60 SEC=ONE
58 J=60,61 SEC=TWO
59 J=61,62 SEC=THREE
GEN=59,69,1
70 J=72,73 SEC=FOUR
71 J=73,74 SEC=FOUR
72 J=74,75 SEC=THREE
GEN=72,82,1
83 J=85,86 SEC=TWO
84 J=86,87 SEC=ONE
2003 J=2003,2004  SEC=COLFIX
3003 J=3003,3004  SEC=COLEXP
4003 J=4003,4004  SEC=COLFIX
5003 J=5003,5004  SEC=COLEXP
6003 J=6003,6004  SEC=COLFIX
101  J=101,102    SEC=LINK
102  J=101,103    SEC=LINK
103  J=101,104    SEC=LINK
104  J=102,103    SEC=LINK
105  J=103,104    SEC=LINK
106  J=102,105    SEC=BEARING
107  J=104,107    SEC=BEARING
108  J=105,106    SEC=LINK
109  J=106,107    SEC=LINK
301  J=301,302    SEC=LINK
302  J=303,301    SEC=LINK
303  J=301,304    SEC=LINK
304  J=302,303    SEC=LINK
305  J=303,304    SEC=LINK
306  J=302,305    SEC=BEARING
307  J=304,307    SEC=BEARING
308  J=305,306    SEC=LINK
309  J=306,307    SEC=LINK
310  J=308,309    SEC=LINK
311  J=308,310    SEC=LINK
312  J=308,311    SEC=LINK
313  J=309,310    SEC=LINK
314  J=310,311    SEC=LINK
315  J=309,312    SEC=BEARING
316  J=311,314    SEC=BEARING
317  J=312,313    SEC=LINK
318  J=313,314    SEC=LINK
501  J=501,502    SEC=LINK
502  J=503,501    SEC=LINK
503  J=501,504    SEC=LINK
504  J=502,503    SEC=LINK
```

```
505 J=503,504   SEC=LINK
506 J=502,505   SEC=BEARING
507 J=504,507   SEC=BEARING
508 J=505,506   SEC=LINK
509 J=506,507   SEC=LINK
510 J=508,509   SEC=LINK
511 J=508,510   SEC=LINK
512 J=508,511   SEC=LINK
513 J=509,510   SEC=LINK
514 J=510,511   SEC=LINK
515 J=509,512   SEC=BEARING
516 J=511,514   SEC=BEARING
517 J=512,513   SEC=LINK
518 J=513,514   SEC=LINK
701 J=701,702   SEC=LINK
702 J=703,701   SEC=LINK
703 J=701,704   SEC=LINK
704 J=702,703   SEC=LINK
705 J=703,704   SEC=LINK
706 J=702,705   SEC=BEARING
707 J=704,707   SEC=BEARING
708 J=705,706   SEC=LINK
709 J=706,707   SEC=LINK

LOAD
NAME=DEAD
TYPE=GRAV ELEM=FRAME
ADD=* UZ=-1
TYPE=FORC
ADD=3004,5004,2000 UZ=-421.440
ADD=101,701,600   UZ=-421.440
TYPE=DIST
ADD=1,84,1        UZ=130
TYPE=FORC
ADD=1,15,14       UY=18087  UZ=-1846
ADD=30,44,14      UY=18087  UZ=-1846
ADD=59,73,14      UY=18087  UZ=-1846
ADD=15,29,14      UY=-18087 UZ=-1846
ADD=44,58,14      UY=-18087 UZ=-1846
ADD=73,87,14      UY=-18087 UZ=-1846

MODES
TYPE=EIGEN N=50

FUNCTION
NAME=ACCSPEC NPL=1
```

```
0   .5
0.1 .5
0.2 .5
0.3 .5
0.4 .5
0.5 .4572
0.6 .4048
0.7 .3653
0.8 .3342
0.9 .3090
1.0 .2880
1.1 .2703
1.2 .2550
1.3 .2418
1.4 .2301
1.5 .2198
1.6 .2105
1.7 .2022
1.8 .1946
1.9 .1877
2.0 .1814

SPEC
NAME=TRANS  ANG=0 DAMP=0.05
ACC=U1 FUNC=ACCSPEC SF=9.81
NAME=LONGIT ANG=0 DAMP=0.05
ACC=U2 FUNC=ACCSPEC SF=9.81

LANE
NAME=TRACK
PATH=102
PATH=1,28,1
PATH=302,311,9
PATH=29,56,1
PATH=502,511,9
PATH=57,84,1
PATH=702

VEHICLE
NAME=1PAIR   TYPE=GEN
P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
D=4.09  P=146
D=2.085 P=146
```

D=13.54 P=146
D=2.085 P=146
NAME=2PAIR TYPE=GEN
P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
D=4.09 P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
D=4.09 P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
D=4.09 P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
NAME=3PAIR TYPE=GEN
P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
D=4.09 P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
D=4.09 P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
D=4.09 P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
D=4.09 P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
D=4.09 P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146

VEHICLE CLASS

```
NAME=ALL
VEHI=1PAIR
VEHI=2PAIR
VEHI=3PAIR

BRIDGE RESPONSE
ELEM=JOINT  TYPE=DISP,REAC,SPRING
ADD=1,87,1
ADD=2003,6003,1000
ADD=2004,6004,1000
ELEM=FRAME
ADD=1,84,1
ADD=2003,6003,1000

MOVING LOAD
NAME=TRAIN  RF=1
CLASS=ALL   LANE=TRACK   SF=1.3

COMBO
NAME=ONE    TYPE=ADD
LOAD=DEAD
MOVE=TRAIN
NAME=TWO    TYPE=ADD
LOAD=DEAD
SPEC=TRANS    SF=1.25
MOVE=TRAIN
NAME=THREE  TYPE=ADD
LOAD=DEAD
SPEC=LONGIT   SF=1.25
MOVE=TRAIN

END
```

A.3 Three-Span-Continuous Design

```
THREE-SPAN-CONTINOUS

SYSTEM
LENGTH=M  FORCE=KN

JOINT
2003 X=0   Y=36    Z=0
2004 X=0   Y=36    Z=6
3003 X=0   Y=72    Z=0
3004 X=0   Y=72    Z=6
4003 X=0   Y=108   Z=0
```

4004 X=0 Y=108 Z=5.9
5003 X=0 Y=144 Z=0
5004 X=0 Y=144 Z=6
6003 X=0 Y=180 Z=0
6004 X=0 Y=180 Z=6
101 X=0 Y=0 Z=5.9
102 X=1.1 Y=1.4 Z=5.9
103 X=0 Y=1.4 Z=5.9
104 X=-1.1 Y=1.4 Z=5.9
105 X=1.1 Y=1.4 Z=6
106 X=0 Y=1.4 Z=6
107 X=-1.1 Y=1.4 Z=6
401 X=0 Y=108 Z=5.9
402 X=1.1 Y=106.6 Z=5.9
403 X=0 Y=106.6 Z=5.9
404 X=-1.1 Y=106.6 Z=5.9
405 X=1.1 Y=106.6 Z=6
406 X=0 Y=106.6 Z=6
407 X=-1.1 Y=106.6 Z=6
408 X=0 Y=108 Z=5.9
409 X=1.1 Y=109.4 Z=5.9
410 X=0 Y=109.4 Z=5.9
411 X=-1.1 Y=109.4 Z=5.9
412 X=1.1 Y=109.4 Z=6
413 X=0 Y=109.4 Z=6
414 X=-1.1 Y=109.4 Z=6
701 X=0 Y=216 Z=5.9
702 X=1.1 Y=214.6 Z=5.9
703 X=0 Y=214.6 Z=5.9
704 X=-1.1 Y=214.6 Z=5.9
705 X=1.1 Y=214.6 Z=6
706 X=0 Y=214.6 Z=6
707 X=-1.1 Y=214.6 Z=6
1 X=0 Y=1.4 Z=6
2 X=0 Y=1.9 Z=6
3 X=0 Y=2.9 Z=6
14 X=0 Y=35 Z=6
LGEN=3,14,1
15 X=0 Y=36 Z=6
16 X=0 Y=37 Z=6
27 X=0 Y=71 Z=6
LGEN=16,27,1
28 X=0 Y=72 Z=6
29 X=0 Y=73 Z=6
40 X=0 Y=105.1 Z=6
LGEN=29,40,1
41 X=0 Y=106.1 Z=6
42 X=0 Y=106.6 Z=6
43 X=0 Y=109.4 Z=6
44 X=0 Y=109.9 Z=6
45 X=0 Y=110.9 Z=6
56 X=0 Y=143 Z=6
LGEN=45,56,1
57 X=0 Y=144 Z=6

```
58  X=0   Y=145   Z=6
69  X=0   Y=179   Z=6
LGEN=58,69,1
70  X=0   Y=180   Z=6
71  X=0   Y=181   Z=6
82  X=0   Y=213.1 Z=6
LGEN=71,82,1
83  X=0   Y=214.1 Z=6
84  X=0   Y=214.6 Z=6

RESTRAINT
ADD=2003,6003,1000 DOF=ALL
ADD=101,701,600   DOF=ALL

WELD
NAME=ALL  TOL=0.000001
ADD=*

MASS
ADD=4004        UX=42.96 UY=42.96 UZ=42.96
ADD=101,701,600  UX=42.96 UY=42.96 UZ=42.96

MATERIAL
NAME=BEAM   TYPE=ISO  M=2.4  W=23.544
E=27670000  U=0.30   A=0.000011
NAME=COLUMN  TYPE=ISO  M=2.4  W=23.544
E=23420000  U=0.25
NAME=TBEAM  TYPE=ISO
E=27670000  U=0.30
NAME=ELASTO  TYPE=ORTHO
E=1284349,1284349,1284349    G=1400,1400,1400

FRAME SECTION
NAME=ONE     TYPE=PRISM  MAT=BEAM   A=4.2374 J=1.054  I=.43567,7.1614
AS=2.862,3.2669 MPL=3.99 WPL=39.127
NAME=TWO     TYPE=PRISM  MAT=BEAM   A=6.6136 J=3.883  I=2.8134,9.1271
AS=2.900,3.480  MPL=3.99 WPL=39.127
NAME=THREE   TYPE=PRISM  MAT=BEAM   A=3.0664 J=2.976  I=1.8551,6.8861
AS=1.175,1.460  MPL=3.99 WPL=39.127
NAME=FOUR    TYPE=PRISM  MAT=BEAM   A=6.9040 J=4.076  I=2.9480,9.1760
AS=3.183,3.850  MPL=3.99 WPL=39.127
NAME=COLFIX  TYPE=PRISM  MAT=COLUMN  SH=P    T=1.83
NAME=COLEXP  TYPE=PRISM  MAT=COLUMN SH=P    T=1.83
NAME=LINK    TYPE=PRISM  MAT=TBEAM  A=1000   J=1000   I=1000,1000
AS=1000,1000
NAME=BEARING  TYPE=PRISM  MAT=ELASTO SH=R    T=0.85,0.85

FRAME
1  J=1,2   SEC=ONE
2  J=2,3   SEC=TWO
3  J=3,4   SEC=THREE
GEN=3,13,1
14 J=14,15 SEC=FOUR
15 J=15,16 SEC=FOUR
```

```
16 J=16,17 SEC=THREE
GEN=16,26,1
27 J=27,28 SEC=FOUR
28 J=28,29 SEC=FOUR
29 J=29,30 SEC=THREE
GEN=29,39,1
40 J=40,41 SEC=TWO
41 J=41,42 SEC=ONE
42 J=43,44 SEC=ONE
43 J=44,45 SEC=TWO
44 J=45,46 SEC=THREE
GEN=44,54,1
55 J=56,57 SEC=FOUR
56 J=57,58 SEC=FOUR
57 J=58,59 SEC=THREE
GEN=57,67,1
68 J=69,70 SEC=FOUR
69 J=70,71 SEC=FOUR
70 J=71,72 SEC=THREE
GEN=70,80,1
81 J=82,83 SEC=TWO
82 J=83,84 SEC=ONE
2003 J=2003,2004  SEC=COLFIX
3003 J=3003,3004  SEC=COLFIX
4003 J=4003,4004  SEC=COLEXP
5003 J=5003,5004  SEC=COLFIX
6003 J=6003,6004  SEC=COLFIX
101  J=101,102   SEC=LINK
102  J=101,103   SEC=LINK
103  J=101,104   SEC=LINK
104  J=102,103   SEC=LINK
105  J=103,104   SEC=LINK
106  J=102,105   SEC=BEARING
107  J=104,107   SEC=BEARING
108  J=105,106   SEC=LINK
109  J=106,107   SEC=LINK
401  J=401,402   SEC=LINK
402  J=403,401   SEC=LINK
403  J=401,404   SEC=LINK
404  J=402,403   SEC=LINK
405  J=403,404   SEC=LINK
406  J=402,405   SEC=BEARING
407  J=404,407   SEC=BEARING
408  J=405,406   SEC=LINK
409  J=406,407   SEC=LINK
410  J=408,409   SEC=LINK
411  J=408,410   SEC=LINK
412  J=408,411   SEC=LINK
413  J=409,410   SEC=LINK
414  J=410,411   SEC=LINK
415  J=409,412   SEC=BEARING
416  J=411,414   SEC=BEARING
417  J=412,413   SEC=LINK
418  J=413,414   SEC=LINK
```

```
701 J=701,702  SEC=LINK
702 J=703,701  SEC=LINK
703 J=701,704  SEC=LINK
704 J=702,703  SEC=LINK
705 J=703,704  SEC=LINK
706 J=702,705  SEC=BEARING
707 J=704,707  SEC=BEARING
708 J=705,706  SEC=LINK
709 J=706,707  SEC=LINK

LOAD
NAME=DEAD
TYPE=GRAV ELEM=FRAME
ADD=* UZ=-1
TYPE=FORC
ADD=4004       UZ=-421.440
ADD=101,701,600 UZ=-421.440
TYPE=DIST
ADD=1,14,1     UZ=130
ADD=15,27,1    UZ=160
ADD=28,41,1    UZ=130
ADD=42,55,1    UZ=130
ADD=56,68,1    UZ=160
ADD=69,82,1    UZ=130
TYPE=FORC
ADD=1          UY=18087        UZ=-1846
ADD=15         UY=-18087+21495 UZ=-1846-2194
ADD=28         UY=18087-21495  UZ=-1846-2194
ADD=42         UY=-18087       UZ=-1846
ADD=43         UY=18087        UZ=-1846
ADD=57         UY=-18087+21495 UZ=-1846-2194
ADD=70         UY=18087-21495  UZ=-1846-2194
ADD=84         UY=-18087       UZ=-1846

MODES
TYPE=EIGEN N=50

FUNCTION
NAME=ACCSPEC NPL=1
0   .5
0.1 .5
0.2 .5
0.3 .5
0.4 .5
0.5 .4572
0.6 .4048
0.7 .3653
0.8 .3342
0.9 .3090
1.0 .2880
1.1 .2703
1.2 .2550
1.3 .2418
1.4 .2301
```

```
1.5 .2198
1.6 .2105
1.7 .2022
1.8 .1946
1.9 .1877
2.0 .1814

SPEC
NAME=TRANS  ANG=0 DAMP=0.05
ACC=U1 FUNC=ACCSPEC SF=9.81
NAME=LONGIT ANG=0 DAMP=0.05
ACC=U2 FUNC=ACCSPEC SF=9.81

LANE
NAME=TRACK
PATH=102
PATH=1,41,1
PATH=402,411,9
PATH=42,82,1
PATH=702

VEHICLE
NAME=1PAIR  TYPE=GEN
P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
D=4.09  P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
NAME=2PAIR  TYPE=GEN
P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
D=4.09  P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
D=4.09  P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
D=4.09  P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
NAME=3PAIR TYPE=GEN
P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
D=4.09  P=146
```

```
D=2.085  P=146
D=13.54  P=146
D=2.085  P=146
D=4.09   P=146
D=2.085  P=146
D=13.54  P=146
D=2.085  P=146
D=4.09   P=146
D=2.085  P=146
D=13.54  P=146
D=2.085  P=146
D=4.09   P=146
D=2.085  P=146
D=13.54  P=146
D=2.085  P=146
D=4.09   P=146
D=2.085  P=146
D=13.54  P=146
D=2.085  P=146

VEHICLE CLASS
NAME=ALL
VEHI=1PAIR
VEHI=2PAIR
VEHI=3PAIR

BRIDGE RESPONSE
ELEM=JOINT  TYPE=DISP,REAC,SPRING
ADD=1,84,1
ADD=2003,6003,1000
ADD=2004,6004,1000
ELEM=FRAME
ADD=1,82,1
ADD=2003,6003,1000

MOVING LOAD
NAME=TRAIN  RF=1
CLASS=ALL   LANE=TRACK   SF=1.3


COMBO
NAME=ONE    TYPE=ADD
LOAD=DEAD
MOVE=TRAIN
NAME=TWO    TYPE=ADD
LOAD=DEAD
SPEC=TRANS    SF=1.25
MOVE=TRAIN
NAME=THREE  TYPE=ADD
LOAD=DEAD
SPEC=LONGIT   SF=1.25
MOVE=TRAIN

