Structural Evaluation of Tied-Arch and Truss Footbridges through a Case Study

Mani Seyed Imani

Submitted to the
Institute of Graduate Studies and Research
in partial fulfillment of the requirements for the Degree of

Master of Science
in
Civil Engineering

Eastern Mediterranean University
September 2014
Gazimağusa, North Cyprus

Approval of the Institute of Graduate Studies and Research

Prof. Dr. Elvan Yılmaz
Director

I certify that this thesis satisfies the requirements as a thesis for the degree of Master of Science in Civil Engineering

Prof. Dr. Özgür Eren
Chair, Department of Civil Engineering

We certify that we have read this thesis and that in our opinion it is fully adequate in scope and quality as a thesis for the degree of Master of Science in Civil Engineering.

Asst. Prof. Dr. Mürüde Çelikağ
Supervisor

Examining Committee

1. Asst. Prof. Dr. Mürüde Çelikağ
2. Asst. Prof. Dr. Mustafa Ergil
3. Asst. Prof. Dr. Serhan Şensoy

ABSTRACT

Footbridges are usually slender structures with comparatively light load bearing requirements. These bridges are constructed over busy roads or other obstacles to provide a safe and easy passage for pedestrians and improve access. Footbridges are often less costly when compared to other bridges and structures. However, their aesthetics, appearance and practicability are also of high importance. The structure's slenderness offer opportunities for engineering innovation, but these characteristics make designers pay more attention to issues, such as, wind, impact and collision loads.

The main objective of this thesis is to propose some design alternatives for a footbridge crossing over Nicosia-Famagusta main road between the North and South Campuses of Eastern Mediterranean University. In this regard, two types of footbridges, tied-arch and truss bridges, with seven alternatives including the original bridge are investigated from four viewpoints; structural behavior, material usage, cost and aesthetics. Design and loading are according to AASHTO guidelines. Modeling and analyses of the structures are carried out by using the general purpose analyses and design program SAP2000, version 16.0.0 Ultimate. The footbridges studied in this research are to be constructed over a busy road, thus, the vulnerability of each alternative to impact and collision loads were investigated. For this purpose, progressive collapse analysis is carried out to study the behavior of design alternatives in case of damage to pier columns. The results showed that single span bridges have higher performances, and among them, arch type ones represent less deflection and lower compressive stress.

In order to determine the bridge with the highest performance each of the investigated characteristics (structural behavior, material usage, cost and aesthetics) were first of all individually compared and then they were compared with each other. The results revealed that single span arch bridge that is designed to be constructed by using simple sections appear to be the best and most appropriate alternative if all parameters receive equal importance weights. However, assigning importance weights to the structural behavior, material usage, and cost, which are as three times as the one allotted to the aesthetics, resulted in selection of single span truss bridge as the most suitable option.

Keywords: Footbridges, pedestrian bridges, structural behavior, progressive collapse analysis.

ÖZ

Üst geçitler diğer yapılara göre genelde daha narin yapılar olduğundan yük taşıyıcı sistemleri de hafif olur. Bu tür köprüler yoğun trafik olan yollarda ve yayaların geçişine engel oluşturan durumlarda yol ve engellerin üzerine inşa edilir ve yayaların güvenli ve rahat bir şekilde geçişini sağlar. Üst geçitler diğer köprü ve yapılara göre daha az maliyetli yapılardır. Fakat estetik görünümleri ve pratik olmaları büyük önem taşır. Yapının narin oluşu mühendislikte yaratıcılığa fırsat verirken bu özellik tasarımcının rüzgar, darbe ve çarpışma yüklerine de daha çok dikkat etmesini gerektirir.

Bu tezin ana hedefi Doğu Akdeniz Üniversitesi kuzey ve güney yerleşkesi arasında kalan Lefkoşa-Mağusa ana yolunun üzerinden geçecek bir üst geçit için alternatifli tasarım üretmektir. Bu bağlamda iki tip üst geçit köprü tasarımı, bağlı-kemer ve makas köprü, mevcut üst geçit dahil yedi alternatif köprü olarak yapısal davranış, malzeme kullanımı, maliyet ve estetik görünüm açısından incelenmiştir. Tasarım ve yükleme AASHTO standardına göre yapılmıştır. Sözkonusu yedi alternatif üstgeçit köprüsünün modelleme ve yapısal analizi genel analiz ve tasarım programı SAP2000 Ultimate, 16. Versiyon kullanılarak yapılmıştır. Bu araştırma kapsamında incelenen üst geçitler yoğun trafik olan bir yol üzerine inşaa edilecektir, dolayısıyla her alternatif tasarımın, darbe ve çarpışma yüklerine karşı güvenirliği de çek edilmiştir. Bu nedenle üst geçit kolonlarında oluşabilecek bir hasar durumunda bahsekonu alternatif üstgeçit köprülerinin yapısal davranışı kademeli çökme analizi kullanılarak incelenmiştir. Elde edilen sonuçlara göre tek açıklıklı köprüler daha iyi performans

göstermiş ve alternatifler arasında kemer tipi köprüler daha az sehim ve daha düşük basma gerilmesi elde etmişlerdir.

Yapılan analizler sonucunda en yüksek performansı elde eden köprüyü bulmak için yapısal davranış, malzeme kullanımı, maliyet ve estetik görünüm özellikleri her bir köprü için önce ayrı ayrı karşılaştırılmış ve sonrasında da biribiriyle karşılaştırılmıştır. Karşılaştırmalar sonucunda yukarıda belirtilen dört özelliğin eşit önem ağırlığı alması durumunda tek açıklıklı, basit kesitlerle yapılmış kemer köprü en iyi performansı vermiş ve en uygun alternatif olmuştur. Diğer yandan yapısal davranış, malzeme kullanımı, maliyet özelliklerinin önem ağırlığının estetik görünüm özelliğinin üç katı olması durumunda tek açıklıklı, makas köprü en uygun alternatif olmuştur.

Anahtar kelimeler: Üsgeçitler, yaya köprüleri,yapısal davranış, kademeli çökme analizi

To My Parents

ACKNOWLEDGEMENT

Foremost, I would like to thank my supervisor Asst. Prof. Dr. Mürüde Çelikağ for her inspiration and commitment to offer me the unique opportunity to obtain a master's degree and her guidance throughout this study. Without her support, it would not have been possible to complete this thesis.

I would also like to express a heartfelt gratitude and love to my family for their support and genuine interest in my work.

Finally, special thanks for valuable help to my friend and colleague, Shiva Faeghi.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

Chapter 1

INTRODUCTION

A bridge is a structure which is built over obstacles such as, rivers, roads, valleys, or streams, for the purpose of carrying loads like highway traffic or pedestrians. Pedestrian bridges demand high aesthetical consideration. Pedestrian bridges or footbridges should be light but at the same time ensuring safety. Moreover, they should be comfortable, designed according to human scale and their appearance should be inviting to encourage pedestrians to use it (Str′ask′y, 2005).

What is generally accepted by architects and engineers is that all structural members of the bridge should transfer the internal forces through the structural system, while it is important for a bridge to be integrated into social surrounding and environment (Str′ask′y, 2005). Thus, it is important for every bridge engineer to design bridges that provide safety, durability and serviceability to the public, while contributing to the urban beauty. To accomplish this task a very good understanding of behavior and a good knowledge of parameters that affect structural response is required. Therefore, the bridge should be analyzed and designed to ensure that it meets the design standards. The design is also required to meet an acceptable deflection to ensure that bridge is secure to use. Footbridges may be subjected to sudden loadings due to human traffic which can cause vibrations on the deck and consequently, cause discomfort to people and could be unsafe. Moreover, these vibrations may progress into lateral torsional buckling (LTB) of the bridge deck, which is one of the important design checks in footbridge design process.

In addition to structural considerations, another important issue is the material used for a bridge construction. The efficient use of material is important for every construction project. It contributes to the cost reduction which is itself an important parameter in successful completion of projects. Furthermore, efficient use of material can be considered as an aesthetic criterion; considering aesthetics is high demand for footbridges.

1.1 Aim and objectives

This study aims at designing two types of footbridges for pedestrian crossing over the Nicosia-Famagusta main road, between North and South Campus of Eastern Mediterranean University. The existing footbridge and the proposed second footbridge will be investigated and compared with the ones designed within the scope of this project. The following are the summary of the work plan:

1. Modeling of Tied-Arch Footbridge
2. Modeling of Truss Footbridge
3. Analysis and design of the footbridges using SAP2000. Dead, imposed, wind and earthquake loading was used.
4. Cost of construction were calculated for the new bridges
5. Comparison of the newly designed bridges with the existing and the proposed footbridges were carried out and their cost of construction was also analyzed.

1.2 Limitations

In this research behavior of footbridges with dead loads, live loads, wind loads and earthquake loads are studied. These loadings are very important for the design process of every footbridge. Other loads such as dynamic live load, fatigue load, temperature loads etc. are considered to be out of the scope of the thesis (It should be noted that according to the considerations of AASHTO guidelines there is no need to take the dynamic live load into account).

Only the following design guidelines are used for modeling, analysis and design of the bridges:

- LRFD Guide Specifications for the Design of Pedestrian Bridges. AASHTO.
- AASHTO LRFD Bridge Design Specifications, customary US units.

Because of the lack of reliable information regarding the geotechnical data of the region where the footbridge is planning to be build, wind speed and earthquake input parameters, such as, spectral acceleration coefficients, there was a need to make appropriate assumptions.

1.3 Method

The work in this thesis is divided into two parts. The first part consists of a literature review to study the existing literature on this subject to help in formulating the details of this research and increase the knowledge before the case studies are investigated. The second part included case studies and the results.

In the literature review on pedestrian bridges, particularly truss and arch bridges, were carried out. Previous studies with similar research content were reviewed. Finally, a brief introduction into structural analysis, finite elements method and the SAP2000 software which is used in this study is provided.

The chapter on case study investigates 5 bridges: the reference (existing) bridge, two types of arch bridges (single span and double span) and two types of truss bridges (single span and double span). The bridges are first modeled with the general purpose analysis and design software SAP2000, subjected to loads based on the AASHTO design guideline for pedestrian bridges, and then their structural behavior were analyzed. All alternatives were also investigated with respect to cost, used material, and aesthetics. Finally, the results were compared to find out the most appropriate option to be applied in the future design of pedestrian bridges in similar conditions.

1.4 Thesis outline

The outline of the chapters in thesis is as follows:

Chapter 2 consists of the literature review, an introduction to the pedestrian and footbridges and the bridge types that were investigated in this thesis. It continues with review of the previous studies in the field of pedestrian bridge design. Chapter 3 describes the case study and the design alternatives. The method of investigation and modeling is also described in this chapter. Chapter 4 provides the results and the output of the comparison. In this chapter the important results were also discussed. Chapter 5 presents the conclusions drawn from the results of the analysis of footbridges from this research work.

Chapter 2

LITERATURE REVIEW

In this chapter, pedestrian bridges, the existing one and two types of truss and arch bridges which are investigated in this study are described. A literature review on the available published material is provided and a brief introduction into structural analysis, finite element modeling and SAP2000 software is presented.

2.1 Footbridges

When there are obstacles, for example, roads, rivers and valleys, a footbridge or a pedestrian bridge can make a connection between adjacent lands and offer a safe overpass. The location and design of a footbridge should provide safety, easy use, inviting connection, while reducing travel time. Recent advances in materials and construction technology have encouraged architects and engineers to move towards structures with longer spans and slender appearances. This approach may need more investigation into structural behavior of such bridges and also budgetary considerations. In addition to structural and economic concerns, there are some other important issues regarding footbridges. New Zealand Transport Agency (NZTA) considers the following issues as the principles of design of pedestrian bridges:

1) Location: During the design of a footbridge natural topography should be considered and the location should ensure maximum use of the bridge.
2) Accessibility: Bridge accessibility for all pedestrians is important. In some cases, ramps may be required for mobility impaired people.

3) Integration: Bridges should be integrated into the urban context and surrounding environment.
4) Landmark design: Bridges are usually prominent structures, thus, they can offer opportunities to create new landmarks and incorporate into the cultural and historic values of the area in which they are constructed.
5) Experience: As pedestrians may spend more time passing over a footbridge, it may offer interesting experiences for users.
6) Form: Pedestrian bridges are light structures as they do not carry vehicle loads. This feature allows more flexibility in form and material choice.
7) Approaches: Approach ramps and stairs are parts of the bridge composition and should be in harmony with the land form and landscape.
8) Safety: The safety of pedestrians is an important issue in the bridge location and design.
9) Lighting: Lighting is important to ensure pedestrians' safety as most of footbridges may be used at night.
10) Maintenance: Selection of durable materials and finishes that do not need significant maintenance over time.
11) Color: Color is important especially in a rural area, as it can attract attention.

The above-mentioned issues are issues that should be considered in early stages of design.

2.2 Bridge types

There are different criteria that bridge can be designed based on. The criteria that are considered in this thesis are as follows:

1. Structural behavior
2. Economy (cost)

3. The amount of used material
4. Aesthetics

In this thesis two general types of bridges, arch and truss bridges, are investigated as alternatives for the project and this selection is based on the aforementioned criteria.

2.3 Arch Bridges

The need for crossing natural obstacles and streams had always existed even in early times. For this reason, arches have been used as structural elements throughout the ages. Ancient arch bridges were built with stone elements, but nowadays they are generally constructed of concrete, steel, wood, masonry or composite (Wai-Fah and Lian, 2000).

"As a structural unit an arch is defined as a member shaped and supported in such a manner that intermediate transverse loads are transmitted to the supports primarily by axial compressive thrusts in the arch" (Xanthakos, 1994). For a given loading, arch shape must be in such a way that it should avoid bending moments (Xanthakos, 1994, Wai-fah and Lian, 2000). Figure 2.1 provides a schematic description of different elements of an arch bridge.

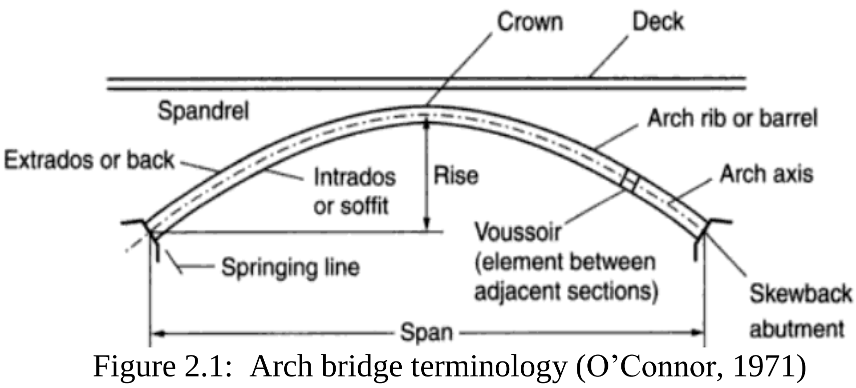


Figure 2.1: Arch bridge terminology (O'Connor, 1971)

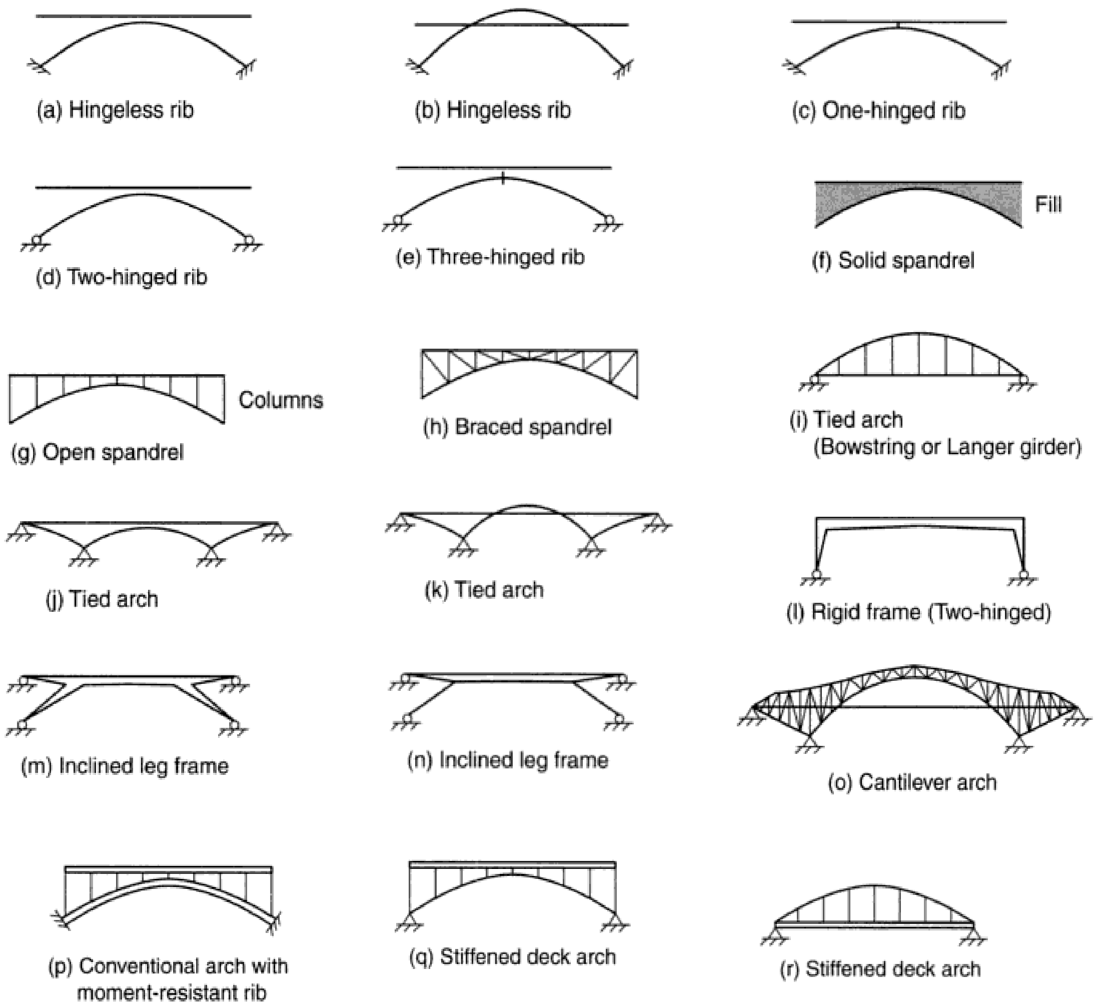


Figure 2.2: Types of Arch bridges (O'Connor, 1971)

Figure 2.2 illustrates possible forms of fixed and hinged arch bridges. The arch can be fixed or hinged. In a fixed arch there is no possibility of rotation at supports, and this causes three degrees of indeterminacy. In the hinged type, one to three hinges can be connected to the arch rib. Introduction of hinge to a fixed arch reduces the indeterminacy. A two-hinged arch has one degree of indeterminacy and the three-hinged arch is determinate and free of the problems of secondary stresses (Wai-Fah and Lian, 2000, Ryall et al., 2000).