END
```

A.4 Four-Span-Continuous Design

```
FOUR-SPAN-CONTINUOUS

SYSTEM
LENGTH=M  FORCE=KN

JOINT
2003 X=0   Y=36    Z=0
2004 X=0   Y=36    Z=6
3003 X=0   Y=72    Z=0
3004 X=0   Y=72    Z=6
4003 X=0   Y=108   Z=0
4004 X=0   Y=108   Z=6
5003 X=0   Y=144   Z=0
5004 X=0   Y=144   Z=5.9
6003 X=0   Y=180   Z=0
6004 X=0   Y=180   Z=6
101  X=0   Y=0     Z=5.9
102  X=1.1 Y=1.4   Z=5.9
103  X=0   Y=1.4   Z=5.9
104  X=-1.1 Y=1.4  Z=5.9
105  X=1.1 Y=1.4   Z=6
106  X=0   Y=1.4   Z=6
107  X=-1.1 Y=1.4  Z=6
501  X=0   Y=144   Z=5.9
502  X=1.1 Y=142.6 Z=5.9
503  X=0   Y=142.6 Z=5.9
504  X=-1.1 Y=142.6 Z=5.9
505  X=1.1 Y=142.6 Z=6
506  X=0   Y=142.6 Z=6
507  X=-1.1 Y=142.6 Z=6
508  X=0   Y=144   Z=5.9
509  X=1.1 Y=145.4 Z=5.9
510  X=0   Y=145.4 Z=5.9
511  X=-1.1 Y=145.4 Z=5.9
512  X=1.1 Y=145.4 Z=6
513  X=0   Y=145.4 Z=6
514  X=-1.1 Y=145.4 Z=6
701  X=0   Y=216   Z=5.9
702  X=1.1 Y=214.6 Z=5.9
703  X=0   Y=214.6 Z=5.9
704  X=-1.1 Y=214.6 Z=5.9
705  X=1.1 Y=214.6 Z=6
706  X=0   Y=214.6 Z=6
707  X=-1.1 Y=214.6 Z=6
1   X=0   Y=1.4   Z=6
2   X=0   Y=1.9   Z=6
3   X=0   Y=2.9   Z=6
14  X=0   Y=35    Z=6
LGEN=3,14,1
15  X=0   Y=36    Z=6
16  X=0   Y=37    Z=6
27  X=0   Y=71    Z=6
```

```
LGEN=16,27,1
28  X=0   Y=72    Z=6
29  X=0   Y=73    Z=6
40  X=0   Y=107   Z=6
LGEN=29,40,1
41  X=0   Y=108   Z=6
42  X=0   Y=109   Z=6
53  X=0   Y=141.1 Z=6
LGEN=42,53,1
54  X=0   Y=142.1 Z=6
55  X=0   Y=142.6 Z=6
56  X=0   Y=145.4 Z=6
57  X=0   Y=145.9 Z=6
58  X=0   Y=146.9 Z=6
69  X=0   Y=179   Z=6
LGEN=58,69,1
70  X=0   Y=180   Z=6
71  X=0   Y=181   Z=6
82  X=0   Y=213.1 Z=6
LGEN=71,82,1
83  X=0   Y=214.1 Z=6
84  X=0   Y=214.6 Z=6

RESTRAINT
ADD=2003,6003,1000 DOF=ALL
ADD=101,701,600   DOF=ALL

WELD
NAME=ALL  TOL=0.000001
ADD=*

MASS
ADD=101,701,600    UX=42.96 UY=42.96 UZ=42.96
ADD=5004           UX=42.96 UY=42.96 UZ=42.96

MATERIAL
NAME=BEAM   TYPE=ISO  M=2.4  W=23.544
E=34970000 U=0.30    A=0.000011
NAME=COLUMN  TYPE=ISO  M=2.4  W=23.544
E=27670000 U=0.25
NAME=TBEAM   TYPE=ISO
E=34970000  U=0.30
NAME=ELASTO  TYPE=ORTHO
E=1496953,1496953,1496953    G=1400,1400,1400
NAME=ELASTO2  TYPE=ORTHO
E=898736,898736,898736        G=1400,1400,1400

FRAME SECTION
NAME=ONE     TYPE=PRISM  MAT=BEAM    A=4.2374 J=1.054  I=.43567,7.1614
AS=2.862,3.2669 MPL=3.99 WPL=39.127
NAME=TWO     TYPE=PRISM  MAT=BEAM    A=6.6136 J=3.883  I=2.8134,9.1271
AS=2.900,3.480  MPL=3.99 WPL=39.127
NAME=THREE   TYPE=PRISM  MAT=BEAM    A=3.0664 J=2.976  I=1.8551,6.8861
AS=1.175,1.460  MPL=3.99 WPL=39.127
```

```
NAME=FOUR    TYPE=PRISM  MAT=BEAM    A=6.9040 J=4.076  I=2.9480,9.1760
AS=3.183,3.850  MPL=3.99 WPL=39.127
NAME=COLFIX   TYPE=PRISM  MAT=COLUMN  SH=P    T=1.83
NAME=COLEXP   TYPE=PRISM  MAT=COLUMN  SH=P    T=1.83
NAME=LINK    TYPE=PRISM  MAT=TBEAM   A=1000  J=1000  I=1000,1000
AS=1000,1000
NAME=BEARING  TYPE=PRISM  MAT=ELASTO  SH=R    T=1.05,1.05
NAME=BEARING2 TYPE=PRISM  MAT=ELASTO2 SH=R    T=0.55,0.60

FRAME
1 J=1,2  SEC=ONE
2 J=2,3  SEC=TWO
3 J=3,4  SEC=THREE
GEN=3,13,1
14 J=14,15 SEC=FOUR
15 J=15,16 SEC=FOUR
16 J=16,17 SEC=THREE
GEN=16,26,1
27 J=27,28 SEC=FOUR
28 J=28,29 SEC=FOUR
29 J=29,30 SEC=THREE
GEN=29,39,1
40 J=40,41 SEC=FOUR
41 J=41,42 SEC=FOUR
42 J=42,43 SEC=THREE
GEN=42,52,1
53 J=53,54 SEC=TWO
54 J=54,55 SEC=ONE
55 J=56,57 SEC=ONE
56 J=57,58 SEC=TWO
57 J=58,59 SEC=THREE
GEN=57,67,1
68 J=69,70 SEC=FOUR
69 J=70,71 SEC=FOUR
70 J=71,72 SEC=THREE
GEN=70,80,1
81 J=82,83 SEC=TWO
82 J=83,84 SEC=ONE
2003 J=2003,2004  SEC=COLFIX
3003 J=3003,3004  SEC=COLFIX
4003 J=4003,4004  SEC=COLFIX
5003 J=5003,5004  SEC=COLEXP
6003 J=6003,6004  SEC=COLFIX
101  J=101,102   SEC=LINK
102  J=101,103   SEC=LINK
103  J=101,104   SEC=LINK
104  J=102,103   SEC=LINK
105  J=103,104   SEC=LINK
106  J=102,105   SEC=BEARING
107  J=104,107   SEC=BEARING
108  J=105,106   SEC=LINK
109  J=106,107   SEC=LINK
501  J=501,502   SEC=LINK
502  J=503,501   SEC=LINK
```

```
503 J=501,504   SEC=LINK
504 J=502,503   SEC=LINK
505 J=503,504   SEC=LINK
506 J=502,505   SEC=BEARING
507 J=504,507   SEC=BEARING
508 J=505,506   SEC=LINK
509 J=506,507   SEC=LINK
510 J=508,509   SEC=LINK
511 J=508,510   SEC=LINK
512 J=508,511   SEC=LINK
513 J=509,510   SEC=LINK
514 J=510,511   SEC=LINK
515 J=509,512   SEC=BEARING2
516 J=511,514   SEC=BEARING2
517 J=512,513   SEC=LINK
518 J=513,514   SEC=LINK
701 J=701,702   SEC=LINK
702 J=703,701   SEC=LINK
703 J=701,704   SEC=LINK
704 J=702,703   SEC=LINK
705 J=703,704   SEC=LINK
706 J=702,705   SEC=BEARING2
707 J=704,707   SEC=BEARING2
708 J=705,706   SEC=LINK
709 J=706,707   SEC=LINK

LOAD
NAME=DEAD
TYPE=GRAV  ELEM=FRAME
ADD=* UZ=-1
TYPE=FORC
ADD=101,701,600   UZ=-421.440
ADD=5004          UZ=-421.440
TYPE=DIST
ADD=1,14,1        UZ=130
ADD=15,27,1       UZ=160
ADD=28,40,1       UZ=160
ADD=41,54,1       UZ=130
ADD=55,68,1       UZ=130
ADD=69,82,1       UZ=130
TYPE=FORC
ADD=1   UY=18087        UZ=-1845
ADD=15  UY=-18087+21495 UZ=-1846-2194
ADD=28  UY=-21495+21495 UZ=-2194-2194
ADD=41  UY=-21495+18087 UZ=-2194-1846
ADD=55  UY=-18087       UZ=-1846
ADD=56  UY=18087        UZ=-1846
ADD=70  UY=-18087+18087 UZ=-1846-1846
ADD=84  UY=-18087       UZ=-1846

MODES
TYPE=EIGEN  N=50

FUNCTION
```

```
NAME=ACCSPEC NPL=1
0   .5
0.1 .5
0.2 .5
0.3 .5
0.4 .5
0.5 .4572
0.6 .4048
0.7 .3653
0.8 .3342
0.9 .3090
1.0 .2880
1.1 .2703
1.2 .2550
1.3 .2418
1.4 .2301
1.5 .2198
1.6 .2105
1.7 .2022
1.8 .1946
1.9 .1877
2.0 .1814

SPEC
NAME=TRANS  ANG=0 DAMP=0.05
ACC=U1 FUNC=ACCSPEC SF=9.81
NAME=LONGIT ANG=0 DAMP=0.05
ACC=U2 FUNC=ACCSPEC SF=9.81

LANE
NAME=TRACK
PATH=102
PATH=1,54,1
PATH=502,511,9
PATH=55,82,1
PATH=702

VEHICLE
NAME=1PAIR   TYPE=GEN
P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
D=4.09  P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
NAME=2PAIR   TYPE=GEN
P=146
D=2.085 P=146
D=13.54 P=146
D=2.085 P=146
D=4.09  P=146
D=2.085 P=146
```

```
D=13.54  P=146
D=2.085  P=146
D=4.09   P=146
D=2.085  P=146
D=13.54  P=146
D=2.085  P=146
D=4.09   P=146
D=2.085  P=146
D=13.54  P=146
D=2.085  P=146
NAME=3PAIR  TYPE=GEN
P=146
D=2.085  P=146
D=13.54  P=146
D=2.085  P=146
D=4.09   P=146
D=2.085  P=146
D=13.54  P=146
D=2.085  P=146
D=4.09   P=146
D=2.085  P=146
D=13.54  P=146
D=2.085  P=146
D=4.09   P=146
D=2.085  P=146
D=13.54  P=146
D=2.085  P=146
D=4.09   P=146
D=2.085  P=146
D=13.54  P=146
D=2.085  P=146
D=4.09   P=146
D=2.085  P=146
D=13.54  P=146
D=2.085  P=146

VEHICLE CLASS
NAME=ALL
VEHI=1PAIR
VEHI=2PAIR
VEHI=3PAIR

BRIDGE RESPONSE
ELEM=JOINT  TYPE=DISP,REAC,SPRING
ADD=1,84,1
ADD=2003,6003,1000
ADD=2004,6004,1000
ELEM=FRAME
ADD=1,82,1
ADD=2003,6003,1000

MOVING LOAD
NAME=TRAIN  RF=1
CLASS=ALL   LANE=TRACK   SF=1.3
```

```
COMBO
NAME=ONE   TYPE=ADD
LOAD=DEAD
MOVE=TRAIN
NAME=TWO   TYPE=ADD
LOAD=DEAD
SPEC=TRANS   SF=1.25
MOVE=TRAIN
NAME=THREE  TYPE=ADD
LOAD=DEAD
SPEC=LONGIT  SF=1.25
MOVE=TRAIN

END
```

Appendix B

Input Files for Creep, Shrinkage, and Temperature Effects

B.1 Simple Span Design

```
SIMPLE SPAN

SYSTEM
LENGTH=M  FORCE=KN

JOINT
1  X=0 Y=1.4  Z=6
11 X=0 Y=1.9  Z=6
LGEN=1,11,1
21 X=0 Y=2.9  Z=6
LGEN=11,21,1
131 X=0 Y=33.1 Z=6
LGEN=21,131,1
141 X=0 Y=34.1 Z=6
LGEN=131,141,1
151 X=0 Y=34.6 Z=6
LGEN=141,151,1
152 X=0 Y=37.4 Z=6
162 X=0 Y=37.9 Z=6
LGEN=152,162,1
172 X=0 Y=39.9 Z=6
LGEN=162,172,1
282 X=0 Y=69.1 Z=6
LGEN=172,282,1
292 X=0 Y=70.1 Z=6
LGEN=282,292,1
302 X=0 Y=70.6 Z=6
LGEN=292,302,1
303 X=0 Y=73.4 Z=6
313 X=0 Y=73.9 Z=6
LGEN=303,313,1
323 X=0 Y=74.9 Z=6
LGEN=313,323,1
433 X=0 Y=105.1 Z=6
LGEN=323,433,1
443 X=0 Y=106.1 Z=6
LGEN=433,443,1
```

```
453 X=0  Y=106.6 Z=6
LGEN=443,453,1
454 X=0  Y=109.4 Z=6
464 X=0  Y=109.9 Z=6
LGEN=454,464,1
474 X=0  Y=110.9 Z=6
LGEN=464,474,1
584 X=0  Y=141.1 Z=6
LGEN=474,584,1
594 X=0  Y=142.1 Z=6
LGEN=584,594,1
604 X=0  Y=142.6 Z=6
LGEN=594,604,1
605 X=0  Y=145.4 Z=6
615 X=0  Y=145.9 Z=6
LGEN=605,615,1
625 X=0  Y=146.9 Z=6
LGEN=615,625,1
735 X=0  Y=177.1 Z=6
LGEN=625,735,1
745 X=0  Y=178.1 Z=6
LGEN=735,745,1
755 X=0  Y=178.6 Z=6
LGEN=745,755,1
756 X=0  Y=181.4 Z=6
766 X=0  Y=181.9 Z=6
LGEN=756,766,1
776 X=0  Y=182.9 Z=6
LGEN=766,776,1
886 X=0  Y=213.1 Z=6
LGEN=776,886,1
896 X=0  Y=214.1 Z=6
LGEN=886,896,1
906 X=0  Y=214.6 Z=6
LGEN=896,906,1
;
2001  X=0 Y=1.4   Z=6.4
2011  X=0 Y=1.9   Z=6.4
LGEN=2001,2011,1
2021  X=0  Y=2.9  Z=6.4
LGEN=2011,2021,1
2131 X=0  Y=33.1  Z=6.4
LGEN=2021,2131,1
2141 X=0  Y=34.1  Z=6.4
LGEN=2131,2141,1
2151 X=0  Y=34.6  Z=6.4
```

```
LGEN=2141,2151,1
2152 X=0  Y=37.4  Z=6.4
2162 X=0  Y=37.9  Z=6.4
LGEN=2152,2162,1
2172 X=0  Y=39.9  Z=6.4
LGEN=2162,2172,1
2282 X=0  Y=69.1  Z=6.4
LGEN=2172,2282,1
2292 X=0  Y=70.1  Z=6.4
LGEN=2282,2292,1
2302 X=0  Y=70.6  Z=6.4
LGEN=2292,2302,1
2303 X=0  Y=73.4  Z=6.4
2313 X=0  Y=73.9  Z=6.4
LGEN=2303,2313,1
2323 X=0  Y=74.9  Z=6.4
LGEN=2313,2323,1
2433 X=0  Y=105.1 Z=6.4
LGEN=2323,2433,1
2443 X=0  Y=106.1 Z=6.4
LGEN=2433,2443,1
2453 X=0  Y=106.6 Z=6.4
LGEN=2443,2453,1
2454 X=0  Y=109.4 Z=6.4
2464 X=0  Y=109.9 Z=6.4
LGEN=2454,2464,1
2474 X=0  Y=110.9 Z=6.4
LGEN=2464,2474,1
2584 X=0  Y=141.1 Z=6.4
LGEN=2474,2584,1
2594 X=0  Y=142.1 Z=6.4
LGEN=2584,2594,1
2604 X=0  Y=142.6 Z=6.4
LGEN=2594,2604,1
2605 X=0  Y=145.4 Z=6.4
2615 X=0  Y=145.9 Z=6.4
LGEN=2605,2615,1
2625 X=0  Y=146.9 Z=6.4
LGEN=2615,2625,1
2735 X=0  Y=177.1 Z=6.4
LGEN=2625,2735,1
2745 X=0  Y=178.1 Z=6.4
LGEN=2735,2745,1
2755 X=0  Y=178.6 Z=6.4
LGEN=2745,2755,1
2756 X=0  Y=181.4 Z=6.4
```

```
2766 X=0  Y=181.9 Z=6.4
LGEN=2756,2766,1
2776 X=0  Y=182.9 Z=6.4
LGEN=2766,2776,1
2886 X=0  Y=213.1 Z=6.4
LGEN=2776,2886,1
2896 X=0  Y=214.1 Z=6.4
LGEN=2886,2896,1
2906 X=0  Y=214.6 Z=6.4
LGEN=2896,2906,1
;
10003  X=0  Y=0    Z=5.9
10013  X=0  Y=1.4  Z=5.9
LGEN=10003,10013,1
10014  X=0  Y=1.4  Z=6
20002  X=0  Y=36   Z=0
20003  X=0  Y=36   Z=5.9
20004  X=0  Y=34.6  Z=5.9
20024  X=0  Y=37.4  Z=5.9
LGEN=20004,20024,1
20025  X=0  Y=34.6  Z=6
20026  X=0  Y=37.4  Z=6
30002  X=0  Y=72   Z=0
30003  X=0  Y=72   Z=5.9
30004  X=0  Y=70.6  Z=5.9
30024  X=0  Y=73.4  Z=5.9
LGEN=30004,30024,1
30025  X=0  Y=70.6  Z=6
30026  X=0  Y=73.4  Z=6
40002  X=0  Y=108   Z=0
40003  X=0  Y=108   Z=5.9
40004  X=0  Y=106.6 Z=5.9
40024  X=0  Y=109.4 Z=5.9
LGEN=40004,40024,1
40025  X=0  Y=106.6 Z=6
40026  X=0  Y=109.4 Z=6
50002  X=0  Y=144   Z=0
50003  X=0  Y=144   Z=5.9
50004  X=0  Y=142.6 Z=5.9
50024  X=0  Y=145.4 Z=5.9
LGEN=50004,50024,1
50025  X=0  Y=142.6 Z=6
50026  X=0  Y=145.4 Z=6
60002  X=0  Y=180   Z=0
60003  X=0  Y=180   Z=5.9
60004  X=0  Y=178.6 Z=5.9
```