2.4 Advantages of arch bridges

A notable advantage of arch bridges is their beauty. Undoubtedly, arch bridges are functional and a pleasure for the users (Wai-Fah and Lian, 2000; Proske, 2009). There are some other advantages in addition to the beauty of arch bridges (Wai-Fah and Lian, 2000; Wai-Fah and Lian, 2014; Proske, 2009)

1- Many kinds of materials, such as timber, masonry, concrete, metal, composite and so on can be used to build and arch bridge;
2- It is required to construct the tie girder before the arch ribs can function;
3- The total strains are often in cyclic pressure load region;
4- Insensitivity to unplanned impacts and high robustness;
5- A high tolerance with respect to damage;
6- Early indication of malfunctioning;
7- Outstanding integration with the landscape.

2.5 Truss bridges

Fundamentally, a truss is a structure with straight and slender members which are assembled in a triangulated way and joined together with their ends. In a typical truss, the central axes of all members are concurrent at the nodes. The external forces are generally applied at the nodes and thus, applied loads are resisted primarily by axial forces induced in the truss members (Xanthakos, 1994).

The main characteristic of a truss bridge is the presence of many bracing and wind carrying members in addition to those members that can be seen in front elevation (Wai-Fah and Lian, 2000). Figure 2.3 illustrates typical members of a simple single span through-truss. The lateral members resist wind loads and provide bracings for the compression chords. Sway frames square the truss and increase the torsional rigidity. Uneven vertical loads and wind loads induce torsional loads which are carried by end portals into bearings (Wai-Fah and Lian, 2000).

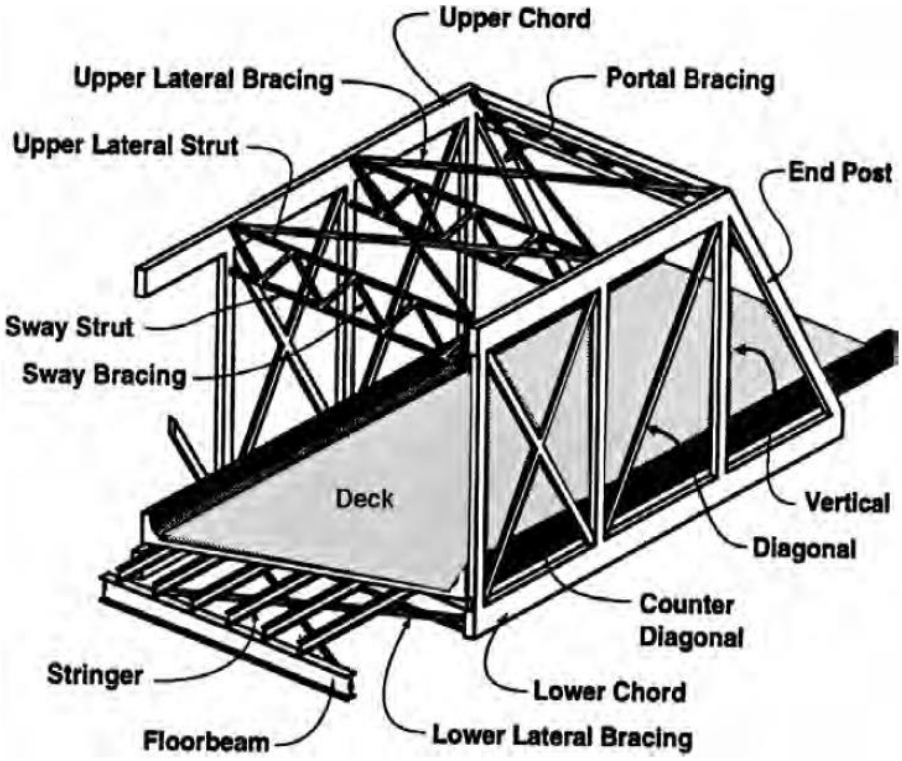


Figure 2.3: Typical Truss members (Hartel et al., 1990)

A truss could be simple span or continuous with vertical or inclined members at both ends. Based on how they carry the load, truss bridges could have different types, such as, deck truss which is built below load, through truss which passes the load between its trusses under an overhead bracing system, half-trough truss which is shallow in depth and do not have an overhead bracing system (Troitsky, 1994). Figure 2.4 shows most common types of trusses.

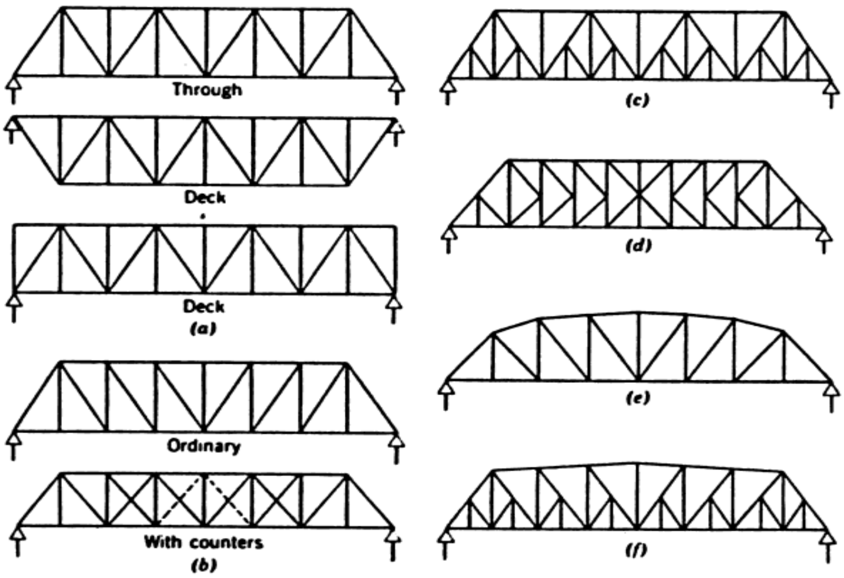


Figure 2.4: Types of truss bridges (Xanthakos, 1994)

2.6 Advantages of truss bridges

A truss bridge has two major structural advantages (Xanthakos, 1994):

1) The primary forces of members are axial loads;
2) The open-web system provides a greater overall depth than in an equivalent solid-web girder.

Moreover (Xanthakos, 1994; Fu, 2013):

1) It has a favorable aerodynamic response;
2) Its relative stiffness is an erection advantage;
3) Its self-weight is remarkably reduced compared with beams of the same span length;
4) The spans can be longer than beams due to relatively lighter weight;
5) Its construction is relatively easier than arches.

2.7 Literature Review

In this section a review of the previous research efforts towards comparing bridge designs is provided. It should be noted that there are limited number of reported research which have done similar work to the one presented in this research.

Malekly et al. (2010) proposed a methodology to evaluate the conceptual design of bridge based on some conflicting criteria. For this purpose, they developed a systematic decision process for choosing the best design alternative by means of an “integrated optimization based methodology”. Their criteria were divided into two main categories: 1) construction including cost, time, availability, and quality; and 2) superstructure including design complexity, speed of construction, durability, environment, aesthetics, construction complexity and geometric design. Their method was an integration of Quality Function Deployment and Technique for Order Performance by Similarity to the ideal solution (TOPSIS).

Welch et al. (2012) presented a conceptual design of a pedestrian bridge located in the south of Indiana-Purdue University Fort Wayne (campus) considering four potential bridge concepts. They have modeled, analyzed and provided design details for the selected arch-type pedestrian bridge. Their analysis was done by using SAP2000 for static dead, live and wind loads according to AASHTO specifications and INDOT (Indiana Department of Transportation) requirements.

In addition to the above research efforts, there are many others that have developed optimization algorithms to optimize the bridge designs or select the optimal options; like Cheng (2010) (genetic algorithm integrated with finite element method), Martí and González-Vidosa (2010) (heuristic optimization and based on the simulated annealing and threshold accepting algorithms), Martí et al. (2013) (genetic algorithm). In the meanwhile, there are also many researchers that have conducted analytical methods to assess bridges from different perspectives, such as Bayraktar et al. (2009) and Sandovič and Jouzapatis (2012) (structural behavior), Lewis (2012) (material requirement), Chen et al. (2014) (structural performance).

2.8 Definition of structural analysis

Structural engineering can be defined as the science of planning, designing and construction of safe and economical structures in a way to serve their intended purpose. Structural analysis is the main part of any structural engineering project. Its task is to predict the performance of the suggested structure (Kassimali, 2009). In other words, structural analysis is a method of engineering design which examines the design to make sure it is safe and serviceable.

From a theoretical point of view, the main purpose of structural analysis is to calculate deformations, internal forces and stresses that detained in structure due to the loads applied to it. In the field of civil engineering, different methods are used to analyze structures, such as analytical method and finite elements. In this section, finite element method is briefly introduced.

2.8.1 Finite element in structural analysis

Finite element method is a major tool for computational mechanics. Finite Element Method, firstly used by R. W. Clough in 1960 and it has already been one of the most powerful numerical techniques for solving various problems in different fields such as mechanics, physics and engineering computation problems (Long et al., 2009)

Finite element method pursues the aim of solving a complicated problem by replacing it with small and simple ones. This process makes the solution approximate rather than exact (Rao, 2010).

2.8.2 General definition of the finite element method

Finite element method divides a continuum or whole domain into a collection of subdivisions called finite elements. In this method, some nodes interconnect these elements at some determined joints. Generally, these nodes are located on the element boundaries, where there is a connection with adjacent elements. The actual variations of the field variables, such as, displacement, stress, temperature, etc. are not known. For this reason, it is assumed that these variations can be estimated by using a simple function. These functions, which are called interpolation models, are in terms of the values of the field variables at the nodes. After definition of all field equations for the whole domain, it is needed to find the nodal values for the field variable. By solving the finite element equations, the nodal values of the field variable are obtainable. Having all of these known, the field variables in whole domain can be defined by interpolation models. The solution of a general problem by the finite element method follows a step-by-step process. For instance, for static structural problems, the procedure can be expressed as follows (Rao, 2010):

1) Divide structure into discrete elements (discretization)
2) Select a proper interpolation or displacement model.
3) Derive element stiffness matrices and load vectors.
4) Assemble element equation to obtain the overall equilibrium equations.
5) Solve for the unknown nodal displacements.
6) Compute element strains and stresses.

2.8.3 SAP2000 software

SAP2000 is a commercial finite element program for structural analysis of structures. SAP2000 can be used for different structures, such as, bridges, dams, stadiums, industrial structures and buildings.

SAP2000 has a great flexibility: from simplest day-to-day 2D structural frames to complicated 3D structures can be analyzed using this software. It also offers different analysis options: linear, nonlinear, static and dynamic analysis. The design codes are integrated in this software and this feature can automatically calculate wind, bridge and seismic loads. It also offers comprehensive automatic code checks for International steel and concrete design standards (CSI, 2014).

From the above mentioned points regarding SAP2000 it can be concluded that this software is a suitable solution for the purpose of this research. The SAP2000 version 16 was used to conduct structural analysis of this study.

2.9 Progressive collapse analysis guidelines

Progressive collapse is a situation in which a localized failure of a primary structural element results in the collapse of adjoining elements, and then propagates to disproportionate collapse of the structure. ASCE 7 states "Progressive collapse is defined as the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or disproportionately large part of it." The failure may have different causes including natural or man-made ones.

The terrorist attack which took place in World Trade Center of New York in 2001 heightened the concerns regarding safety and vulnerability of buildings against similar threats. This enforced many federal agencies to provide protections for buildings. In this regard, some building security designs have been developed to address threats such as explosion and progressive collapse. Different branches of the federal government of the United States developed design standards for the protection of federal facilities, namely, General Services Administration (GSA), and the Department of Defense Security Engineering Working group (DOD-SEWG). The design guidelines developed by the above mentioned agencies (GSA and UFC) have the most complete criteria, and are widely accepted and referenced.

2.9.1 General Services Administration

The U.S. General Services Administration (GSA) has developed a guidance to provide a facility security requirement which includes the calculations for blast loads, material strength factors, glazed system response criteria, structure performance, flexure and shear response, and progressive collapse resistances.

The document "Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects" is developed to consider potential for progressive collapse in the design, planning, and construction phases of new buildings and renovation of old buildings. The first GSA document, issued in 2000, focused on reinforced concrete structures. The next version in 2003 addressed steel structures.

2.9.2 Department of Defense

The Department of Defense developed Utility Facility Criteria (UFC) established their requirements in construction of facilities. UFCs has several documents, among them, the UFC 4-023-03 is developed for "Design of Buildings to Resist Progressive Collapse" and it provides criteria for planning, design, construction, sustainment, and modernization criteria which should be applied to the Military Departments, the

Defense Agencies and the activities related to DoD. The design requirements of this document are for reducing the potential of progressive collapse.

2.9.3 Comparison of GSA and UFC

The analysis and design criteria which are used in both GSA and UFC are independent from threats. They suggest that a in a structure good ductility, continuity of reinforcement and redundancy of elements or load paths should be provided so that the internal energy could dissipate sufficiently to reduce the potential of progressive collapse of damaged structures.

UFC divides the new and existing structures into four categories: 1) Very Low Level of Protection (VLLOP), 2) Low Level of Protection (LLOP), Medium Level of Protection (MLOP), and High Level of Protection (HLOP). Analysis and design in UFC are based on these categories.

GSA adopts a different approach. The approach consists of three examination steps, and in each step different set of criteria is included to analyze the structures.

Both the GSA and UFC suggest nonlinear static and dynamic analysis for complicated structures. However, GSA takes the linear static and dynamic analyses into account due to fast results. Nevertheless, GSA limits the use of linear analyses to small structure (buildings with of fewer than ten stories).

The concept of notional removal of critical columns is considered in both the GSA and UFC. This concept considers the removal of a column when it is badly damaged so that it loses its load bearing function. Load combinations for these Guidelines are different. The dynamic factor to simulate the dynamic effects because of column removal is 2.0 for both guidelines. The load combinations for GSA and UFC are as the following:

GSA:

- Static analysis: $2(DL + 0.25LL)$
- Dynamic analysis $(DL + 0.25LL)$

UFC:

- Static analysis: $2(1.2DL + 0.5LL + 0.2W)$
- Dynamic analysis $(1.2DL + 0.5LL + 0.2W)$

where, DL, LL, W are dead, live and wind loads, respectively.

The GSA guideline states that when a Demand Capacity Ratio (Demand capacity ration is the ratio of acting force (demand) to the ultimate, unfactored capacity) exceeds 2 for structures which have irregularities and 1.5 for structures with irregularities the possibility of progressive collapse becomes high. Demand capacity ratio can be used in linear static analysis. In contrast, the UFC guideline does not determine a specific demand capacity ratio.

The load sequence when a critical column is removed is to some extent different in GSA and UFC. GSA states that initial removal of a column should be conducted before any analysis; however, UFC recommends that the analysis of the structure in undamaged conditions should be conducted under gravity load, and then, a critical column can be removed. Both guidelines specify damage limits for structures, and those of UFC are more conservative (Kim, 2006).

2.10 Progressive collapse of bridges in the literature

There is really limited number of reported research studies done in the field of progressive collapse of bridges. Additionally, there are no developed specific guidelines for progressive collapse of bridges. This makes the assessments and judgments regarding progressive collapse of bridges to be more expert related.

Astane-Asl (2008) investigated the influence of the failure of gusset plate connection failure in progressive collapse of a steel bridge. In this research progressive collapse of I-53W steel deck truss bridge located in Minneapolis in U.S. was studied. Finally, the author suggested that regular inspections and evaluations should be done to detect potential failure and provide remedies. The author also recommended the construction companies to study the effect of adding heavy loads of construction equipment on the bridge which increases the stresses in the bridge.

Wollff and Starossek (2009) indicated that progressive collapse investigations are mainly for buildings and a quasi-static analysis with a dynamic amplification factor of 2.0 is too large and results in uneconomic solutions. They examined the structural behavior of cable-stayed bridges due to the loss of one cable. They conducted a dynamic analysis including large displacements.

Miyachi et al (2012) conducted a progressive collapse analysis for three truss bridges. Their bridges were continuous steel truss bridges with the total span length of 230 m. Their analyses focused on the influence of live load intensity and distribution. They applied design loads, and then increased the live load until the bridge collapsed. Their study determined the collapse process under live load distribution, and also examined the span ratio and buckling strength.

Chapter 3

RESEARCH METHODLOGY

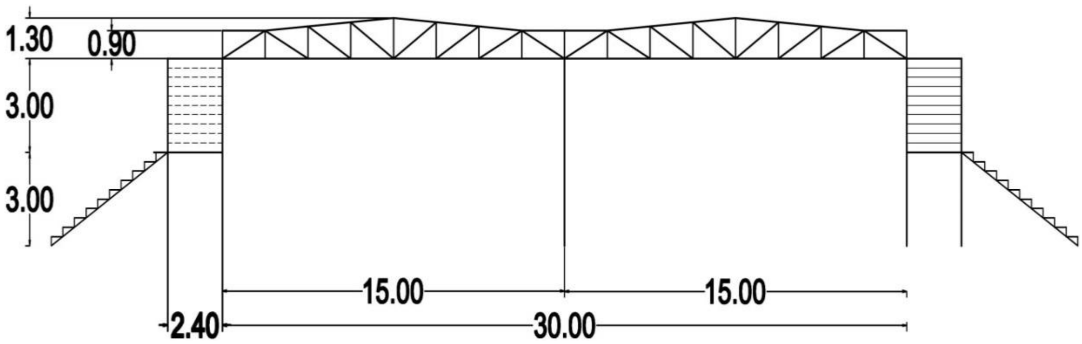
This chapter describes the structural system of the studied bridge and its alternatives and explains how they are modeled in SAP2000. The chapter ends by describing how the loads were implemented and suggests a cost estimation function for bridges.