```
60024  X=0  Y=181.4 Z=5.9
LGEN=60004,60024,1
60025  X=0  Y=178.6 Z=6
60026  X=0  Y=181.4 Z=6
70003  X=0  Y=216   Z=5.9
70013  X=0  Y=214.6 Z=5.9
LGEN=70003,70013,1
70014  X=0  Y=214.6 Z=6
;
12003  X=0  Y=0     Z=6.4
12013  X=0  Y=1.4   Z=6.4
LGEN=12003,12013,1
22004  X=0  Y=34.6  Z=6.4
22024  X=0  Y=37.4  Z=6.4
LGEN=22004,22024,1
32004  X=0  Y=70.6  Z=6.4
32024  X=0  Y=73.4  Z=6.4
LGEN=32004,32024,1
42004  X=0  Y=106.6 Z=6.4
42024  X=0  Y=109.4 Z=6.4
LGEN=42004,42024,1
52004  X=0  Y=142.6 Z=6.4
52024  X=0  Y=145.4 Z=6.4
LGEN=52004,52024,1
62004  X=0  Y=178.6 Z=6.4
62024  X=0  Y=181.4 Z=6.4
LGEN=62004,62024,1
72003  X=0  Y=216   Z=6.4
72013  X=0  Y=214.6 Z=6.4
LGEN=72003,72013,1

RESTRAINT
ADD=20002,60002,10000  DOF=ALL
ADD=12003,72003,60000  DOF=ALL
ADD=10003,70003,60000  DOF=ALL

WELD
NAME=ALL  TOL=0.000001
ADD=*

MATERIAL
NAME=BEAM    TYPE=ISO  M=2.4  W=23.544
E=27670000  U=0.30   A=0.000011
NAME=COLUMN  TYPE=ISO  M=2.4  W=23.544
E=23420000  U=0.25
NAME=TBEAM   TYPE=ISO
```

```
E=27670000 U=0.30 A=0.000011
NAME=ELASTO TYPE=ORTHO
E=1619956,1619956,1619956 G=1400,1400,1400
NAME=RAIL TYPE=ISO
E=2.1E8 U=0.30 A=12E-6
NAME=RSPRING TYPE=ISO
E=8000

FRAME SECTION
NAME=ONE TYPE=PRISM MAT=BEAM A=4.2374 J=1.054 I=.43567,7.1614
AS=2.862,3.2669
NAME=TWO TYPE=PRISM MAT=BEAM A=6.6136 J=3.883 I=2.8134,9.1271
AS=2.900,3.480
NAME=THREE TYPE=PRISM MAT=BEAM A=3.0664 J=2.976 I=1.8551,6.8861
AS=1.175,1.460
NAME=FOUR TYPE=PRISM MAT=BEAM A=6.9040 J=4.076 I=2.9480,9.1760
AS=3.183,3.850
NAME=COLFIX TYPE=PRISM MAT=COLUMN SH=P T=2.135
NAME=COLEXP TYPE=PRISM MAT=COLUMN SH=P T=1.83
NAME=LINK TYPE=PRISM MAT=TBEAM A=1000 J=1000 I=1000,1000
AS=1000,1000
NAME=BEARING TYPE=PRISM MAT=ELASTO SH=R T=2*1.200,1.200
NAME=S1 TYPE=PRISM MAT=RSPRING A=1000 I=0.5/10/187.5
J=1000
NAME=S2 TYPE=PRISM MAT=RSPRING A=1000 I=1/10/187.5
J=1000
NAME=S3 TYPE=PRISM MAT=RSPRING A=1000 I=32.1/110/187.5
J=1000
NAME=S4 TYPE=PRISM MAT=RSPRING A=1000 I=1/10/187.5
J=1000
NAME=S5 TYPE=PRISM MAT=RSPRING A=1000 I=1.4/10/96
J=1000
NAME=RAIL TYPE=PRISM MAT=RAIL A=2*7257.34/1000/1000 I=2*27304781.52/
1000/1000/1000/1000

FRAME
1 J=1,2 SEC=ONE
GEN=1,10,1
11 J=11,12 SEC=TWO
GEN=11,20,1
21 J=21,22 SEC=THREE
GEN=21,130,1
131 J=131,132 SEC=TWO
GEN=131,140,1
141 J=141,142 SEC=ONE
GEN=141,150,1
151 J=152,153 SEC=ONE
GEN=151,160,1
```

161 J=162,163 SEC=TWO
GEN=161,170,1
171 J=172,173 SEC=THREE
GEN=171,280,1
281 J=282,283 SEC=TWO
GEN=281,290,1
291 J=292,293 SEC=ONE
GEN=291,300,1
301 J=303,304 SEC=ONE
GEN=301,310,1
311 J=313,314 SEC=TWO
GEN=311,320,1
321 J=323,324 SEC=THREE
GEN=321,430,1
431 J=433,434 SEC=TWO
GEN=431,440,1
441 J=443,444 SEC=ONE
GEN=441,450,1
451 J=454,455 SEC=ONE
GEN=451,460,1
461 J=464,465 SEC=TWO
GEN=461,470,1
471 J=474,475 SEC=THREE
GEN=471,580,1
581 J=584,585 SEC=TWO
GEN=581,590,1
591 J=594,595 SEC=ONE
GEN=591,600,1
601 J=605,606 SEC=ONE
GEN=601,610,1
611 J=615,616 SEC=TWO
GEN=611,620,1
621 J=625,626 SEC=THREE
GEN=621,730,1
731 J=735,736 SEC=TWO
GEN=731,740,1
741 J=745,746 SEC=ONE
GEN=741,750,1
751 J=756,757 SEC=ONE
GEN=751,760,1
761 J=766,767 SEC=TWO
GEN=761,770,1
771 J=776,777 SEC=THREE
GEN=771,880,1
881 J=886,887 SEC=TWO
GEN=881,890,1

```
891 J=896,897 SEC=ONE
GEN=891,900,1
;
2001 J=2001,2002 SEC=RAIL
GEN=2001,2010,1
2011 J=2011,2012 SEC=RAIL
GEN=2011,2020,1
2021 J=2021,2022 SEC=RAIL
GEN=2021,2130,1
2131 J=2131,2132 SEC=RAIL
GEN=2131,2140,1
2141 J=2141,2142 SEC=RAIL
GEN=2141,2150,1
2151 J=2152,2153 SEC=RAIL
GEN=2151,2160,1
2161 J=2162,2163 SEC=RAIL
GEN=2161,2170,1
2171 J=2172,2173 SEC=RAIL
GEN=2171,2280,1
2281 J=2282,2283 SEC=RAIL
GEN=2281,2290,1
2291 J=2292,2293 SEC=RAIL
GEN=2291,2300,1
2301 J=2303,2304 SEC=RAIL
GEN=2301,2310,1
2311 J=2313,2314 SEC=RAIL
GEN=2311,2320,1
2321 J=2323,2324 SEC=RAIL
GEN=2321,2430,1
2431 J=2433,2434 SEC=RAIL
GEN=2431,2440,1
2441 J=2443,2444 SEC=RAIL
GEN=2441,2450,1
2451 J=2454,2455 SEC=RAIL
GEN=2451,2460,1
2461 J=2464,2465 SEC=RAIL
GEN=2461,2470,1
2471 J=2474,2475 SEC=RAIL
GEN=2471,2580,1
2581 J=2584,2585 SEC=RAIL
GEN=2581,2590,1
2591 J=2594,2595 SEC=RAIL
GEN=2591,2600,1
2601 J=2605,2606 SEC=RAIL
GEN=2601,2610,1
2611 J=2615,2616 SEC=RAIL
```

```
GEN=2611,2620,1
2621 J=2625,2626 SEC=RAIL
GEN=2621,2730,1
2731 J=2735,2736 SEC=RAIL
GEN=2731,2740,1
2741 J=2745,2746 SEC=RAIL
GEN=2741,2750,1
2751 J=2756,2757 SEC=RAIL
GEN=2751,2760,1
2761 J=2766,2767 SEC=RAIL
GEN=2761,2770,1
2771 J=2776,2777 SEC=RAIL
GEN=2771,2880,1
2881 J=2886,2887 SEC=RAIL
GEN=2881,2890,1
2891 J=2896,2897 SEC=RAIL
GEN=2891,2900,1
;
12003 J=12003,12004  SEC=RAIL
GEN=12003,12012,1
22004 J=22004,22005  SEC=RAIL
GEN=22004,22023,1
32004 J=32004,32005  SEC=RAIL
GEN=32004,32023,1
42004 J=42004,42005  SEC=RAIL
GEN=42004,42023,1
52004 J=52004,52005  SEC=RAIL
GEN=52004,52023,1
62004 J=62004,62005  SEC=RAIL
GEN=62004,62023,1
72003 J=72003,72004  SEC=RAIL
GEN=72003,72012,1
;
10003 J=10003,10004  SEC=LINK
GEN=10003,10012,1
10013 J=10013,10014  SEC=BEARING
20002 J=20002,20003  SEC=COLEXP
20003 J=20004,20005  SEC=LINK
GEN=20003,20022,1
20023 J=20004,20025  SEC=BEARING
20024 J=20024,20026  SEC=BEARING
30002 J=30002,30003  SEC=COLEXP
30003 J=30004,30005  SEC=LINK
GEN=30003,30022,1
30023 J=30004,30025  SEC=BEARING
30024 J=30024,30026  SEC=BEARING
```

```
40002 J=40002,40003  SEC=COLEXP
40003 J=40004,40005  SEC=LINK
GEN=40003,40022,1
40023 J=40004,40025  SEC=BEARING
40024 J=40024,40026  SEC=BEARING
50002 J=50002,50003  SEC=COLEXP
50003 J=50004,50005  SEC=LINK
GEN=50003,50022,1
50023 J=50004,50025  SEC=BEARING
50024 J=50024,50026  SEC=BEARING
60002 J=60002,60003  SEC=COLEXP
60003 J=60004,60005  SEC=LINK
GEN=60003,60022,1
60023 J=60004,60025  SEC=BEARING
60024 J=60024,60026  SEC=BEARING
70003 J=70003,70004  SEC=LINK
GEN=70003,70012,1
70013 J=70013,70014  SEC=BEARING
;
1001  J=1,2001      SEC=S1
GEN=1001,1011,1
1012  J=12,2012     SEC=S2
GEN=1012,1021,1
1022  J=22,2022     SEC=S3
GEN=1022,1131,1
1132  J=132,2132    SEC=S2
GEN=1132,1141,1
1142  J=142,2142    SEC=S1
GEN=1142,1151,1
1152  J=152,2152    SEC=S1
GEN=1152,1162,1
1163  J=163,2163    SEC=S2
GEN=1163,1172,1
1173  J=173,2173    SEC=S3
GEN=1173,1302,1
1283  J=283,2283    SEC=S2
GEN=1283,1292,1
1293  J=293,2293    SEC=S1
GEN=1293,1302,1
1303  J=303,2303    SEC=S1
GEN=1303,1313,1
1314  J=314,2314    SEC=S2
GEN=1314,1323,1
1324  J=324,2324    SEC=S3
GEN=1324,1433,1
1434  J=434,2434    SEC=S2
```

```
GEN=1434,1443,1
1444  J=444,2444     SEC=S1
GEN=1444,1453,1
1454  J=454,2454     SEC=S1
GEN=1454,1464,1
1465  J=465,2465     SEC=S2
GEN=1465,1474,1
1475  J=475,2475     SEC=S3
GEN=1475,1584,1
1585  J=585,2585     SEC=S2
GEN=1585,1594,1
1595  J=595,2595     SEC=S1
GEN=1595,1604,1
1605  J=605,2605     SEC=S1
GEN=1605,1615,1
1616  J=616,2616     SEC=S2
GEN=1616,1625,1
1626  J=626,2626     SEC=S3
GEN=1626,1735,1
1736  J=736,2736     SEC=S2
GEN=1736,1745,1
1746  J=746,2746     SEC=S1
GEN=1746,1755,1
1756  J=756,2756     SEC=S1
GEN=1756,1766,1
1767  J=767,2767     SEC=S2
GEN=1767,1776,1
1777  J=777,2777     SEC=S3
GEN=1777,1886,1
1887  J=887,2887     SEC=S2
GEN=1887,1896,1
1897  J=897,2897     SEC=S1
GEN=1897,1906,1
;
11004 J=10004,12004  SEC=S5
GEN=11004,11012,1
21005 J=20005,22005  SEC=S5
GEN=21005,21023,1
31005 J=30005,32005  SEC=S5
GEN=31005,31023,1
41005 J=40005,42005  SEC=S5
GEN=41005,41023,1
51005 J=50005,52005  SEC=S5
GEN=51005,51023,1
61005 J=60005,62005  SEC=S5
GEN=61005,61023,1
```

```
71004 J=70004,72004  SEC=S5
GEN=71004,71012,1

REFTEMP
ELEM=FRAME
ADD=1,900,1       T=299.15
ADD=10003,10012,1 T=299.15
ADD=20003,20022,1 T=299.15
ADD=30003,30022,1 T=299.15
ADD=40003,40022,1 T=299.15
ADD=50003,50022,1 T=299.15
ADD=60003,60022,1 T=299.15
ADD=70003,70012,1 T=299.15
ADD=2001,2900,1   T=300.15
ADD=12003,12012,1 T=300.15
ADD=22003,22012,1 T=300.15
ADD=32004,32023,1 T=300.15
ADD=42003,42012,1 T=300.15
ADD=52004,52023,1 T=300.15
ADD=62003,62012,1 T=300.15
ADD=72003,72012,1 T=300.15

LOAD
NAME=CST
TYPE=TEMP  ELEM=FRAME
ADD=1,900,1       T=288.15-45  T2=10
ADD=10003,10012,1 T=288.15-45  T2=10
ADD=20003,20022,1 T=288.15-45  T2=10
ADD=30003,30022,1 T=288.15-45  T2=10
ADD=40003,40022,1 T=288.15-45  T2=10
ADD=50003,50022,1 T=288.15-45  T2=10
ADD=60003,60022,1 T=288.15-45  T2=10
ADD=70003,70012,1 T=288.15-45  T2=10
ADD=2001,2900,1   T=283.15
ADD=12003,12012,1 T=283.15
ADD=22003,22012,1 T=283.15
ADD=32004,32023,1 T=283.15
ADD=42003,42012,1 T=283.15
ADD=52004,52023,1 T=283.15
ADD=62003,62012,1 T=283.15
ADD=72003,72012,1 T=283.15
```

B.2 Two-Span-Continuous Design

```
TWO-SPAN-CONTINUOUS

SYSTEM
LENGTH=M FORCE=KN

JOINT
1  X=0 Y=1.4  Z=6
11 X=0 Y=1.9  Z=6
LGEN=1,11,1
21 X=0 Y=2.9  Z=6
LGEN=11,21,1
131 X=0 Y=35   Z=6
LGEN=21,131,1
141 X=0 Y=36   Z=6
LGEN=131,141,1
151 X=0 Y=37   Z=6
LGEN=141,151,1
261 X=0 Y=69.1 Z=6
LGEN=151,261,1
271 X=0 Y=70.1 Z=6
LGEN=261,271,1
281 X=0 Y=70.6 Z=6
LGEN=271,281,1
282 X=0 Y=73.4 Z=6
292 X=0 Y=73.9 Z=6
LGEN=282,292,1
302 X=0 Y=74.9 Z=6
LGEN=292,302,1
412 X=0 Y=107  Z=6
LGEN=302,412,1
422 X=0 Y=108  Z=6
LGEN=412,422,1
432 X=0 Y=109  Z=6
LGEN=422,432,1
542 X=0 Y=141.1 Z=6
LGEN=432,542,1
552 X=0 Y=142.1 Z=6
LGEN=542,552,1
562 X=0 Y=142.6 Z=6
LGEN=552,562,1
563 X=0 Y=145.4 Z=6
573 X=0 Y=145.9 Z=6
LGEN=563,573,1
583 X=0 Y=146.9 Z=6
LGEN=573,583,1
```

```
693 X=0  Y=179   Z=6
LGEN=583,693,1
703 X=0  Y=180   Z=6
LGEN=693,703,1
713 X=0  Y=181   Z=6
LGEN=703,713,1
823 X=0  Y=213.1 Z=6
LGEN=713,823,1
833 X=0  Y=214.1 Z=6
LGEN=823,833,1
843 X=0  Y=214.6 Z=6
LGEN=833,843,1
;
2001  X=0  Y=1.4  Z=6.4
2011  X=0  Y=1.9  Z=6.4
LGEN=2001,2011,1
2021  X=0  Y=2.9  Z=6.4
LGEN=2011,2021,1
2131 X=0  Y=35    Z=6.4
LGEN=2021,2131,1
2141 X=0  Y=36    Z=6.4
LGEN=2131,2141,1
2151 X=0  Y=37    Z=6.4
LGEN=2141,2151,1
2261 X=0  Y=69.1  Z=6.4
LGEN=2151,2261,1
2271 X=0  Y=70.1  Z=6.4
LGEN=2261,2271,1
2281 X=0  Y=70.6  Z=6.4
LGEN=2271,2281,1
2282 X=0  Y=73.4  Z=6.4
2292 X=0  Y=73.9  Z=6.4
LGEN=2282,2292,1
2302 X=0  Y=74.9  Z=6.4
LGEN=2292,2302,1
2412 X=0  Y=107   Z=6.4
LGEN=2302,2412,1
2422 X=0  Y=108   Z=6.4
LGEN=2412,2422,1
2432 X=0  Y=109   Z=6.4
LGEN=2422,2432,1
2542 X=0  Y=141.1 Z=6.4
LGEN=2432,2542,1
2552 X=0  Y=142.1 Z=6.4
LGEN=2542,2552,1
2562 X=0  Y=142.6 Z=6.4
```

```
LGEN=2552,2562,1
2563 X=0  Y=145.4 Z=6.4
2573 X=0  Y=145.9 Z=6.4
LGEN=2563,2573,1
2583 X=0  Y=146.9 Z=6.4
LGEN=2573,2583,1
2693 X=0  Y=179   Z=6.4
LGEN=2583,2693,1
2703 X=0  Y=180   Z=6.4
LGEN=2693,2703,1
2713 X=0  Y=181   Z=6.4
LGEN=2703,2713,1
2823 X=0  Y=213.1 Z=6.4
LGEN=2713,2823,1
2833 X=0  Y=214.1 Z=6.4
LGEN=2823,2833,1
2843 X=0  Y=214.6 Z=6.4
LGEN=2833,2843,1
;
10003  X=0  Y=0    Z=5.9
10013  X=0  Y=1.4   Z=5.9
LGEN=10003,10013,1
10014  X=0  Y=1.4   Z=6
20002  X=0  Y=36    Z=0
20003  X=0  Y=36    Z=6
30002  X=0  Y=72    Z=0
30003  X=0  Y=72    Z=5.9
30004  X=0  Y=70.6  Z=5.9
30024  X=0  Y=73.4  Z=5.9
LGEN=30004,30024,1
30025  X=0  Y=70.6  Z=6
30026  X=0  Y=73.4  Z=6
40002  X=0  Y=108   Z=0
40003  X=0  Y=108   Z=6
50002  X=0  Y=144   Z=0
50003  X=0  Y=144   Z=5.9
50004  X=0  Y=142.6 Z=5.9
50024  X=0  Y=145.4 Z=5.9
LGEN=50004,50024,1
50025  X=0  Y=142.6 Z=6
50026  X=0  Y=145.4 Z=6
60002  X=0  Y=180   Z=0
60003  X=0  Y=180   Z=6
70003  X=0  Y=216   Z=5.9
70013  X=0  Y=214.6 Z=5.9
LGEN=70003,70013,1
```

```
70014 X=0 Y=214.6 Z=6
;
12003 X=0 Y=0    Z=6.4
12013 X=0 Y=1.4  Z=6.4
LGEN=12003,12013,1
32004 X=0 Y=70.6  Z=6.4
32024 X=0 Y=73.4  Z=6.4
LGEN=32004,32024,1
52004 X=0 Y=142.6 Z=6.4
52024 X=0 Y=145.4 Z=6.4
LGEN=52004,52024,1
72003 X=0 Y=216   Z=6.4
72013 X=0 Y=214.6 Z=6.4
LGEN=72003,72013,1

RESTRAINT
ADD=20002,60002,10000 DOF=ALL
ADD=12003,72003,60000 DOF=ALL
ADD=10003,70003,60000 DOF=ALL

WELD
NAME=ALL  TOL=0.000001
ADD=*

MATERIAL
NAME=BEAM   TYPE=ISO M=2.4  W=23.544
E=27670000 U=0.30  A=0.000011
NAME=COLUMN TYPE=ISO M=2.4 W=23.544
E=23420000 U=0.25
NAME=TBEAM   TYPE=ISO
E=27670000 U=0.30  A=0.000011
NAME=SPRING TYPE=ISO
E=10000
NAME=ELASTO TYPE=ORTHO
E=1110242,1110242,1110242   G=1400,1400,14000
NAME=RAIL   TYPE=ISO
E=2.1E8    U=0.30  A=12E-6
NAME=RSPRING TYPE=ISO
E=8000

FRAME SECTION
NAME=ONE     TYPE=PRISM MAT=BEAM    A=4.2374 J=1.054 I=.43567,7.1614
AS=2.862,3.2669
NAME=TWO     TYPE=PRISM MAT=BEAM    A=6.6136 J=3.883 I=2.8134,9.1271
AS=2.900,3.480
NAME=THREE   TYPE=PRISM MAT=BEAM    A=3.0664 J=2.976 I=1.8551,6.8861
AS=1.175,1.460
```

```
NAME=FOUR    TYPE=PRISM  MAT=BEAM   A=6.9040  J=4.076  I=2.9480,9.1760
AS=3.183,3.850
NAME=COLFIX  TYPE=PRISM  MAT=COLUMN  SH=P    T=2.135
NAME=COLEXP  TYPE=PRISM  MAT=COLUMN  SH=P    T=1.83
NAME=LINK    TYPE=PRISM  MAT=TBEAM   A=1000   J=1000   I=1000,1000
AS=1000,1000
NAME=BEARING TYPE=PRISM  MAT=ELASTO  SH=R    T=2*0.70,0.75
NAME=S1      TYPE=PRISM  MAT=RSPRING A=1000   I=0.5/10/187.5
J=1000
NAME=S2      TYPE=PRISM  MAT=RSPRING A=1000   I=1/10/187.5
J=1000
NAME=S3      TYPE=PRISM  MAT=RSPRING A=1000   I=32.1/110/187.5
J=1000
NAME=S4      TYPE=PRISM  MAT=RSPRING A=1000   I=1/10/187.5
J=1000
NAME=S5      TYPE=PRISM  MAT=RSPRING A=1000   I=1.4/10/96
J=1000
NAME=RAIL    TYPE=PRISM  MAT=RAIL    A=2*7257.34/1000/1000 I=2*27304781.52/
1000/1000/1000/1000

FRAME
1  J=1,2    SEC=ONE
GEN=1,10,1
11  J=11,12  SEC=TWO
GEN=11,20,1
21  J=21,22  SEC=THREE
GEN=21,130,1
131 J=131,132 SEC=FOUR
GEN=131,150,1
151 J=151,152 SEC=THREE
GEN=151,260,1
261 J=261,262 SEC=TWO
GEN=261,270,1
271 J=271,272 SEC=ONE
GEN=271,280,1
281 J=282,283 SEC=ONE
GEN=281,290,1
291 J=292,293 SEC=TWO
GEN=291,300,1
301 J=302,303 SEC=THREE
GEN=301,410,1
411 J=412,413 SEC=FOUR
GEN=411,430,1
431 J=432,433 SEC=THREE
GEN=431,540,1
541 J=542,543 SEC=TWO
GEN=541,550,1
```

```
551 J=552,553 SEC=ONE
GEN=551,560,1
561 J=563,564 SEC=ONE
GEN=561,570,1
571 J=573,574 SEC=TWO
GEN=571,580,1
581 J=583,584 SEC=THREE
GEN=581,690,1
691 J=693,694 SEC=FOUR
GEN=691,710,1
711 J=713,714 SEC=THREE
GEN=711,820,1
821 J=823,824 SEC=TWO
GEN=821,830,1
831 J=833,834 SEC=ONE
GEN=831,840,1
;
2001 J=2001,2002 SEC=RAIL
GEN=2001,2010,1
2011 J=2011,2012 SEC=RAIL
GEN=2011,2020,1
2021 J=2021,2022 SEC=RAIL
GEN=2021,2130,1
2131 J=2131,2132 SEC=RAIL
GEN=2131,2150,1
2151 J=2151,2152 SEC=RAIL
GEN=2151,2260,1
2261 J=2261,2262 SEC=RAIL
GEN=2261,2270,1
2271 J=2271,2272 SEC=RAIL
GEN=2271,2280,1
2281 J=2282,2283 SEC=RAIL
GEN=2281,2290,1
2291 J=2292,2293 SEC=RAIL
GEN=2291,2300,1
2301 J=2302,2303 SEC=RAIL
GEN=2301,2410,1
2411 J=2412,2413 SEC=RAIL
GEN=2411,2430,1
2431 J=2432,2433 SEC=RAIL
GEN=2431,2540,1
2541 J=2542,2543 SEC=RAIL
GEN=2541,2550,1
2551 J=2552,2553 SEC=RAIL
GEN=2551,2560,1
2561 J=2563,2564 SEC=RAIL
```

```
GEN=2561,2570,1
2571 J=2573,2574 SEC=RAIL
GEN=2571,2580,1
2581 J=2583,2584 SEC=RAIL
GEN=2581,2690,1
2691 J=2693,2694 SEC=RAIL
GEN=2691,2710,1
2711 J=2713,2714 SEC=RAIL
GEN=2711,2820,1
2821 J=2823,2824 SEC=RAIL
GEN=2821,2830,1
2831 J=2833,2834 SEC=RAIL
GEN=2831,2840,1
;
12003 J=12003,12004  SEC=RAIL
GEN=12003,12012,1
32004 J=32004,32005  SEC=RAIL
GEN=32004,32023,1
52004 J=52004,52005  SEC=RAIL
GEN=52004,52023,1
72003 J=72003,72004  SEC=RAIL
GEN=72003,72012,1
;
10003 J=10003,10004  SEC=LINK
GEN=10003,10012,1
10013 J=10013,10014  SEC=BEARING
20002 J=20002,20003  SEC=COLFIX
30002 J=30002,30003  SEC=COLEXP
30003 J=30004,30005  SEC=LINK
GEN=30003,30022,1
30023 J=30004,30025  SEC=BEARING
30024 J=30024,30026  SEC=BEARING
40002 J=40002,40003  SEC=COLFIX
50002 J=50002,50003  SEC=COLEXP
50003 J=50004,50005  SEC=LINK
GEN=50003,50022,1
50023 J=50004,50025  SEC=BEARING
50024 J=50024,50026  SEC=BEARING
60002 J=60002,60003  SEC=COLFIX
70003 J=70003,70004  SEC=LINK
GEN=70003,70012,1
70013 J=70013,70014  SEC=BEARING
;
1001  J=1,2001      SEC=S1
GEN=1001,1011,1
1012  J=12,2012     SEC=S2
```

```
GEN=1012,1021,1
1022  J=22,2022      SEC=S3
GEN=1022,1131,1
1132  J=132,2132    SEC=S4
GEN=1132,1151,1
1152  J=152,2152    SEC=S3
GEN=1152,1261,1
1262  J=262,2262    SEC=S2
GEN=1262,1271,1
1272  J=272,2272    SEC=S1
GEN=1272,1281,1
1282  J=282,2282    SEC=S1
GEN=1282,1292,1
1293  J=293,2293    SEC=S2
GEN=1293,1302,1
1303  J=303,2303    SEC=S3
GEN=1303,1412,1
1413  J=413,2413    SEC=S4
GEN=1413,1432,1
1433  J=433,2433    SEC=S3
GEN=1433,1542,1
1543  J=543,2543    SEC=S2
GEN=1543,1552,1
1553  J=553,2553    SEC=S1
GEN=1553,1562,1
1563  J=563,2563    SEC=S1
GEN=1563,1573,1
1574  J=574,2574    SEC=S2
GEN=1574,1583,1
1584  J=584,2584    SEC=S3
GEN=1584,1693,1
1694  J=694,2694    SEC=S4
GEN=1694,1713,1
1714  J=714,2714    SEC=S3
GEN=1714,1823,1
1824  J=824,2824    SEC=S2
GEN=1824,1833,1
1834  J=834,2834    SEC=S1
GEN=1834,1843,1
;
11004 J=10004,12004  SEC=S5
GEN=11004,11012,1
31005 J=30005,32005  SEC=S5
GEN=31005,31023,1
51005 J=50005,52005  SEC=S5
GEN=51005,51023,1
```

```
71004 J=70004,72004  SEC=S5
GEN=71004,71012,1

REFTEMP
ELEM=FRAME
ADD=1,840,1      T=299.15
ADD=10003,10012,1 T=299.15
ADD=30003,30022,1 T=299.15
ADD=50003,50022,1 T=299.15
ADD=70003,70012,1 T=299.15
ADD=2000,2840,1   T=300.15
ADD=12003,12012,1 T=300.15
ADD=32004,32023,1 T=300.15
ADD=52004,52023,1 T=300.15
ADD=72003,72012,1 T=300.15

LOAD
NAME=CST
TYPE=TEMP  ELEM=FRAME
ADD=1,840,1      T=288.15-45 T2=10
ADD=10003,10012,1 T=288.15-45  T2=10
ADD=30003,30022,1 T=288.15-45  T2=10
ADD=50003,50022,1 T=288.15-45  T2=10
ADD=70003,70012,1 T=288.15-45  T2=10
ADD=2001,2840,1   T=283.15
ADD=12003,12012,1 T=283.15
ADD=32004,32023,1 T=283.15
ADD=52004,52023,1 T=283.15
ADD=72003,72012,1 T=283.15
```

B.3 Three-Span-Continuous Design

```
THREE-SPAN-CONTINUOUS, SINGLE TRACK, THERMAL ANALYSIS

SYSTEM
LENGTH=M  FORCE=KN

JOINT
1  X=0 Y=1.4  Z=6
11  X=0  Y=1.9  Z=6
LGEN=1,11,1
21  X=0  Y=2.9  Z=6
LGEN=11,21,1
131 X=0  Y=33.1  Z=6
LGEN=21,131,1
141 X=0  Y=34.1  Z=6
LGEN=131,141,1
151 X=0  Y=34.6  Z=6
LGEN=141,151,1
```

```
152 X=0  Y=37.4  Z=6
162 X=0  Y=37.9  Z=6
LGEN=152,162,1
172 X=0  Y=39.9  Z=6
LGEN=162,172,1
282 X=0  Y=69.1  Z=6
LGEN=172,282,1
292 X=0  Y=70.1  Z=6
LGEN=282,292,1
302 X=0  Y=70.6  Z=6
LGEN=292,302,1
303 X=0  Y=73.4  Z=6
313 X=0  Y=73.9  Z=6
LGEN=303,313,1
323 X=0  Y=74.9  Z=6
LGEN=313,323,1
433 X=0  Y=105.1 Z=6
LGEN=323,433,1
443 X=0  Y=106.1 Z=6
LGEN=433,443,1
453 X=0  Y=106.6 Z=6
LGEN=443,453,1
454 X=0  Y=109.4 Z=6
464 X=0  Y=109.9 Z=6
LGEN=454,464,1
474 X=0  Y=110.9 Z=6
LGEN=464,474,1
584 X=0  Y=141.1 Z=6
LGEN=474,584,1
594 X=0  Y=142.1 Z=6
LGEN=584,594,1
604 X=0  Y=142.6 Z=6
LGEN=594,604,1
605 X=0  Y=145.4 Z=6
615 X=0  Y=145.9 Z=6
LGEN=605,615,1
625 X=0  Y=146.9 Z=6
LGEN=615,625,1
735 X=0  Y=177.1 Z=6
LGEN=625,735,1
745 X=0  Y=178.1 Z=6
LGEN=735,745,1
755 X=0  Y=178.6 Z=6
LGEN=745,755,1
756 X=0  Y=181.4 Z=6
766 X=0  Y=181.9 Z=6
LGEN=756,766,1
776 X=0  Y=182.9 Z=6
LGEN=766,776,1
886 X=0  Y=213.1 Z=6
LGEN=776,886,1
896 X=0  Y=214.1 Z=6
LGEN=886,896,1
906 X=0  Y=214.6 Z=6
```

```
LGEN=896,906,1
;
2001  X=0 Y=1.4   Z=6.4
2011  X=0 Y=1.9   Z=6.4
LGEN=2001,2011,1
2021  X=0  Y=2.9  Z=6.4
LGEN=2011,2021,1
2131 X=0  Y=33.1  Z=6.4
LGEN=2021,2131,1
2141 X=0  Y=34.1  Z=6.4
LGEN=2131,2141,1
2151 X=0  Y=34.6  Z=6.4
LGEN=2141,2151,1
2152 X=0  Y=37.4  Z=6.4
2162 X=0  Y=37.9  Z=6.4
LGEN=2152,2162,1
2172 X=0  Y=39.9  Z=6.4
LGEN=2162,2172,1
2282 X=0  Y=69.1  Z=6.4
LGEN=2172,2282,1
2292 X=0  Y=70.1  Z=6.4
LGEN=2282,2292,1
2302 X=0  Y=70.6  Z=6.4
LGEN=2292,2302,1
2303 X=0  Y=73.4  Z=6.4
2313 X=0  Y=73.9  Z=6.4
LGEN=2303,2313,1
2323 X=0  Y=74.9  Z=6.4
LGEN=2313,2323,1
2433 X=0  Y=105.1 Z=6.4
LGEN=2323,2433,1
2443 X=0  Y=106.1 Z=6.4
LGEN=2433,2443,1
2453 X=0  Y=106.6 Z=6.4
LGEN=2443,2453,1
2454 X=0  Y=109.4 Z=6.4
2464 X=0  Y=109.9 Z=6.4
LGEN=2454,2464,1
2474 X=0  Y=110.9 Z=6.4
LGEN=2464,2474,1
2584 X=0  Y=141.1 Z=6.4
LGEN=2474,2584,1
2594 X=0  Y=142.1 Z=6.4
LGEN=2584,2594,1
2604 X=0  Y=142.6 Z=6.4
LGEN=2594,2604,1
2605 X=0  Y=145.4 Z=6.4
2615 X=0  Y=145.9 Z=6.4
LGEN=2605,2615,1
2625 X=0  Y=146.9 Z=6.4
LGEN=2615,2625,1
2735 X=0  Y=177.1 Z=6.4
LGEN=2625,2735,1
2745 X=0  Y=178.1 Z=6.4
```