3.1 Description of the reference bridge

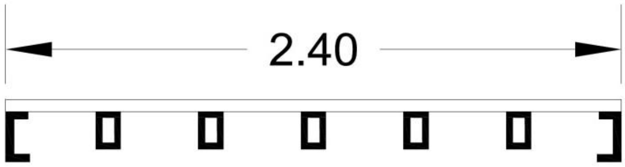
In this study, a pedestrian bridge located between northern and southern campuses of the Eastern Mediterranean University, Famagusta, Cyprus (Figure 3.1 from Google map and Figure 3.2) was chosen as a reference. The bridge is composed of two spans with the length of L=15m as depicted in Figure 3.3. The concrete deck has a thickness of 15 cm and width of 2.4 m. The deck has two steel main girders that are located at the outer edges. These girders are attached by a set of equally spaced floor beams. Geometrical dimensions of the bridge are given in Figure 3.3. The 3D model of the bridge can be seen in Figure 3.4.

Figure 3.1: Satellite view of the bridge (obtained from Google map)


Figure 3.2: The photo of the constructed bridge



a) Bridge elevation dimensions



b) Cross section of the bridge deck

Figure 3.3: Geometrical dimensions of the reference bridge

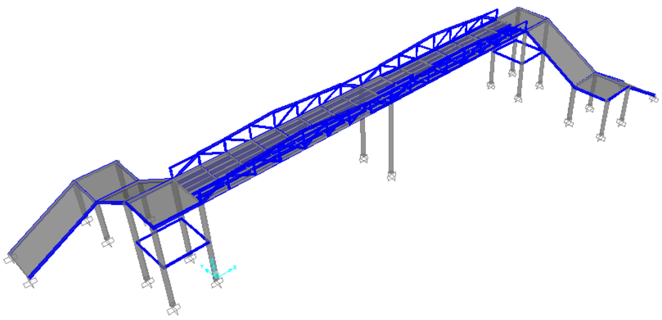


Figure 3.4: 3D model of the reference bridge

3.2 Design alternatives

Six other design alternatives are considered in this research to determine the best one from four perspectives of structural behavior, cost, aesthetics and weight.

Some considerations were considered in the design process as follows:

For arch bridges, the rise-to-span ratio should be in the range of ${}^{1}/_{2}$ to ${}^{1}/_{10}$ so that the design could be economic and show appropriate structural behavior (Wai-Fah and Lian, 2014).

The truss bridges considered here are parallel trusses. For this type of truss, economic span length ranges from 6m to 50m and the span-to-depth ratio ranges from 15 to 25 depending on the induced loads (Davison and Owens, 2011).

The thickness of concrete deck is chosen based on ACI 318-11. According to this standard, the minimum thickness should be L/20 (L is the span length between two beams).

Alternatives are as follows:

1- Single span arch bridge (with variable and simple sections);
2- Double span arch bridge;
3- Single span truss bridge;
4- Double span truss bridge;
5- Single span truss bridge based on Capstone project

3.2.1 Single span arch bridge

This alternative is a single span arch bridge with a span length of 30 m (Figure 3.5). The deck width is similar to the reference and is modeled as a horizontal cross bracing. Two types of cross-sections are considered: simple and variable. Figure 3.6 shows a schematic view of these sections. Figure 3.7 represents the 3D model of the bridge.

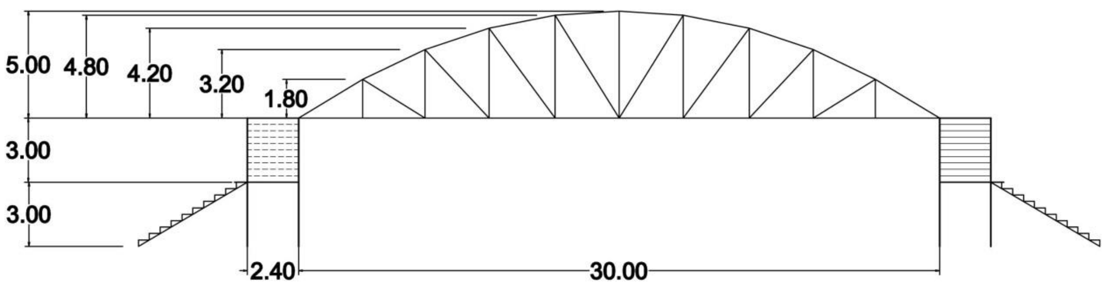


Figure 3.5: Geometrical dimensions of single span arch bridge

Figure 3.6: Partial schematic view of a) simple and b) variable cross-sections

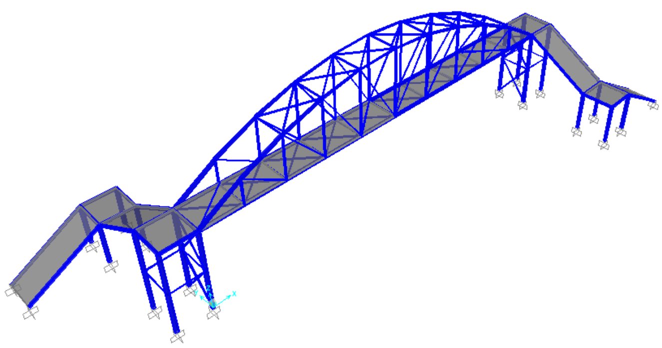


Figure 3.7: 3D model of single span arch bridge

3.2.2 Double span arch bridge

Double span arch bridge consists of two spans each one having a length of L=15m. The deck width is similar to that of the reference bridge and is modeled as a horizontal cross bracing. Figure 3.8 provides more information regarding the geometry of this alternative. Figure 3.9 represents the 3D model of the bridge.

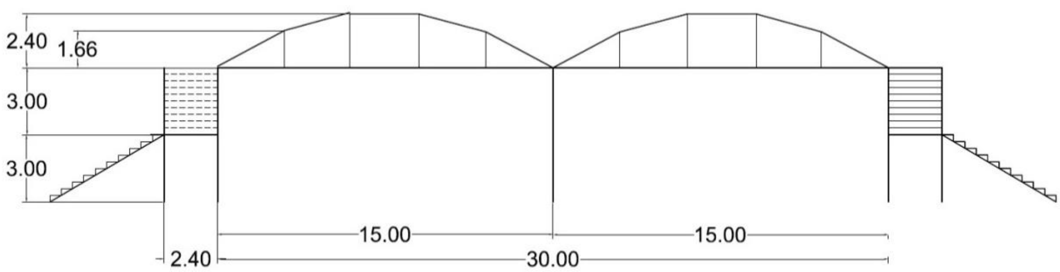


Figure 3.8: Geometry of double span arch bridge

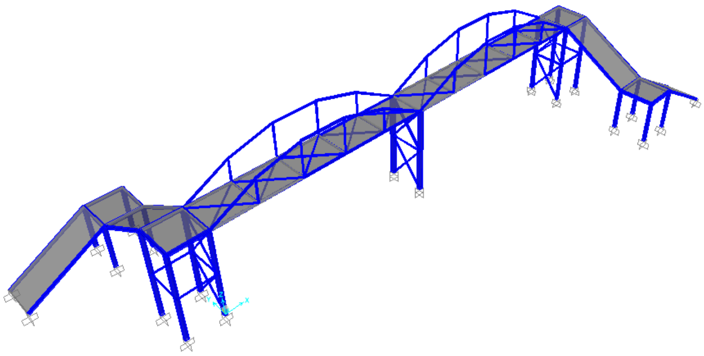


Figure 3.9: 3D model of the double span arch bridge

3.2.3 Single span truss bridge

This bridge has a span with the length of 30m and a truss superstructure. The deck system is identical to the previous alternatives and has horizontal cross bracing. More details about its geometric dimensions are depicted in Figure 3.10. Figure 3.11 provides the 3D model of the bridge.

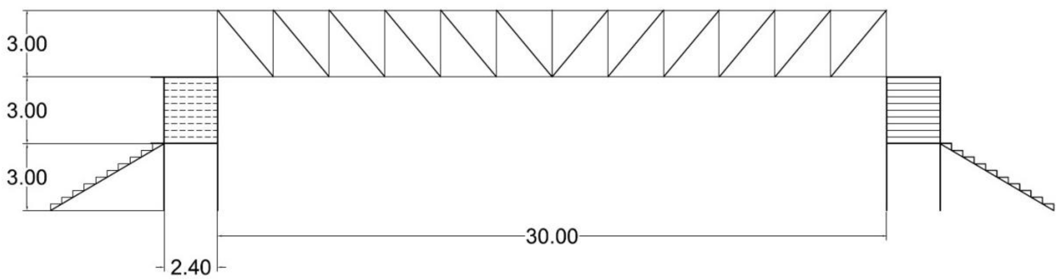


Figure 3.10: Geometrical dimensions of single span truss bridge

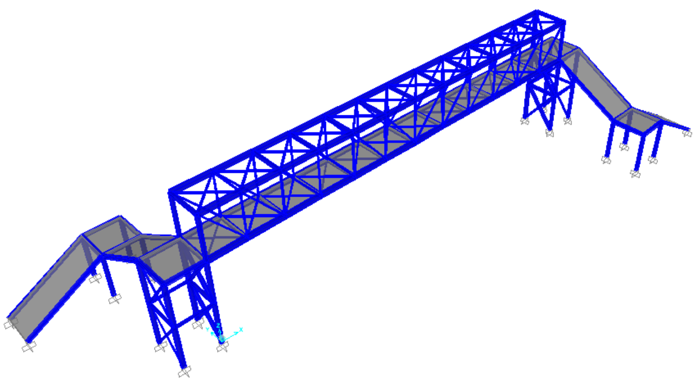


Figure 3.11: 3D model of the single span truss bridge

3.2.4 Double span truss bridge

This alternative had double span truss bridge with two equally divided spans (L=15 m). The deck system was a horizontal cross bracing. Figure 3.12 provides a schemtaic view of this design alternative. The 3D model of this alternative is represented in Figure 3.13.