```
LGEN=2735,2745,1
2755 X=0  Y=178.6 Z=6.4
LGEN=2745,2755,1
2756 X=0  Y=181.4 Z=6.4
2766 X=0  Y=181.9 Z=6.4
LGEN=2756,2766,1
2776 X=0  Y=182.9 Z=6.4
LGEN=2766,2776,1
2886 X=0  Y=213.1 Z=6.4
LGEN=2776,2886,1
2896 X=0  Y=214.1 Z=6.4
LGEN=2886,2896,1
2906 X=0  Y=214.6 Z=6.4
LGEN=2896,2906,1
;
10003  X=0  Y=0     Z=5.9
10013  X=0  Y=1.4   Z=5.9
LGEN=10003,10013,1
10014  X=0  Y=1.4   Z=6
20002  X=0  Y=36    Z=0
20003  X=0  Y=36    Z=6
20004  X=0  Y=34.6  Z=6
20024  X=0  Y=37.4  Z=6
LGEN=20004,20024,1
30002  X=0  Y=72    Z=0
30003  X=0  Y=72    Z=6
30004  X=0  Y=70.6  Z=6
30024  X=0  Y=73.4  Z=6
LGEN=30004,30024,1
40002  X=0  Y=108   Z=0
40003  X=0  Y=108   Z=5.9
40004  X=0  Y=106.6 Z=5.9
40024  X=0  Y=109.4 Z=5.9
LGEN=40004,40024,1
40025  X=0  Y=106.6 Z=6
40026  X=0  Y=109.4 Z=6
50002  X=0  Y=144   Z=0
50003  X=0  Y=144   Z=6
50004  X=0  Y=142.6 Z=6
50024  X=0  Y=145.4 Z=6
LGEN=50004,50024,1
60002  X=0  Y=180   Z=0
60003  X=0  Y=180   Z=6
60004  X=0  Y=178.6 Z=6
60024  X=0  Y=181.4 Z=6
LGEN=60004,60024,1
70003  X=0  Y=216   Z=5.9
70013  X=0  Y=214.6 Z=5.9
LGEN=70003,70013,1
70014  X=0  Y=214.6 Z=6
;
12003  X=0  Y=0     Z=6.4
12013  X=0  Y=1.4   Z=6.4
LGEN=12003,12013,1
```

```
22004  X=0  Y=34.6   Z=6.4
22024  X=0  Y=37.4   Z=6.4
LGEN=22004,22024,1
32004  X=0  Y=70.6   Z=6.4
32024  X=0  Y=73.4   Z=6.4
LGEN=32004,32024,1
42004  X=0  Y=106.6  Z=6.4
42024  X=0  Y=109.4  Z=6.4
LGEN=42004,42024,1
52004  X=0  Y=142.6  Z=6.4
52024  X=0  Y=145.4  Z=6.4
LGEN=52004,52024,1
62004  X=0  Y=178.6  Z=6.4
62024  X=0  Y=181.4  Z=6.4
LGEN=62004,62024,1
72003  X=0  Y=216    Z=6.4
72013  X=0  Y=214.6  Z=6.4
LGEN=72003,72013,1

RESTRAINT
ADD=20002,60002,10000  DOF=ALL
ADD=12003,72003,60000  DOF=ALL
ADD=10003,70003,60000  DOF=ALL

WELD
NAME=ALL  TOL=0.000001
ADD=*

MATERIAL
NAME=BEAM   TYPE=ISO  M=2.4  W=23.544
E=27670000  U=0.30   A=0.000011
NAME=COLUMN  TYPE=ISO  M=2.4  W=23.544
E=23420000  U=0.25
NAME=TBEAM  TYPE=ISO
E=27670000  U=0.30   A=0.000011
NAME=ELASTO  TYPE=ORTHO
E=1284349,1284349,1284349    G=1400,1400,1400
NAME=RAIL   TYPE=ISO
E=2.1E8 U=0.30  A=12E-6
NAME=RSPRING TYPE=ISO
E=8000

FRAME SECTION
NAME=ONE     TYPE=PRISM  MAT=BEAM   A=4.2374  J=1.054  I=.43567,7.1614
AS=2.862,3.2669
NAME=TWO     TYPE=PRISM  MAT=BEAM   A=6.6136  J=3.883  I=2.8134,9.1271
AS=2.900,3.480
NAME=THREE   TYPE=PRISM  MAT=BEAM   A=3.0664  J=2.976  I=1.8551,6.8861
AS=1.175,1.460
NAME=FOUR    TYPE=PRISM  MAT=BEAM   A=6.9040  J=4.076  I=2.9480,9.1760
AS=3.183,3.850
NAME=COLFIX  TYPE=PRISM  MAT=COLUMN  SH=P    T=1.83
NAME=COLEXP  TYPE=PRISM  MAT=COLUMN  SH=P    T=1.83
NAME=LINK    TYPE=PRISM  MAT=TBEAM  A=1000   J=1000   I=1000,1000
```

```
AS=1000,1000
NAME=BEARING TYPE=PRISM MAT=ELASTO SH=R    T=2*0.85,0.85
NAME=S1     TYPE=PRISM  MAT=RSPRING A=1000   I=0.5/10/187.5
J=1000
NAME=S2     TYPE=PRISM  MAT=RSPRING A=1000   I=1/10/187.5
J=1000
NAME=S3     TYPE=PRISM  MAT=RSPRING A=1000   I=32.1/110/187.5
J=1000
NAME=S4     TYPE=PRISM  MAT=RSPRING A=1000   I=1/10/187.5
J=1000
NAME=S5     TYPE=PRISM  MAT=RSPRING A=1000   I=1.4/10/96
J=1000
NAME=RAIL   TYPE=PRISM  MAT=RAIL   A=2*7257.34/1000/1000 I=2*27304781.52/
1000/1000/1000/1000

FRAME
1  J=1,2    SEC=ONE
GEN=1,10,1
11  J=11,12   SEC=TWO
GEN=11,20,1
21  J=21,22   SEC=THREE
GEN=21,130,1
131 J=131,132 SEC=THREE
GEN=131,140,1
141 J=141,142 SEC=THREE
GEN=141,150,1
151 J=152,153 SEC=THREE
GEN=151,160,1
161 J=162,163 SEC=THREE
GEN=161,170,1
171 J=172,173 SEC=THREE
GEN=171,280,1
281 J=282,283 SEC=THREE
GEN=281,290,1
291 J=292,293 SEC=THREE
GEN=291,300,1
301 J=303,304 SEC=THREE
GEN=301,310,1
311 J=313,314 SEC=THREE
GEN=311,320,1
321 J=323,324 SEC=THREE
GEN=321,430,1
431 J=433,434 SEC=TWO
GEN=431,440,1
441 J=443,444 SEC=ONE
GEN=441,450,1
451 J=454,455 SEC=ONE
GEN=451,460,1
461 J=464,465 SEC=TWO
GEN=461,470,1
471 J=474,475 SEC=THREE
GEN=471,580,1
581 J=584,585 SEC=THREE
GEN=581,590,1
```

```
591 J=594,595 SEC=THREE
GEN=591,600,1
601 J=605,606 SEC=THREE
GEN=601,610,1
611 J=615,616 SEC=THREE
GEN=611,620,1
621 J=625,626 SEC=THREE
GEN=621,730,1
731 J=735,736 SEC=THREE
GEN=731,740,1
741 J=745,746 SEC=THREE
GEN=741,750,1
751 J=756,757 SEC=THREE
GEN=751,760,1
761 J=766,767 SEC=THREE
GEN=761,770,1
771 J=776,777 SEC=THREE
GEN=771,880,1
881 J=886,887 SEC=TWO
GEN=881,890,1
891 J=896,897 SEC=ONE
GEN=891,900,1
;
2001 J=2001,2002 SEC=RAIL
GEN=2001,2010,1
2011 J=2011,2012 SEC=RAIL
GEN=2011,2020,1
2021 J=2021,2022 SEC=RAIL
GEN=2021,2130,1
2131 J=2131,2132 SEC=RAIL
GEN=2131,2140,1
2141 J=2141,2142 SEC=RAIL
GEN=2141,2150,1
2151 J=2152,2153 SEC=RAIL
GEN=2151,2160,1
2161 J=2162,2163 SEC=RAIL
GEN=2161,2170,1
2171 J=2172,2173 SEC=RAIL
GEN=2171,2280,1
2281 J=2282,2283 SEC=RAIL
GEN=2281,2290,1
2291 J=2292,2293 SEC=RAIL
GEN=2291,2300,1
2301 J=2303,2304 SEC=RAIL
GEN=2301,2310,1
2311 J=2313,2314 SEC=RAIL
GEN=2311,2320,1
2321 J=2323,2324 SEC=RAIL
GEN=2321,2430,1
2431 J=2433,2434 SEC=RAIL
GEN=2431,2440,1
2441 J=2443,2444 SEC=RAIL
GEN=2441,2450,1
2451 J=2454,2455 SEC=RAIL
```

```
GEN=2451,2460,1
2461 J=2464,2465 SEC=RAIL
GEN=2461,2470,1
2471 J=2474,2475 SEC=RAIL
GEN=2471,2580,1
2581 J=2584,2585 SEC=RAIL
GEN=2581,2590,1
2591 J=2594,2595 SEC=RAIL
GEN=2591,2600,1
2601 J=2605,2606 SEC=RAIL
GEN=2601,2610,1
2611 J=2615,2616 SEC=RAIL
GEN=2611,2620,1
2621 J=2625,2626 SEC=RAIL
GEN=2621,2730,1
2731 J=2735,2736 SEC=RAIL
GEN=2731,2740,1
2741 J=2745,2746 SEC=RAIL
GEN=2741,2750,1
2751 J=2756,2757 SEC=RAIL
GEN=2751,2760,1
2761 J=2766,2767 SEC=RAIL
GEN=2761,2770,1
2771 J=2776,2777 SEC=RAIL
GEN=2771,2880,1
2881 J=2886,2887 SEC=RAIL
GEN=2881,2890,1
2891 J=2896,2897 SEC=RAIL
GEN=2891,2900,1
;
12003 J=12003,12004  SEC=RAIL
GEN=12003,12012,1
22004 J=22004,22005  SEC=RAIL
GEN=22004,22023,1
32004 J=32004,32005  SEC=RAIL
GEN=32004,32023,1
42004 J=42004,42005  SEC=RAIL
GEN=42004,42023,1
52004 J=52004,52005  SEC=RAIL
GEN=52004,52023,1
62004 J=62004,62005  SEC=RAIL
GEN=62004,62023,1
72003 J=72003,72004  SEC=RAIL
GEN=72003,72012,1
;
10003 J=10003,10004  SEC=LINK
GEN=10003,10012,1
10013 J=10013,10014  SEC=BEARING
20002 J=20002,20003  SEC=COLFIX
20003 J=20004,20005  SEC=THREE
GEN=20003,20009,1
20010 J=20011,20012  SEC=FOUR
GEN=20010,20015,1
20016 J=20017,20018  SEC=THREE
```

```
GEN=20016,20022,1
30002 J=30002,30003  SEC=COLFIX
30003 J=30004,30005  SEC=THREE
GEN=30003,30009,1
30010 J=30011,30012  SEC=FOUR
GEN=30010,30015,1
30016 J=30017,30018  SEC=THREE
GEN=30016,30022,1
40002 J=40002,40003  SEC=COLEXP
40003 J=40004,40005  SEC=LINK
GEN=40003,40022,1
40023 J=40004,40025  SEC=BEARING
40024 J=40024,40026  SEC=BEARING
50002 J=50002,50003  SEC=COLFIX
50003 J=50004,50005  SEC=THREE
GEN=50003,50009,1
50010 J=50011,50012  SEC=FOUR
GEN=50010,50015,1
50016 J=50017,50018  SEC=THREE
GEN=50016,50022,1
60002 J=60002,60003  SEC=COLFIX
60003 J=60004,60005  SEC=THREE
GEN=60003,60009,1
60010 J=60011,60012  SEC=FOUR
GEN=60010,60015,1
60016 J=60017,60018  SEC=THREE
GEN=60016,60022,1
70003 J=70003,70004  SEC=LINK
GEN=70003,70012,1
70013 J=70013,70014  SEC=BEARING
;
1001  J=1,2001       SEC=S1
GEN=1001,1011,1
1012  J=12,2012      SEC=S2
GEN=1012,1021,1
1022  J=22,2022      SEC=S3
GEN=1022,1131,1
1132  J=132,2132     SEC=S2
GEN=1132,1141,1
1142  J=142,2142     SEC=S1
GEN=1142,1151,1
1152  J=152,2152     SEC=S1
GEN=1152,1162,1
1163  J=163,2163     SEC=S2
GEN=1163,1172,1
1173  J=173,2173     SEC=S3
GEN=1173,1302,1
1283  J=283,2283     SEC=S2
GEN=1283,1292,1
1293  J=293,2293     SEC=S1
GEN=1293,1302,1
1303  J=303,2303     SEC=S1
GEN=1303,1313,1
1314  J=314,2314     SEC=S2
```

```
GEN=1314,1323,1
1324 J=324,2324    SEC=S3
GEN=1324,1433,1
1434 J=434,2434    SEC=S2
GEN=1434,1443,1
1444 J=444,2444    SEC=S1
GEN=1444,1453,1
1454 J=454,2454    SEC=S1
GEN=1454,1464,1
1465 J=465,2465    SEC=S2
GEN=1465,1474,1
1475 J=475,2475    SEC=S3
GEN=1475,1584,1
1585 J=585,2585    SEC=S2
GEN=1585,1594,1
1595 J=595,2595    SEC=S1
GEN=1595,1604,1
1605 J=605,2605    SEC=S1
GEN=1605,1615,1
1616 J=616,2616    SEC=S2
GEN=1616,1625,1
1626 J=626,2626    SEC=S3
GEN=1626,1735,1
1736 J=736,2736    SEC=S2
GEN=1736,1745,1
1746 J=746,2746    SEC=S1
GEN=1746,1755,1
1756 J=756,2756    SEC=S1
GEN=1756,1766,1
1767 J=767,2767    SEC=S2
GEN=1767,1776,1
1777 J=777,2777    SEC=S3
GEN=1777,1886,1
1887 J=887,2887    SEC=S2
GEN=1887,1896,1
1897 J=897,2897    SEC=S1
GEN=1897,1906,1
;
11004 J=10004,12004  SEC=S5
GEN=11004,11012,1
21005 J=20005,22005  SEC=S5
GEN=21005,21023,1
31005 J=30005,32005  SEC=S5
GEN=31005,31023,1
41005 J=40005,42005  SEC=S5
GEN=41005,41023,1
51005 J=50005,52005  SEC=S5
GEN=51005,51023,1
61005 J=60005,62005  SEC=S5
GEN=61005,61023,1
71004 J=70004,72004  SEC=S5
GEN=71004,71012,1

REFTEMP
```

```
ELEM=FRAME
ADD=1,900,1      T=299.15
ADD=10003,10012,1 T=299.15
ADD=20003,20012,1 T=299.15
ADD=20013,20022,1 T=299.15
ADD=30003,30012,1 T=299.15
ADD=30013,30022,1 T=299.15
ADD=40003,40012,1 T=299.15
ADD=40013,40022,1 T=299.15
ADD=50003,50012,1 T=299.15
ADD=50013,50022,1 T=299.15
ADD=60003,60012,1 T=299.15
ADD=60013,60022,1 T=299.15
ADD=70003,70012,1 T=299.15
ADD=2001,2900,1  T=300.15
ADD=12003,12012,1 T=300.15
ADD=22003,22012,1 T=300.15
ADD=32004,32023,1 T=300.15
ADD=42003,42012,1 T=300.15
ADD=52004,52023,1 T=300.15
ADD=62003,62012,1 T=300.15
ADD=72003,72012,1 T=300.15

LOAD
NAME=CST
TYPE=TEMP  ELEM=FRAME
ADD=1,150,1      T=288.15-45  T2=10
ADD=151,300,1    T=288.15-45  T2=10
ADD=301,450,1    T=288.15-45  T2=10
ADD=451,600,1    T=288.15-45  T2=10
ADD=601,750,1    T=288.15-45  T2=10
ADD=751,900,1    T=288.15-45  T2=10
ADD=10003,10012,1 T=288.15-45  T2=10
ADD=20003,20012,1 T=288.15-45  T2=10
ADD=20013,20022,1 T=288.15-45  T2=10
ADD=30003,30012,1 T=288.15-45  T2=10
ADD=30013,30022,1 T=288.15-45  T2=10
ADD=40003,40012,1 T=288.15-45  T2=10
ADD=40013,40022,1 T=288.15-45  T2=10
ADD=50003,50012,1 T=288.15-45  T2=10
ADD=50013,50022,1 T=288.15-45  T2=10
ADD=60003,60012,1 T=288.15-45  T2=10
ADD=60013,60022,1 T=288.15-45  T2=10
ADD=70003,70012,1 T=288.15-45  T2=10
ADD=2001,2900,1  T=283.15
ADD=12003,12012,1 T=283.15
ADD=22003,22012,1 T=283.15
ADD=32004,32023,1 T=283.15
ADD=42003,42012,1 T=283.15
ADD=52004,52023,1 T=283.15
ADD=62003,62012,1 T=283.15
ADD=72003,72012,1 T=283.15
```

B.4 Four-Span-Continuous Design

```
FOUR-SPAN-CONTINUOUS

SYSTEM
LENGTH=M  FORCE=KN

JOINT
1  X=0  Y=1.4  Z=6
11  X=0  Y=1.9  Z=6
LGEN=1,11,1
21  X=0  Y=2.9  Z=6
LGEN=11,21,1
131 X=0  Y=35   Z=6
LGEN=21,131,1
141 X=0  Y=36   Z=6
LGEN=131,141,1
151 X=0  Y=37   Z=6
LGEN=141,151,1
261 X=0  Y=69.1  Z=6
LGEN=151,261,1
271 X=0  Y=70.1  Z=6
LGEN=261,271,1
281 X=0  Y=70.6  Z=6
LGEN=271,281,1
282 X=0  Y=73.4  Z=6
292 X=0  Y=73.9  Z=6
LGEN=282,292,1
302 X=0  Y=74.9  Z=6
LGEN=292,302,1
412 X=0  Y=107   Z=6
LGEN=302,412,1
422 X=0  Y=108   Z=6
LGEN=412,422,1
432 X=0  Y=109   Z=6
LGEN=422,432,1
542 X=0  Y=141.1 Z=6
LGEN=432,542,1
552 X=0  Y=142.1 Z=6
LGEN=542,552,1
562 X=0  Y=142.6 Z=6
LGEN=552,562,1
563 X=0  Y=145.4 Z=6
573 X=0  Y=145.9 Z=6
LGEN=563,573,1
583 X=0  Y=146.9 Z=6
LGEN=573,583,1
693 X=0  Y=179   Z=6
LGEN=583,693,1
703 X=0  Y=180   Z=6
LGEN=693,703,1
713 X=0  Y=181   Z=6
LGEN=703,713,1
823 X=0  Y=213.1 Z=6
```

```
LGEN=713,823,1
833 X=0  Y=214.1 Z=6
LGEN=823,833,1
843 X=0  Y=214.6 Z=6
LGEN=833,843,1
;
2001  X=0  Y=1.4  Z=6.4
2011  X=0  Y=1.9  Z=6.4
LGEN=2001,2011,1
2021  X=0  Y=2.9  Z=6.4
LGEN=2011,2021,1
2131 X=0  Y=35   Z=6.4
LGEN=2021,2131,1
2141 X=0  Y=36   Z=6.4
LGEN=2131,2141,1
2151 X=0  Y=37   Z=6.4
LGEN=2141,2151,1
2261 X=0  Y=69.1  Z=6.4
LGEN=2151,2261,1
2271 X=0  Y=70.1  Z=6.4
LGEN=2261,2271,1
2281 X=0  Y=70.6  Z=6.4
LGEN=2271,2281,1
2282 X=0  Y=73.4  Z=6.4
2292 X=0  Y=73.9  Z=6.4
LGEN=2282,2292,1
2302 X=0  Y=74.9  Z=6.4
LGEN=2292,2302,1
2412 X=0  Y=107   Z=6.4
LGEN=2302,2412,1
2422 X=0  Y=108   Z=6.4
LGEN=2412,2422,1
2432 X=0  Y=109   Z=6.4
LGEN=2422,2432,1
2542 X=0  Y=141.1 Z=6.4
LGEN=2432,2542,1
2552 X=0  Y=142.1 Z=6.4
LGEN=2542,2552,1
2562 X=0  Y=142.6 Z=6.4
LGEN=2552,2562,1
2563 X=0  Y=145.4 Z=6.4
2573 X=0  Y=145.9 Z=6.4
LGEN=2563,2573,1
2583 X=0  Y=146.9 Z=6.4
LGEN=2573,2583,1
2693 X=0  Y=179   Z=6.4
LGEN=2583,2693,1
2703 X=0  Y=180   Z=6.4
LGEN=2693,2703,1
2713 X=0  Y=181   Z=6.4
LGEN=2703,2713,1
2823 X=0  Y=213.1 Z=6.4
LGEN=2713,2823,1
2833 X=0  Y=214.1 Z=6.4
```

```
LGEN=2823,2833,1
2843 X=0  Y=214.6 Z=6.4
LGEN=2833,2843,1
;
10003  X=0  Y=0    Z=5.9
10013  X=0  Y=1.4  Z=5.9
LGEN=10003,10013,1
10014  X=0  Y=1.4  Z=6
20002  X=0  Y=36   Z=0
20003  X=0  Y=36   Z=6
30002  X=0  Y=72   Z=0
30003  X=0  Y=72   Z=6
30004  X=0  Y=70.6  Z=6
30024  X=0  Y=73.4  Z=6
LGEN=30004,30024,1
40002  X=0  Y=108   Z=0
40003  X=0  Y=108   Z=6
50002  X=0  Y=144   Z=0
50003  X=0  Y=144   Z=5.9
50004  X=0  Y=142.6 Z=5.9
50024  X=0  Y=145.4 Z=5.9
LGEN=50004,50024,1
50025  X=0  Y=142.6 Z=6
50026  X=0  Y=145.4 Z=6
60002  X=0  Y=180   Z=0
60003  X=0  Y=180   Z=6
70003  X=0  Y=216   Z=5.9
70013  X=0  Y=214.6 Z=5.9
LGEN=70003,70013,1
70014  X=0  Y=214.6 Z=6
;
12003  X=0  Y=0     Z=6.4
12013  X=0  Y=1.4   Z=6.4
LGEN=12003,12013,1
32004  X=0  Y=70.6   Z=6.4
32024  X=0  Y=73.4   Z=6.4
LGEN=32004,32024,1
52004  X=0  Y=142.6  Z=6.4
52024  X=0  Y=145.4  Z=6.4
LGEN=52004,52024,1
72003  X=0  Y=216    Z=6.4
72013  X=0  Y=214.6  Z=6.4
LGEN=72003,72013,1

RESTRAINT
ADD=20002,60002,10000  DOF=ALL
ADD=12003,72003,60000  DOF=ALL
ADD=10003,70003,60000  DOF=ALL

WELD
NAME=ALL  TOL=0.000001
ADD=*

MATERIAL
```

```
NAME=BEAM   TYPE=ISO  M=2.4  W=23.544
E=34970000  U=0.30  A=0.000011
NAME=COLUMN  TYPE=ISO  M=2.4  W=23.544
E=27670000  U=0.25
NAME=TBEAM  TYPE=ISO
E=34970000  U=0.30  A=0.000011
NAME=ELASTO  TYPE=ORTHO
E=1496953,1496953,1496953    G=1400,1400,1400
NAME=ELASTO2  TYPE=ORTHO
E=898736,898736,898736    G=1400,1400,1400
NAME=RAIL   TYPE=ISO
E=2.1E8 U=0.30  A=12E-6
NAME=RSPRING TYPE=ISO
E=8000

FRAME SECTION
NAME=ONE     TYPE=PRISM  MAT=BEAM    A=4.2374  J=1.054  I=.43567,7.1614
AS=2.862,3.2669
NAME=TWO     TYPE=PRISM  MAT=BEAM    A=6.6136  J=3.883  I=2.8134,9.1271
AS=2.900,3.480
NAME=THREE   TYPE=PRISM  MAT=BEAM    A=3.0664  J=2.976  I=1.8551,6.8861
AS=1.175,1.460
NAME=FOUR    TYPE=PRISM  MAT=BEAM    A=6.9040  J=4.076  I=2.9480,9.1760
AS=3.183,3.850
NAME=COLFIX  TYPE=PRISM  MAT=COLUMN  SH=P    T=1.83
NAME=COLEXP  TYPE=PRISM  MAT=COLUMN  SH=P    T=1.83
NAME=LINK    TYPE=PRISM  MAT=TBEAM   A=1000   J=1000   I=1000,1000
AS=1000,1000
NAME=BEARING  TYPE=PRISM  MAT=ELASTO  SH=R    T=2*1.05,1.05
NAME=BEARING2 TYPE=PRISM  MAT=ELASTO2 SH=R    T=2*0.55,0.60
NAME=S1      TYPE=PRISM  MAT=RSPRING A=1000   I=0.5/10/187.5
J=1000
NAME=S2      TYPE=PRISM  MAT=RSPRING A=1000   I=1/10/187.5
J=1000
NAME=S3      TYPE=PRISM  MAT=RSPRING A=1000   I=32.1/110/187.5
J=1000
NAME=S4      TYPE=PRISM  MAT=RSPRING A=1000   I=1/10/187.5
J=1000
NAME=S5      TYPE=PRISM  MAT=RSPRING A=1000   I=1.4/10/96
J=1000
NAME=RAIL    TYPE=PRISM  MAT=RAIL    A=2*7257.34/1000/1000 I=2*27304781.52/
1000/1000/1000/1000

FRAME
1  J=1,2   SEC=ONE
GEN=1,10,1
11  J=11,12   SEC=TWO
GEN=11,20,1
21  J=21,22   SEC=THREE
GEN=21,130,1
131 J=131,132 SEC=FOUR
GEN=131,150,1
151 J=151,152 SEC=THREE
GEN=151,260,1
```

```
261 J=261,262 SEC=THREE
GEN=261,270,1
271 J=271,272 SEC=THREE
GEN=271,280,1
281 J=282,283 SEC=THREE
GEN=281,290,1
291 J=292,293 SEC=THREE
GEN=291,300,1
301 J=302,303 SEC=THREE
GEN=301,410,1
411 J=412,413 SEC=FOUR
GEN=411,430,1
431 J=432,433 SEC=THREE
GEN=431,540,1
541 J=542,543 SEC=TWO
GEN=541,550,1
551 J=552,553 SEC=ONE
GEN=551,560,1
561 J=563,564 SEC=ONE
GEN=561,570,1
571 J=573,574 SEC=TWO
GEN=571,580,1
581 J=583,584 SEC=THREE
GEN=581,690,1
691 J=693,694 SEC=FOUR
GEN=691,710,1
711 J=713,714 SEC=THREE
GEN=711,820,1
821 J=823,824 SEC=TWO
GEN=821,830,1
831 J=833,834 SEC=ONE
GEN=831,840,1
;
2001 J=2001,2002 SEC=RAIL
GEN=2001,2010,1
2011 J=2011,2012 SEC=RAIL
GEN=2011,2020,1
2021 J=2021,2022 SEC=RAIL
GEN=2021,2130,1
2131 J=2131,2132 SEC=RAIL
GEN=2131,2150,1
2151 J=2151,2152 SEC=RAIL
GEN=2151,2260,1
2261 J=2261,2262 SEC=RAIL
GEN=2261,2270,1
2271 J=2271,2272 SEC=RAIL
GEN=2271,2280,1
2281 J=2282,2283 SEC=RAIL
GEN=2281,2290,1
2291 J=2292,2293 SEC=RAIL
GEN=2291,2300,1
2301 J=2302,2303 SEC=RAIL
GEN=2301,2410,1
2411 J=2412,2413 SEC=RAIL
```

```
GEN=2411,2430,1
2431 J=2432,2433 SEC=RAIL
GEN=2431,2540,1
2541 J=2542,2543 SEC=RAIL
GEN=2541,2550,1
2551 J=2552,2553 SEC=RAIL
GEN=2551,2560,1
2561 J=2563,2564 SEC=RAIL
GEN=2561,2570,1
2571 J=2573,2574 SEC=RAIL
GEN=2571,2580,1
2581 J=2583,2584 SEC=RAIL
GEN=2581,2690,1
2691 J=2693,2694 SEC=RAIL
GEN=2691,2710,1
2711 J=2713,2714 SEC=RAIL
GEN=2711,2820,1
2821 J=2823,2824 SEC=RAIL
GEN=2821,2830,1
2831 J=2833,2834 SEC=RAIL
GEN=2831,2840,1
;
12003 J=12003,12004  SEC=RAIL
GEN=12003,12012,1
32004 J=32004,32005  SEC=RAIL
GEN=32004,32023,1
52004 J=52004,52005  SEC=RAIL
GEN=52004,52023,1
72003 J=72003,72004  SEC=RAIL
GEN=72003,72012,1
;
10003 J=10003,10004  SEC=LINK
GEN=10003,10012,1
10013 J=10013,10014  SEC=BEARING
20002 J=20002,20003  SEC=COLFIX
30002 J=30002,30003  SEC=COLFIX
30003 J=30004,30005  SEC=THREE
GEN=30003,30009,1
30010 J=30011,30012  SEC=FOUR
GEN=30010,30015,1
30016 J=30017,30018  SEC=THREE
GEN=30016,30022,1
40002 J=40002,40003  SEC=COLFIX
50002 J=50002,50003  SEC=COLEXP
50003 J=50004,50005  SEC=LINK
GEN=50003,50022,1
50023 J=50004,50025  SEC=BEARING
50024 J=50024,50026  SEC=BEARING2
60002 J=60002,60003  SEC=COLFIX
70003 J=70003,70004  SEC=LINK
GEN=70003,70012,1
70013 J=70013,70014  SEC=BEARING2
;
1001  J=1,2001       SEC=S1
```

```
GEN=1001,1011,1
1012  J=12,2012     SEC=S2
GEN=1012,1021,1
1022  J=22,2022     SEC=S3
GEN=1022,1131,1
1132  J=132,2132    SEC=S4
GEN=1132,1151,1
1152  J=152,2152    SEC=S3
GEN=1152,1261,1
1262  J=262,2262    SEC=S2
GEN=1262,1271,1
1272  J=272,2272    SEC=S1
GEN=1272,1281,1
1282  J=282,2282    SEC=S1
GEN=1282,1292,1
1293  J=293,2293    SEC=S2
GEN=1293,1302,1
1303  J=303,2303    SEC=S3
GEN=1303,1412,1
1413  J=413,2413    SEC=S4
GEN=1413,1432,1
1433  J=433,2433    SEC=S3
GEN=1433,1542,1
1543  J=543,2543    SEC=S2
GEN=1543,1552,1
1553  J=553,2553    SEC=S1
GEN=1553,1562,1
1563  J=563,2563    SEC=S1
GEN=1563,1573,1
1574  J=574,2574    SEC=S2
GEN=1574,1583,1
1584  J=584,2584    SEC=S3
GEN=1584,1693,1
1694  J=694,2694    SEC=S4
GEN=1694,1713,1
1714  J=714,2714    SEC=S3
GEN=1714,1823,1
1824  J=824,2824    SEC=S2
GEN=1824,1833,1
1834  J=834,2834    SEC=S1
GEN=1834,1843,1
;
11004 J=10004,12004  SEC=S5
GEN=11004,11012,1
31005 J=30005,32005  SEC=S5
GEN=31005,31023,1
51005 J=50005,52005  SEC=S5
GEN=51005,51023,1
71004 J=70004,72004  SEC=S5
GEN=71004,71012,1

REFTEMP
ELEM=FRAME
ADD=1,840,1     T=299.15
```

```
ADD=10003,10012,1 T=299.15
ADD=30003,30012,1 T=299.15
ADD=30013,30022,1 T=299.15
ADD=50003,50012,1 T=299.15
ADD=50013,50022,1 T=299.15
ADD=70003,70012,1 T=299.15
ADD=2000,2840,1   T=300.15
ADD=12003,12012,1 T=300.15
ADD=32004,32023,1 T=300.15
ADD=52004,52023,1 T=300.15
ADD=72003,72012,1 T=300.15