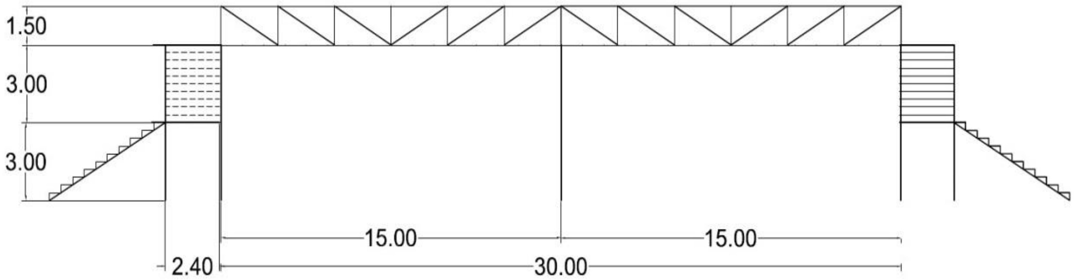


Figure 3.12: Geometrical dimensions of double span truss bridge

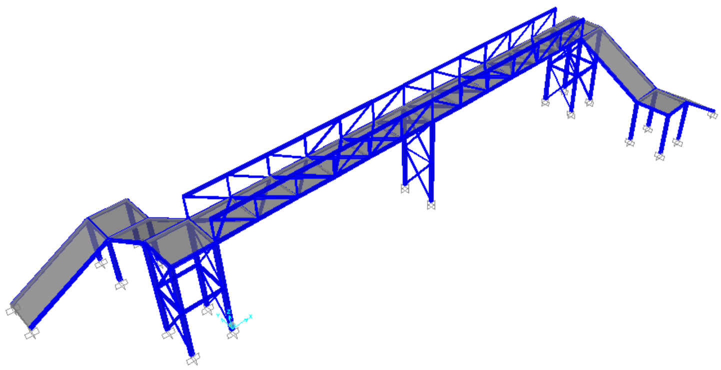


Figure 3.13: 3D model of the double span truss bridge

3.2.5 Single span truss bridge based on Capstone project (Bahmani and Aghajani Namin, 2010)

This alternative is based on a Capstone Project done in civil engineering department of Eastern Mediterranean University (Bahmani and Aghajani Namin, 2010). The aim of this project was to design a durable footbridge at the end of the road in front of Deniz plaza located in city of Magusa, Cyprus. The original design has the span length of 24 m and the bridge deck height was 6 m above the road centerline. A 3D model of this capstone project bridge is shown in Figure 3.14.

Since the required span length for the project studied in the current research is 30 m, some changes were applied to the original design to meet the requirements. Figure 3.15 represents the design considered and its geometric specifications. The deck system is single horizontal bracing. Figure 3.16 shows the 3D model of the bridge which is used in this research as the seventh alternative.

Figure 3.14: 3D model of capstone project pedestrian bridge structure

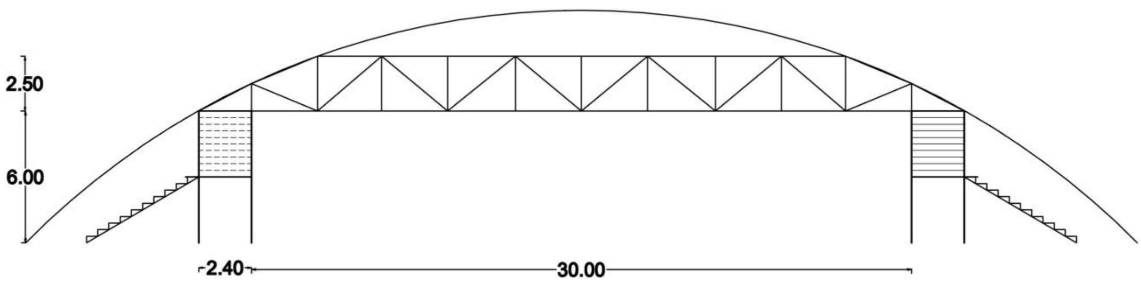


Figure 3.15: Geometrical dimensions of single span bridge based on Capstone project

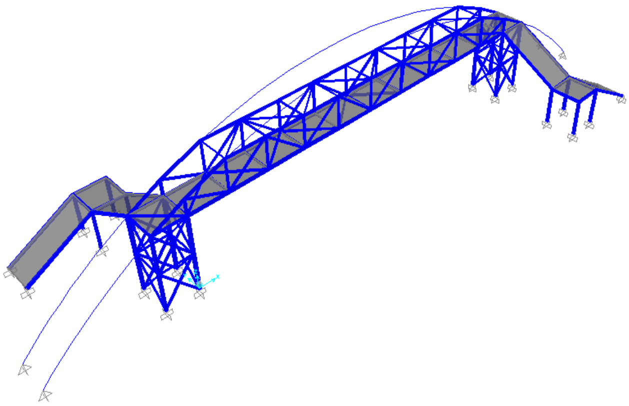


Figure 3.16: 3D Model of the single span truss bridge (Capstone project)

3.3 Modeling procedure

Pedestrian bridges have four main components: superstructure, deck, stairs and columns. These components were modeled in SAP2000 using three dimensional line elements (Figures 3.11 to 3.16). Then, the loading criteria and load combinations were defined. Afterwards, materials of the structures are defined. In this step, different materials like, concrete, deck, steel are defined and applied to the structures.

After completion of modeling, analyses are done and stresses and deformations were checked. The cross-sections that did not satisfy the design requirements were changed. Figure 3.10 illustrates the step by step modeling procedure.

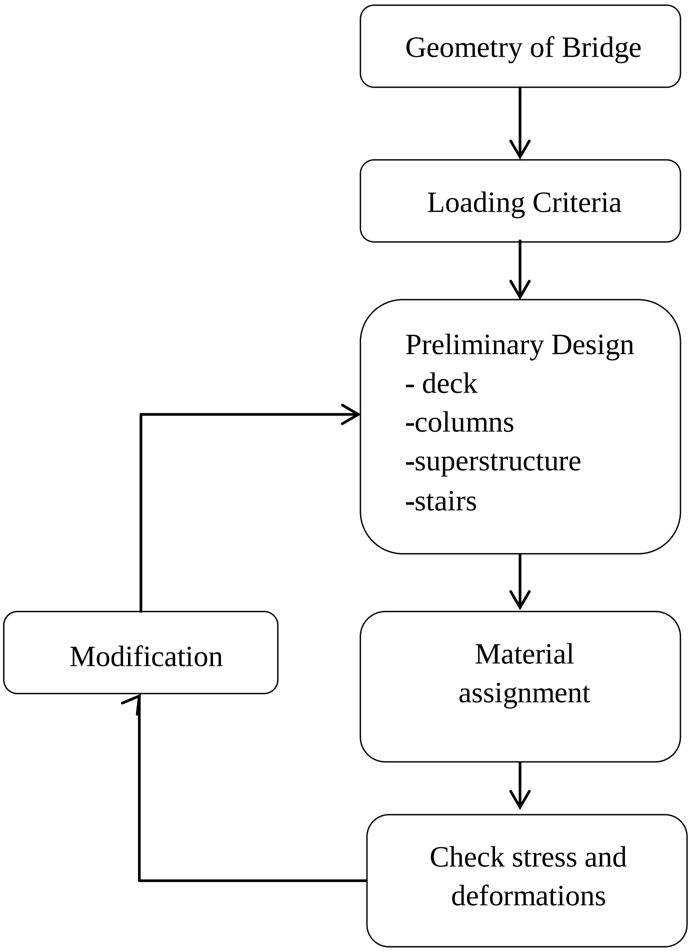


Figure 3.17: Modeling procedure

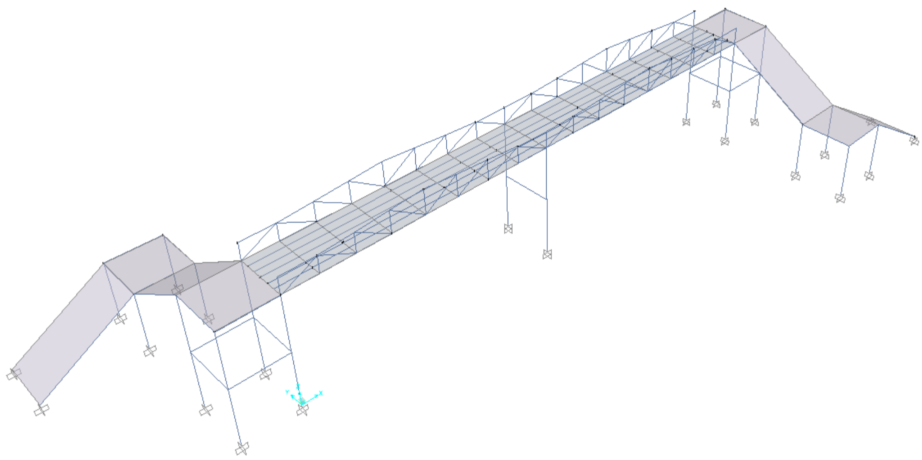


Figure 3.18: Model of the reference bridge in SAP2000

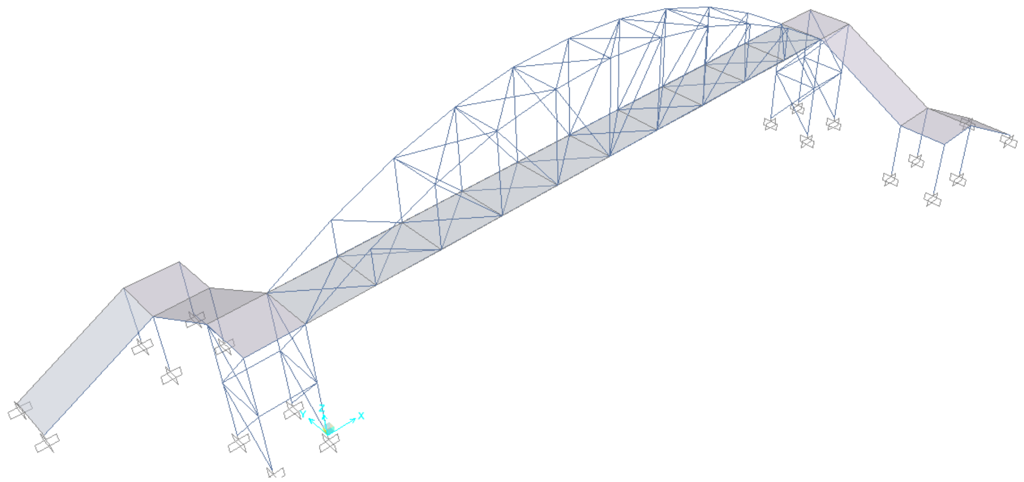


Figure 3.19: Model of the single span arch bridge in SAP2000

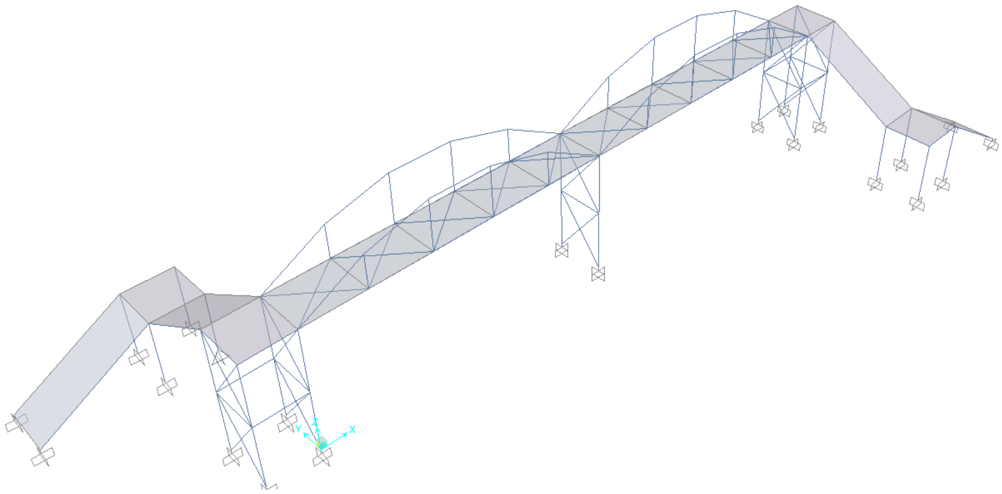


Figure 3.20: Model of the double span arch bridge in SAP2000

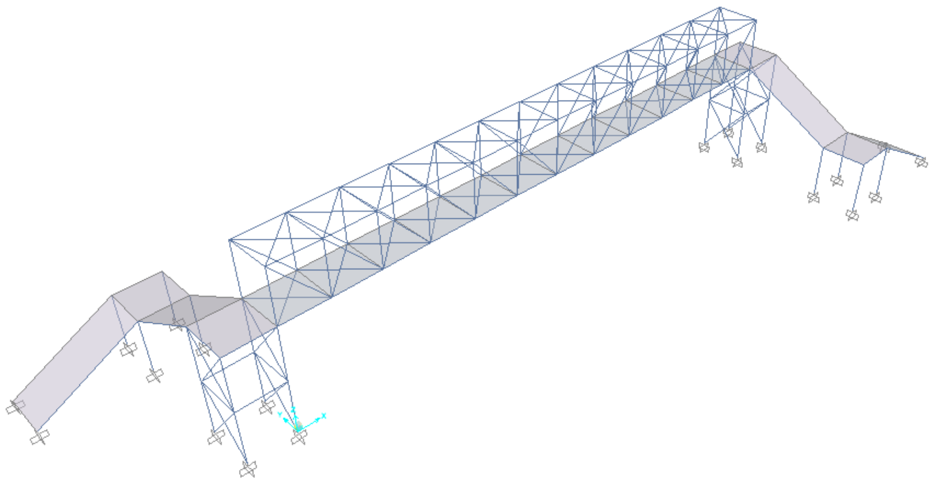


Figure 3.21: Model of the single span truss bridge in SAP2000

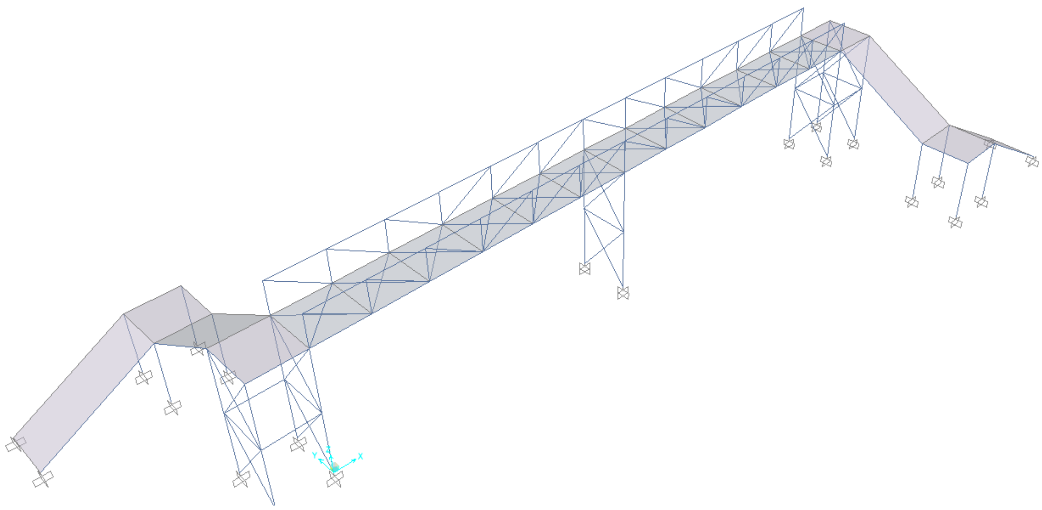


Figure 3.22: Model of the double span truss bridge in SAP2000

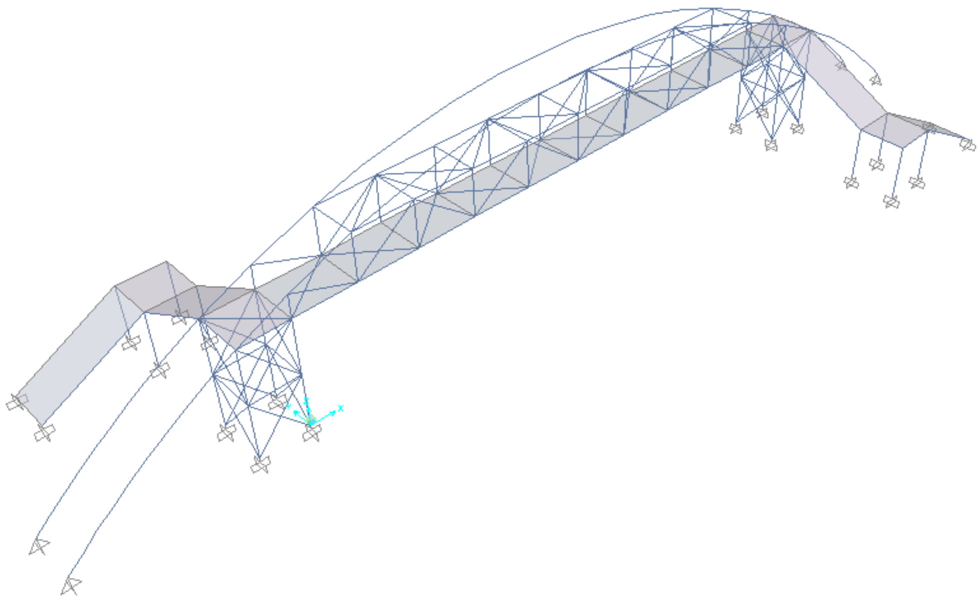


Figure 3.23: Model of the single span truss bridge (Capstone Project) in SAP2000

3.4 Design Loads

The design of any component of a bridge is based on a set of loading conditions. There are various types of loads depending on duration (permanent or transient (temporary)), direction of action, type of deformation and nature of structural action (shear, bending, torsion, etc.) (Jagadeesh and Jayaram, 2004).

There are different set of design guidelines, such as, 1) BS 5400 loads for United Kingdom, 2) Ontario Highway Bridge Design Code (OHBDC) for Canada, and 3) American Association of State Highway and Transportation Officials (AASHTO) for USA (Jagadeesh and Jayaram, 2004). In this study, as previously stated, AASHTO design guidelines were implemented. Table 3.1 presents the loads considered in the current research.