LOAD
NAME=CST
TYPE=TEMP  ELEM=FRAME
ADD=1,140,1      T=288.15-45  T2=10
ADD=141,280,1    T=288.15-45  T2=10
ADD=281,420,1    T=288.15-45  T2=10
ADD=421,560,1    T=288.15-45  T2=10
ADD=561,700,1    T=288.15-45  T2=10
ADD=701,840,1    T=288.15-45  T2=10
ADD=10003,10012,1 T=288.15-45  T2=10
ADD=30003,30012,1 T=288.15-45  T2=10
ADD=30013,30022,1 T=288.15-45  T2=10
ADD=50003,50012,1 T=288.15-45  T2=10
ADD=50013,50022,1 T=288.15-45  T2=10
ADD=70003,70012,1 T=288.15-45  T2=10
ADD=2000,2840,1   T=283.15
ADD=12003,12012,1 T=283.15
ADD=32004,32023,1 T=283.15
ADD=52004,52023,1 T=283.15
ADD=72003,72012,1 T=283.15
```

Appendix C

Elastomeric Bearings Properties

The bearings used in the Tren Urbano Project are laminated bearings comprising various slabs of elastomer bonded to metal plates in sandwich form. These are sometimes known as reinforced elastomeric bearings. The elastomer used is natural rubber, and the metal is stainless steel.

For the analysis presented in Chapter 5, the thickness of all bearings was taken as 10 cm. The material and thickness taken for each of the layers are listed in Table C1.

Table C1: Material and thickness of layers of elastomeric bearings.

Layer No.	Material	Thickness (mm)
1	Steel	8
2	Rubber	5
3	Steel	4
4	Rubber	10
5	Steel	4
6	Rubber	10
7	Steel	4
8	Rubber	10
9	Steel	4
10	Rubber	10
11	Steel	4
12	Rubber	10
13	Steel	4
14	Rubber	5
15	Steel	8

To model the bearings in the analysis of Chapter 5, it was necessary to assign to the bearing models a modulus of elasticity and a shear modulus. The rubber used in the bearings has a hardness of 70 IRHD. The properties of this rubber are known: elastic modulus of E = 6200 kN/m^2, a bulk modulus of E_b = 2,200,000 kN/m^2, and a shear modulus of G= 1400 kN/m^2. Reinforcing the bearing with steel does not affect its shear modulus, but it changes the elastic modulus. Therefore, the shear modulus given to the model is that of the rubber.

The calculate the elastic modulus, first, the shape factor (*S*) is computed:

$$S = \frac{LB}{2t(L+B)}$$

where:

L = Length of the bearing;

B = width of the bearing;

t = thickness of an inner layer of elastomer (10 mm).

The purpose of adding steel to the bearing is to increase the shape factor. For example, if the bearing of the analysis done for this research is not reinforced, then *t* would be equal to 10 cm, and the shape factor would be greatly reduced.

Knowing the shape factor, the "apparent modulus" can be calculated:

$$Ea = E(1 + 2KS^2)$$

where *K* is an empirically determined constant (Long, 1974). For rubber with a hardness of 70 IRHD, *K* is equal to 0.55.

Finally, the effects of the bulk modulus of the elastomer have to be considered. The "modified modulus is then computed:

$$Em = \frac{Ea}{1 + \frac{Ea}{Eb}}$$

The modified modulus was then taken as the elastic modulus of the bearing for the model discussed in Chapter 5.

Appendix D

Column Design

For the model used in the analysis presented in Chapter 5, column dimensions of length and diameter were given. Also given were properties of modulus of elasticity, density, and Poisson's ratio. This properties corresponded to a concrete of a certain strength f'_c. Once the loads that were acting on the column were known, it was necessary to determine how much reinforcement the column required.

The steps taken to calculate the reinforcement were:

1. Check for slenderness effects. Slenderness effects have to be considered if:

$$\frac{kL_u}{h} > 5.5$$

where k is the effective length factor for compression members, L_u is the unsupported length of the column, and h is the diameter of the column. The columns had to be checked for slenderness in both the longitudinal, and the transversal direction of the bridge. For the transversal direction of the bridge is a good estimate to take $k = 2$. Using this value, it was always found in the analysis that slenderness effects had to be considered.

2. Compute factor to approximate the effects of creep in the columns:

$$\beta_d = \frac{M_d}{M}$$

where M_d is the moment produced by the dead load, and M is the maximum moment acting in the column.

3. Compute the gross moment of inertia of the column:

$$I_g = \frac{\pi h^4}{64}$$

4. Compute flexural rigidity:

$$EI = \frac{\frac{E_c I_g}{2.5}}{1 + \beta_d}$$

where E_c is the modulus of Elasticity of the concrete.

5. Compute critical buckling load:

$$P_c = \frac{\pi^2 EI}{(kL_u)^2}$$

6. Compute moment magnification factor:

$$\delta = \frac{1}{1 - \frac{2.5P}{P_c}}$$

where P is the maximum axial load acting on the column.

7. Magnify P and M by mutiplying each by δ.

8. Compute P / f'_c and $M / h A_g$ to use the appropiate interaction diagram prepared by the American Concrete Institute (ACI) according to the strength of the concrete used, to the reinforcing steel to be used, and to:

$$\gamma = \frac{h - 2c}{h}$$

where c is the cover given to the reinforcing steel. The cover was always taken such that $\gamma =$ 0.90. Figure D.1 shows an example of an interaction diagram for a concrete of strength of 4000 psi (28 MPa), reinforcing steel with a yield strength of 60 ksi, and $\gamma = 0.90$.

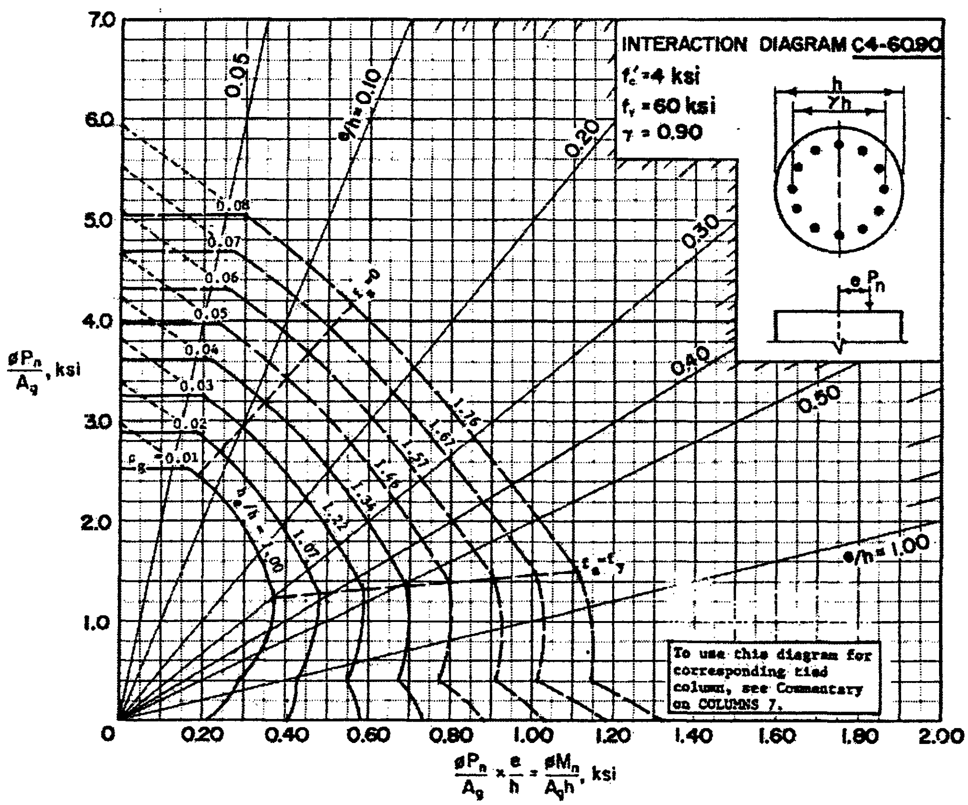


Figure D.1: Example of column interaction diagram. (ACI, 1990)

Appendix E

Spread Footing Design

After analyzing the models using the program SAP2000 (as presented in Chapter 5), the loads at the bottom of the columns were used to design rectangular spread footings for the structure. What makes difficult this design is that the maximum allowable pressure on the soil will depend on the size of the foundation, but so will the pressure acting on the soil. Also adding to the difficulty is that there is a situation when there are biaxial moments acting on the footing. This occurs when the seismic loads are applied in the transversal direction of the bridge. The foundation design has to work for earthquakes in both the longitudinal, and the transversal direction.

Following is a list of steps taken in this research to estimate the dimensions of spread footings. It was assumed that the soil had and angle of internal friction of $\phi = 40^o$, a cohesion of $c = 0$, and a unit weight of $\gamma = 18.85$ kN/m^3. With the value of ϕ, values of bearing capacity factors (Nc, $N\gamma$, Nq) can be obtained from Table 4.4.7.1A of the Standard Specification of Highway Bridges (AASHTO, 1996).

1. Assume tentative sizes of length (L) and width (B) for the spread footings. B is taken along the longitudinal direction of the bridge. Also assume a depth from ground level to the base of the footing (Df). (Figure E.1)

2. Checking first for maximum biaxial moments, calculate the eccentricities in each direction of the foundation: $e_L = M_B / P$ and $e_B = M_L / P$, where P is the axial load from the column acting in the foundation, and M_B and M_L are the moments about the width and the length of the foundation, respectively.

3. a) If $e_L / L > 1/6$, and $e_B / B > 1/6$, go to step 4.

   b) If $e_L / L < 0.5$, and $0 < e_B / B < 1/6$, go to step 5.

   c) If $e_L / L < 1/6$, and $0 < eB / B < 0.5$, go to step 6.

Note: Another case is when both e_L / L, and e_B / B are less than 1/6. This case never occurred in the analysis. For more details on this case, or any other case, refer to Das (1987).

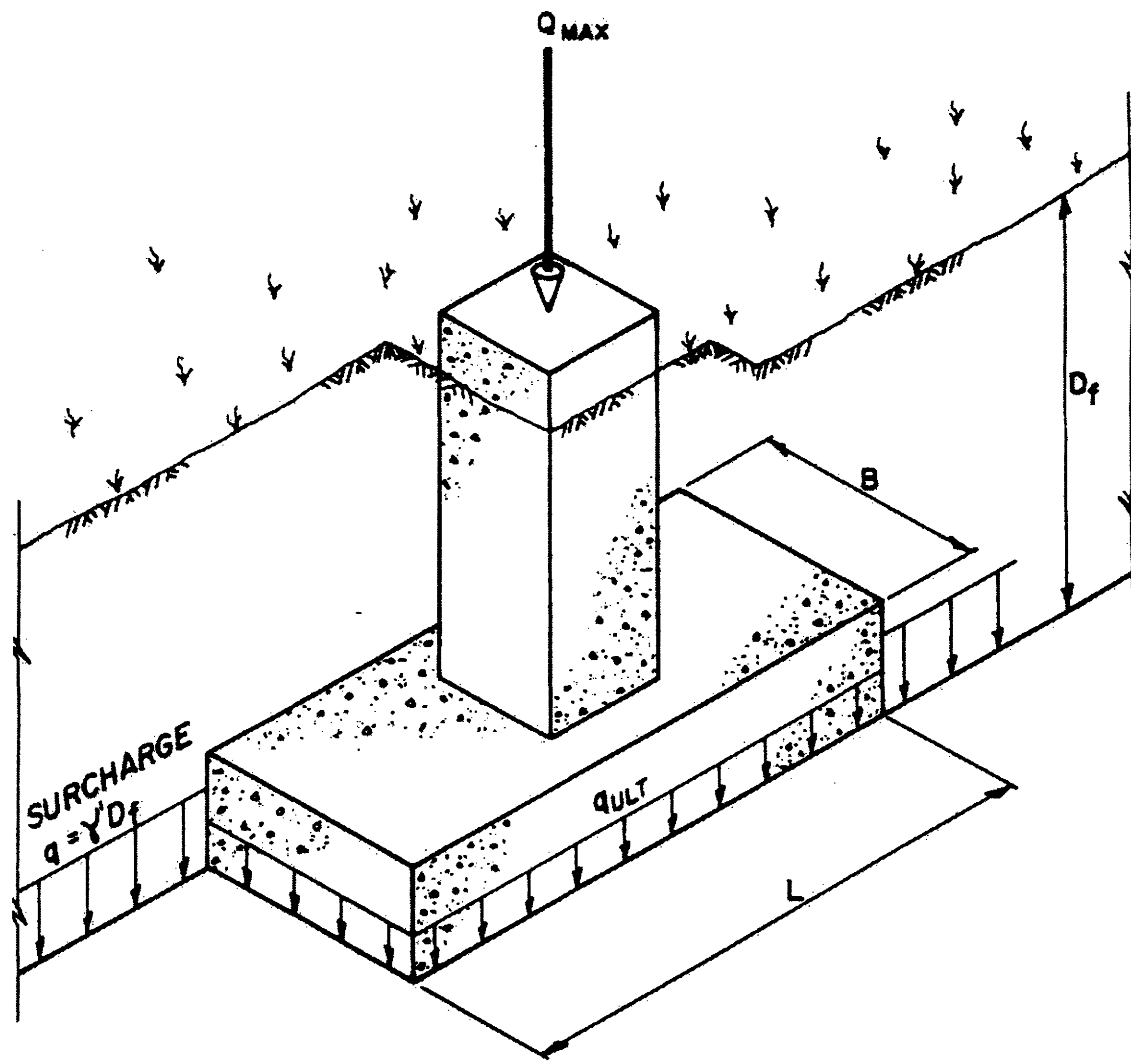


Figure E.1: Spread footing dimensions. (AASHTO, 1996)

4. This case corresponds to a pressure distribution shown in Figure E.2. Compute:

$$B_1 = B\left(1.5 - \frac{3e_B}{B}\right)$$

$$L_1 = L\left(1.5 - \frac{3e_L}{L}\right)$$

Take smaller of B_1 and L_1 is taken as B, and the larger as L for steps 8 and 9. Go to step 7.

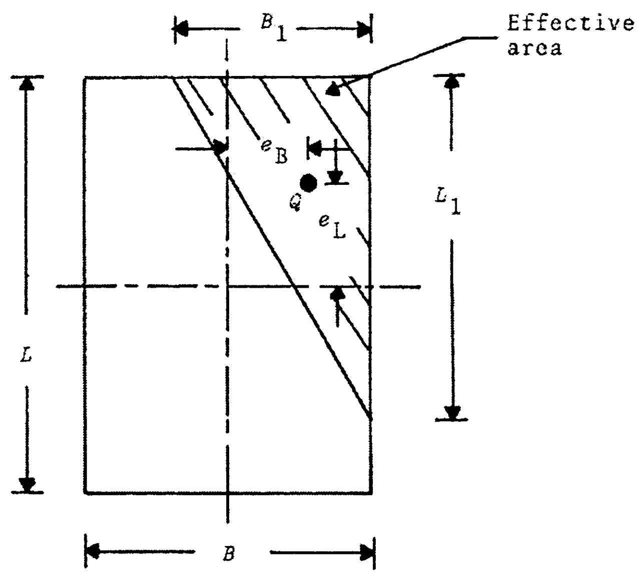


Figure E.2: Pressure distribution for e_L / L >1/6, and e_B / B > 1/6. (Das, 1987)

5. With values of e_L / L and e_B / B, obtain from Figure E.3 values of L_1 / L and L_2 / L. Compute:

$$A = \frac{1}{2}(L_1 + L_2)B$$

$$B_1 = \frac{A}{L_1}$$

where L_1 is the larger of L_1 and L_2. Take B_1 as B, and L_1 as L for steps 8 and 9. Go to step 7.

6. With values of e_L/L and e_B/B, obtain from Figure E.4 values of B_1/B and B_2/B. Compute:

$$A = \frac{1}{2}(B1 + B2)L$$

$$B1 = \frac{A}{L}$$

Take B_1 as B for step 8. Go to step 7.

7. Calculate the moment of inertia around the axis ZZ of the shaded area in Figures E.2, E.3, or E.4 depending on the case being designed for. This is the area where the pressure is acting on the soil. Referring to Figure E.5, calculate the maximum pressure acting on the soil:

$$q_{max} = \frac{\Sigma Vzb}{Izz}$$

where ΣV is equal to the axial load P.

8. Compute footing shape factors:

$$sc = 1\left(\frac{B}{L}\right)\left(\frac{Nq}{Nc}\right)$$

$$sq = 1 + \frac{B}{L}\tan\phi$$

$$s\gamma = 1 - 0.4\left(\frac{B}{L}\right)$$

9. Compute ultimate soil pressure:

$$q_{ult} = s_c c N_c + 0.5 \gamma s_\gamma B N_\gamma + \gamma D_f s_q N_q$$

10. Compute maximum allowable pressure:

$$q_{all} = \frac{q_{ult}}{FS}$$

where FS is a factor of security. In this research it was taken as 1.5.

11. Check that q_{max} is less than q_{all}. If it is not, then return to step 1.

12. Check the foundation size works for uniaxial bending, computing:

$$B_1 = B - 2e_B$$

$$A = B_1 L$$

Use B_1 as B in step 14.

13. Compute maximum pressure acting on the soil:

$$q_{all} = \frac{2P}{A}$$

14. Compute ultimate soil pressure:

$$q_{ult} = s_c c N_c + 0.5 \gamma s_\gamma B N_\gamma + \gamma D_f s_q N_q$$

15. Compute maximum allowable pressure:

$$q_{all} = \frac{q_{ult}}{FS}$$

16. Check that q_{max} is less than q_{all}. If it is not, then return to step 1.

17. Once in this step, the foundation works for both situations of earthquake in the longitudinal and the transversal direction. It can be returned to step 1 to try to reduce the size of the foundation, otherwise, the design is finish.

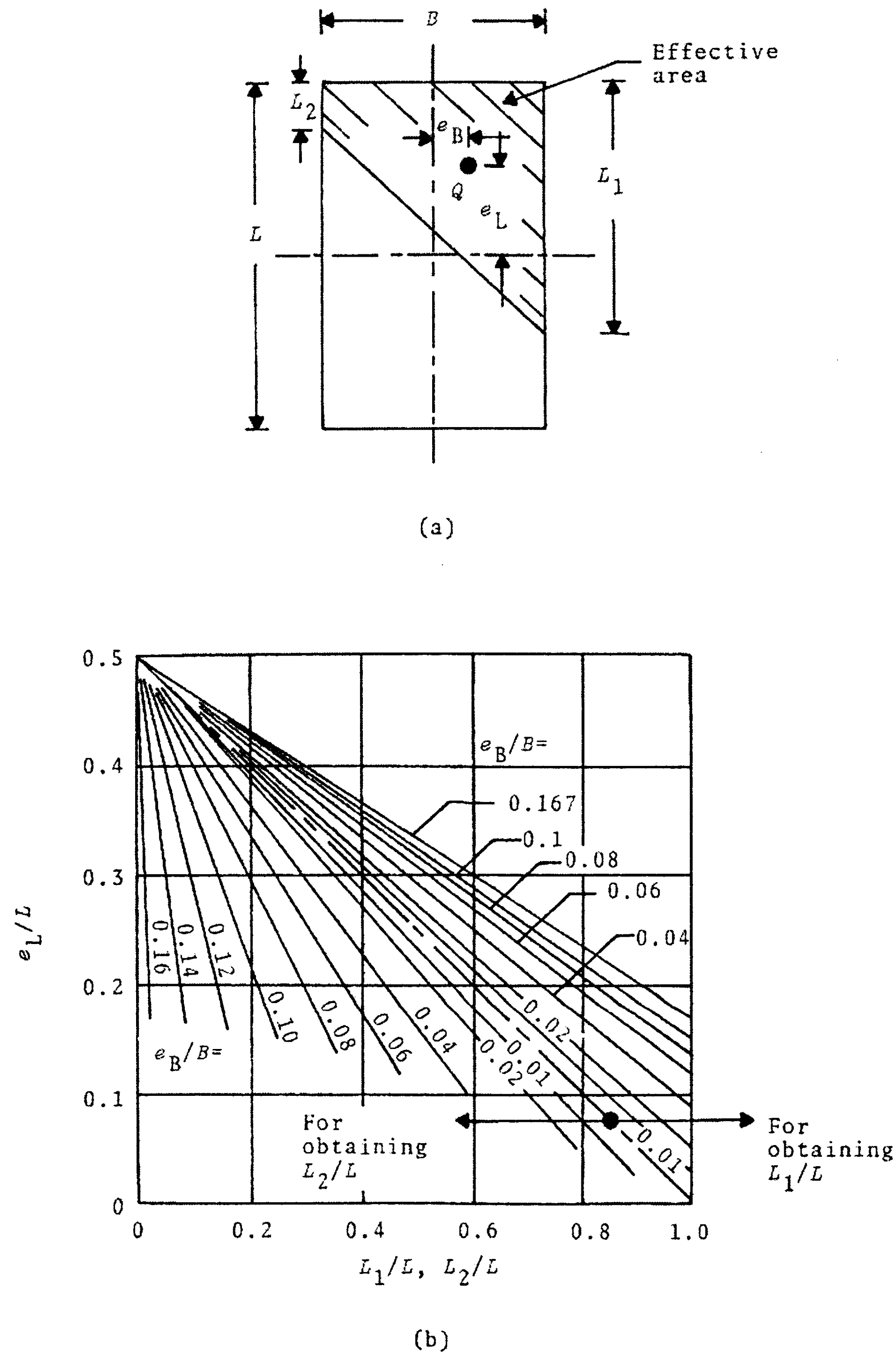
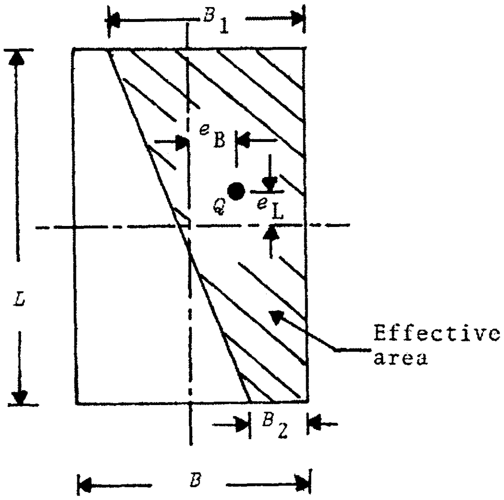
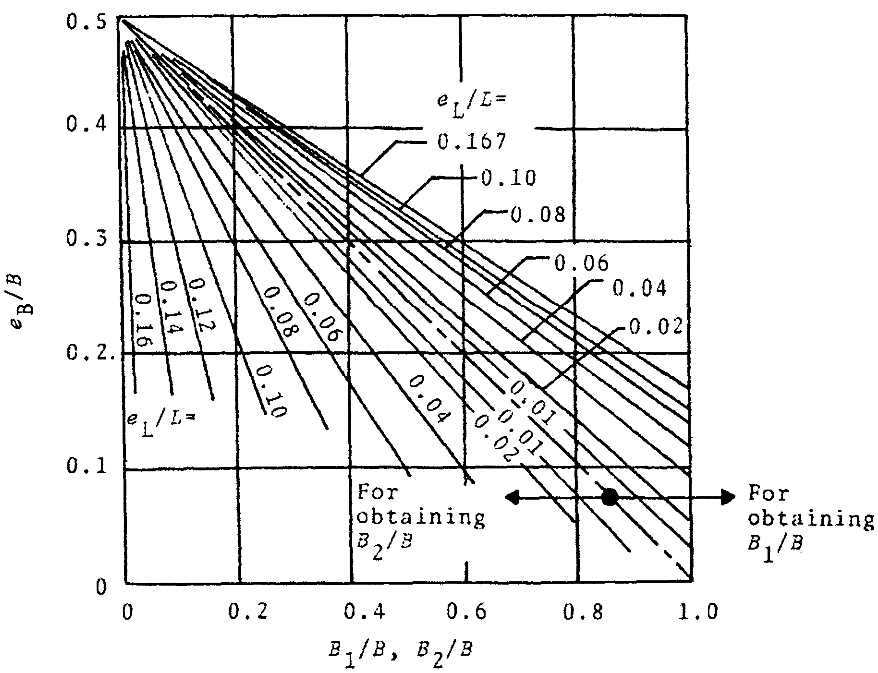


Figure E.3: Pressure distribution for $e_L / L < 0.5$, and $0 < e_B /B < 1/6$. (Das, 1987)



(a)



(b)

Figure E.4: Pressure distribution for $e_L / L < 1/6$, and $0 < e_B / B < 0.5$. (Das, 1987)

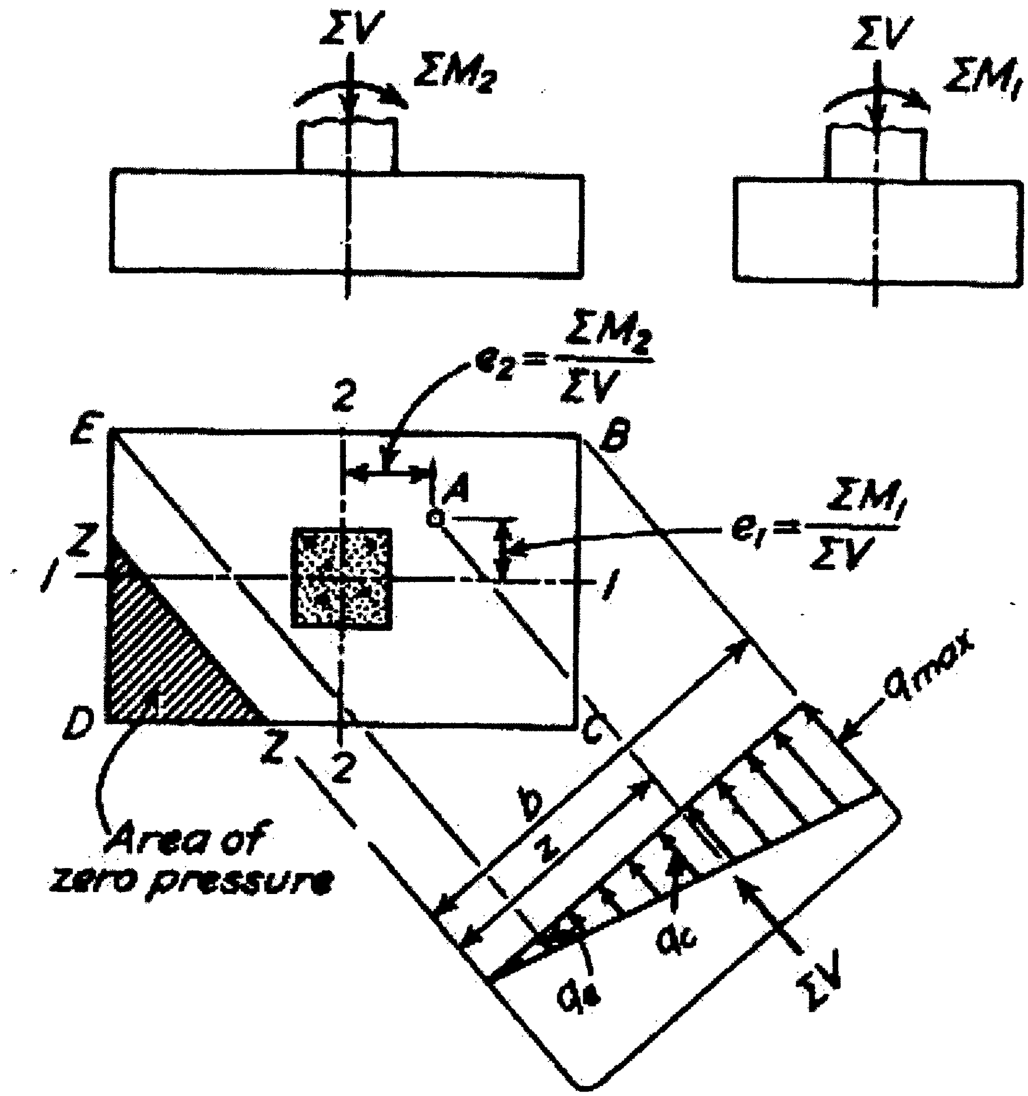
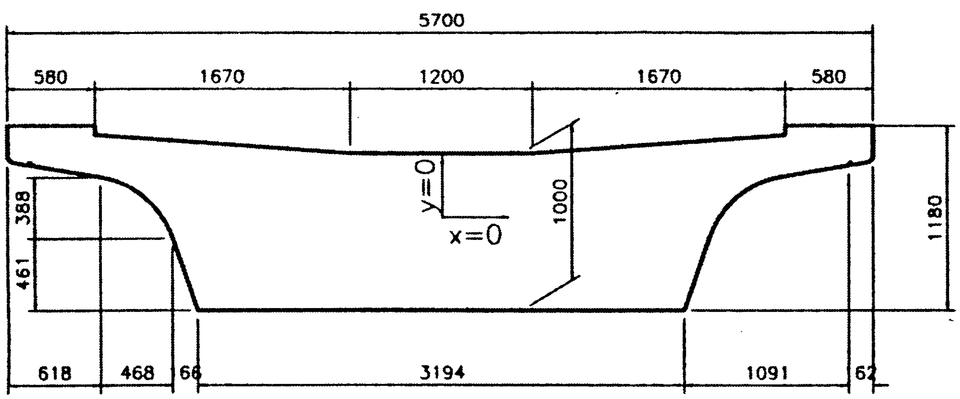


Figure E.5: Determination of maximum pressure. (Peck, Hanson, and Thornburn, 1973)

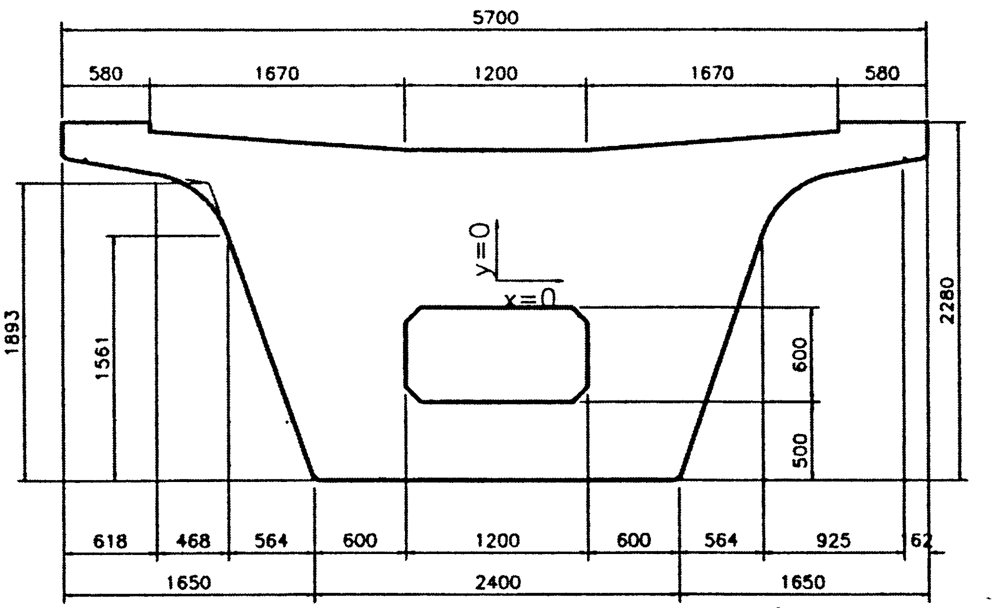
Appendix F

Precast Segments Cross-Sections



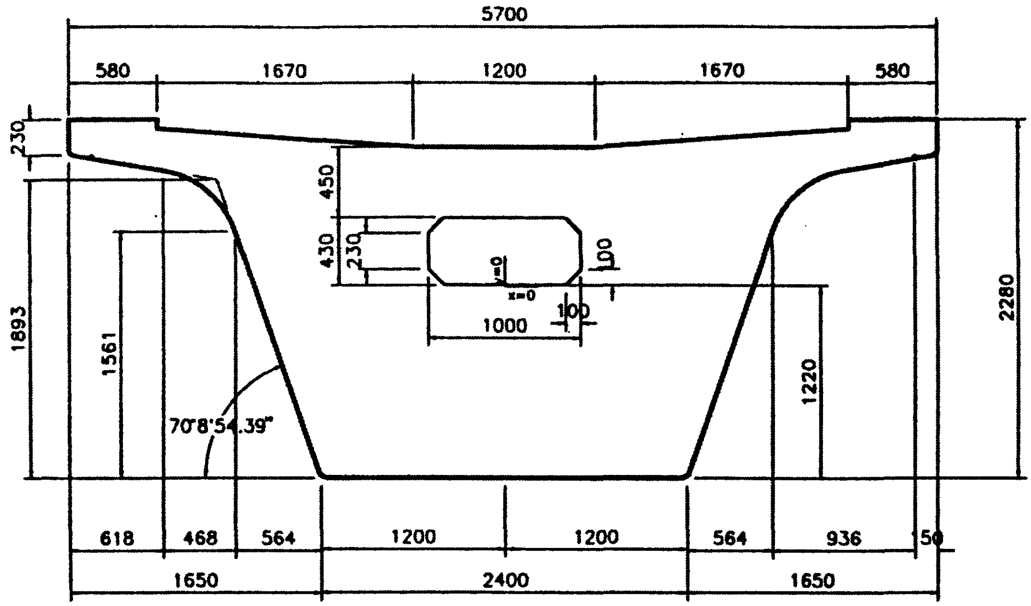
Area: 4.237371 sq m
Perimeter: 13.01848 m
Bounding box: X: -2.85 -- 2.85 m
Y: -0.5944844 -- 0.5855156 m
Centroid: X: 0 m
Y: 0 m
Moments of inertia: X: 0.4356741 sq m sq m
Y: 7.161368 sq m sq m
Product of inertia: XY: -1.997796e-14 sq m sq m
Radii of gyration: X: 0.320651 m
Y: 1.300019 m
Principal moments(sq m sq m) and X-Y directions about centroid:
I: 0.4356741 about [1 0]
J: 7.161368 about [0 1]

Figure F.1: Expansion segment, cross-section one. (Euro Estudios, 1997)



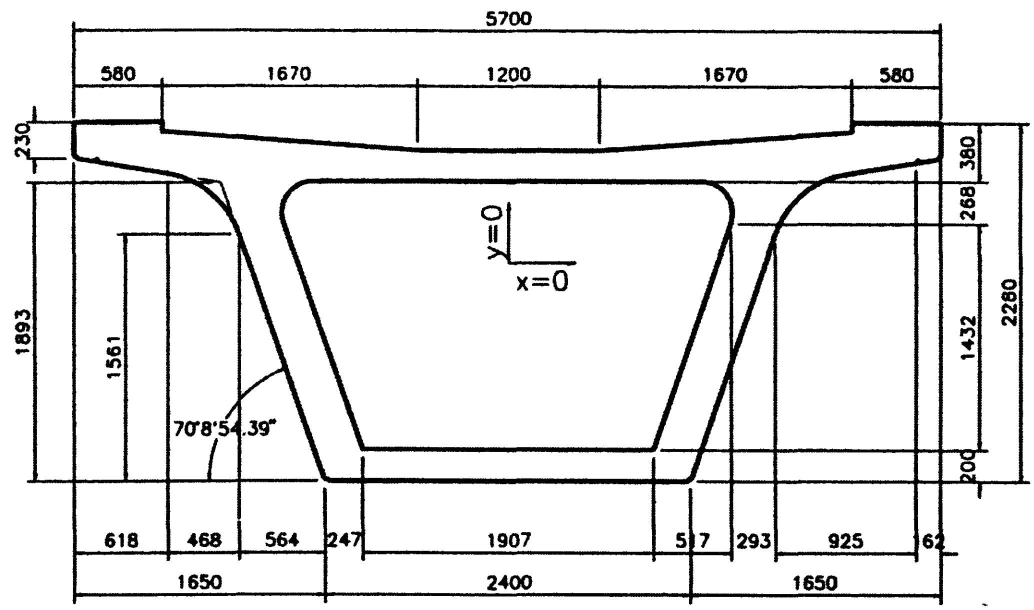
Area: 6.613641 sq m
Perimeter: 17.91072 m
Bounding box: X: −2.85 -- 2.85 m
Y: −1.268971 -- 1.011029 m
Centroid: X: 0 m
Y: 0 m
Moments of inertia: X: 2.813415 sq m sq m
Y: 9.127117 sq m sq m
Product of inertia: XY: −3.953726e−14 sq m sq m
Radii of gyration: X: 0.6522238 m
Y: 1.174753 m
Principal moments(sq m sq m) and X−Y directions about centroid:
I: 2.813415 about [1 0]
J: 9.127117 about [0 1]

Figure F.2: Expansion segment, cross-section two. (Euro Estudios, 1997)



Area: 6.903641 sq m
Perimeter: 17.17072 m
Bounding box: X: −2.85 -- 2.85 m
Y: −1.211559 -- 1.068441 m
Centroid: X: 0 m
Y: 0 m
Moments of inertia: X: 2.947524 sq m sq m
Y: 9.175617 sq m sq m
Product of inertia: XY: 5.897599e−15 sq m sq m
Radii of gyration: X: 0.6534157 m
Y: 1.152865 m
Principal moments(sq m sq m) and X−Y directions about centr(
I: 2.947524 about [1 0]
J: 9.175617 about [0 1]

Figure F.3: Pier segment. (Euro Estudios, 1997)



Area: 3.06642 sq m
Perimeter: 22.82792 m
Bounding box: X: −2.85 -- 2.85 m
Y: −1.378351 -- 0.9016492 m
Centroid: X: 0. m
Y: 0 m
Moments of inertia: X: 1.855144 sq m sq m
Y: 6.886054 sq m sq m
Product of inertia: XY: 8.511437e−14 sq m sq m
Radii of gyration: X: 0.777809 m
Y: 1.498544 m
Principal moments(sq m sq m) and X−Y directions about centroid:
I: 1.855144 about [1 0]
J: 6.886054 about [0 1]

Figure F.4: Typical segment. (Euro Estudios, 1997)

Appendix G

Stresses in the Rail

The rail to be used in the Tren Urbano Project is a 115 RE section. It has the following properties:

moment of inertia	=	2730 cm^4 (65.6 in^4)
height	=	16.83 cm (6-5/8 in)
distance from base to neutral axis	=	7.57 cm (2.98 in)
area	=	72.57 cm^2 (11.25 in^2)

Using the program SAP2000, the stresses in the rail due to both thermal effects in the rail and due to the superstructure/rail interaction can be obtained. To these stresses, the effects of the load of the rail vehicle must be added. To obtain these stresses, first, the bending moment (M) due to the wheel load of the vehicle is calculated:

$$M = P\left(\frac{EI}{64u}\right)^{1/4}$$

where:

P = one half of the axle load (73 kN);

E = modulus of elasticity of the rail (2.1 x 10^8 kPa);

I = moment of inertia of the rail;

u = modulus of track elasticity.

The value of u was assumed to be 17,238 kPa (2500 psi).

After computing M, the stress due to bending (S) in the rail can be checked with the equation:

$$S = \frac{Mc}{I}$$

where c can be taken as either the distance from the neutral axis to the bottom or to the top of the rail. These stresses are added to the ones obtained using SAP2000 to compute the total stress.

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