Table 3.1: Loads considered in the design process

	Definition	Abbreviation
Permanent Loads	Dead Loads	DD
Transient Loads	Earthquake	EQ
	Pedestrian Live Load	PL
	Wind Load on Structure	WS

3.4.1 Dead loads

The dead load on superstructure is the summation of the weight of all superstructure elements, such as the deck, ducts, stiffeners, utilities, miscellaneous furniture and etc. (Jagadeesh and Jayaram, 2004).

Based on the unit weights of materials existing in AASHTO LRFD Bridge Design Specifications, the calculated dead loads are as given in Tables 3.2 and 3.3 for different types of bridges.

Table 3.2: Dead loads calculated for truss bridges

Material	**Galvanized plate**	**Reinforced Concrete**	**Concrete**
Density (Kg/m^3)	7850	2400	2400
Thickness (m)	0.003	0.10	0.05
Weight (kN/m^2)	0.23	2.35	1.18
Dead Load (kN/m^2)	3.76		

Table 3.3: Dead loads for single and double span arch bridges

Material	Galvanized plate	Reinforced Concrete	Concrete
Density (Kg/m^3)	7850	2400	2400
Thickness (m)	0.003	0.15	0.05
Weight (kN/m^2)	0.23	3.35	1.18
Dead Load (kN/m^2)	4.94		

3.4.2 Live loads

Live loads are vertical loads due to the traffic and pedestrian. Live loads are those moving along the length of the span. According to this definition, in case of pedestrian bridges, a pedestrian walking on the bridge is also a live load (Jagadeesh and Jayaram 2004).

According to AASHTO, "pedestrian bridges shall be designed for a uniform pedestrian load of 90 psf (which is equal to 4.31 kN/m^2)". This loading should be modeled in a way that it produces the maximum load effects. AASHTO states that consideration of dynamic load allowance is not required with this loading.

3.4.3 Wind loads

Wind loads are complicated set of loading conditions. In order to provide a workable design, these conditions must be idealized. Wind forces must be modeled as dynamic forces. These forces can be considered as a static load which is uniformly distributed over the exposed region of the bridge. The exposed region of the bridge is considered as the summation of surface areas of all elements seen in elevation (Jagadeesh and Jayaram, 2004).

According to LRFD guide specification for the design of pedestrian bridges (AASHTO), these types of bridges should be designed for wind loads based on the AASHTO Signs Articles 3.8 and 3.9. Thus, for wind load one can do the following assumptions:

- It is assumed that the design wind speed is 80 mph (based on the 35 m/s of regional wind speed)
- The wind load on the live vehicle load is neglected.
- For calculation of the wind loading, the design life shall be taken as 50 years.

3.4.3.1 Horizontal Wind Loading

According to AASHTO signs, the design wind pressure on superstructure, P_z(psf), is defined by

$$P_z = 0.00256 K_z G V^2 I_r C_d \qquad (3.1)$$

Where, K_z is height and exposure factor, G is gust effect factor, V is basic wind velocity, I_r is wind importance factor, and C_d is wind drag coefficient.

From AASHTO signs we have:

K_z= 1.00, G=1.69, V=80 mph, I_r=1.00, and $C_d = 1.00$.

Thus, P_z=27.68 psf (1.33 kN/m^2)

Table 3.4 gives information about the vertical areas that are projected to the horizontal wind loading.

Table 3.4: Projected vertical area per linear meter

Elements	**width (m)**	**W_H (kN/m)**
Chords	0.2	0.27
Verticals	0.15	0.20
Diagonals	0.15	0.20
Deck	0.2	0.27
Arch Beams	0.3	0.40
Vertical Beams	0.2	0.27
Horizontal Beams	0.25	0.33

3.4.3.2 Vertical wind loading

According to AASHTO, for vertical wind pressure, a vertical "upward force equal to 0.02 ksf (0.958 kN/m^2) times the width of the deck, including parapets and sidewalks, is applied. This lineal force shall be applied at the windward quarter point of the deck width in conjunction with the horizontal wind loads."

The vertical wind load on the entire projected area of the superstructure applied at the windward quarter point is defined as follows:

$$WS_V = P_V . w_{deck} \qquad (3.2)$$

Where, P_V is vertical wind loading (0.958 kN/m^2), and w_{deck} is the total deck width (m).

Considering that w_{deck}=2.4 m, then,

$WS_V = 0.958 \text{ x } 2.4 = 2.30\ kN/m^2$.

Vertical load on leeward face =$2.30 \frac{kN}{m^2} \text{x} \frac{1.8m}{2.4} = 1.7\ kN/m$

Vertical load on windward face =$2.30 \frac{kN}{m^2} \text{x} \frac{0.6m}{2.4} = 2.4\ kN/m$ (uplift).

3.4.4 Seismic loads

Seismic forces are dependent on the geographic location of the bridge. Like pedestrian live loads, seismic forces are transient loads on a structure. An earthquake force is the function of the following factors (Jagadeesh and Jayaram, 2004):

- Dead load of the structure
- Ground motion
- Period of vibration
- Nature of soil

One of the important steps in calculation of seismic forces is the classification of the site according to AASHTO LRFD bridge design specifications. This guideline classifies sites based on the shear wave velocity in the upper 30.48 m (100 ft), Standard Penetration Test (SPT), blow counts and undrained shear strengths of soil samples from soil bore holes. In a previous research study on soil types in the area of Tuzla located in the eastern coast of Cyprus, (Erhan, 2009), 10 boreholes were drilled in the region from which two boreholes are close to the location of the bridge under investigation (BH4 and BH5) (Figure 3.14). Table 3.5 presents the relevant borehole test results that can be used for site classification.

Figure 3.24: Cone penetration tests and borehole locations for Tuzla region (Erhan, 2009 (scale 1:500))

AASHTO classifies sites into six groups from A through F. First of all, it recommends checking the sites for the Site Class F, which should have the following three criteria (AASHTO LRFD Bridge design specifications):

- Peats or highly organic clays (H>10 ft of peat or highly organic clay where H=thickness of soil)
- Very high plasticity clays (H>25 ft with PI>75)
- Very thick soft/medium stiff clays (H>120 ft)

According to the geotechnical information of the region the existing soil is not peat or highly organic clay and the plasticity index of the soil does not exceed 75. For the third criteria, there is insufficient information about the extent of the depth of the existing soft clay due to the investigations being conducted up to the depth of 14.5m. For these reasons, the site cannot be classified as F.

Table 3.5: Soil classification information for Tuzla region (Erhan, 2009)

BH4					**BH5**				
Depth (m)	Wc (%)	Depth (m)	PI (%)	Soil type	Depth (m)	Wc (%)	Depth (m)	PI (%)	Soil type
1.00	26.42	0.00-1.00	27.32	CL	1.00	21.28	0.00-1.00	34.20	CH
2.00	29.33	1.00-3.00	34.12	CH	2.00	30.02	1.00-3.00	34.86	CH
3.00	41.18	3.00-3.45	16.82	CL	2.50	30.06	3.00-4.00	36.40	CH
3.45	43.13	3.45-4.50	23.38	CL	3.00	31.98	4.00-6.00	22.96	CL
4.50	40.98	5.00-6.00	20.05	CL	3.45	36.49	6.00-7.00	33.06	CH
5.00	34.51	6.50-7.00	6.30	CL-ML	4.00	39.44	7.00-8.00	33.28	CH
5.50	46.85	7.00-7.45	3.01	ML	5.00	48.66	8.00-10.50	30.80	CH
6.00	41.76	7.45-8.00	3.39	ML	5.50	40.94	10.50-12.00	31.40	CH
6.50	38.37	8.00-8.20	19.53	CL	6.00	47.37	12.00-14.50	33.45	CH
7.00	13.60	8.20-10.00	31.08	CH	7.00	49.61			
7.45	16.77				7.50	51.99			
8.20	17.34				8.00	54.67			
10.00	25.11				9.00	54.02			
					10.00	56.25			
					11.00	54.48			
					12.00	50.00			
					12.50	60.18			
					13.00	55.68			
					14.00	55.25			
					14.50	56.20			
Undrained Shear Strength (S_u) =0.406 ksf (depth 4.5-5.0) < 0.5 ksf					Undrained Shear Strength (S_u)=0.94 ksf (depth 5.0-5.5) and 0.22 ksf (depth 14.0-14.5)				
Average Standard Penetration Test (SPT) blow count= 3.9					Average Standard Penetration Test (SPT) blow count= 4.65				

If we check the conditions for Site Class E (AASHTO LRFD Bridge Design Specifications, 2012):

> "Check for existence of a soft layer with total thickness > 10 ft, where soft layer is defined by S_u < 0.5 ksf, w > 40%, and PI >20. If these criteria are met, the site can be classified as Site Class E."

It can be seen from Table 3.5 that with a little tolerance the site is among Class E ones.

According to CEN (2007) in which PGA values for Cyprus determined on a probabilistic hazard map, it can be seen that PGA in Famagusta city is about 0.25g (Figure 3.15). Thus, in this study PGA is assumed to be equal to 0.25. It should be noted that there are no reported research regarding spectral acceleration coefficient at period 0.1 sec (S_1) and at period 0.2 sec (S_2) for Famugusta area. For this reason, information on a region in the United States (San Francisco bay area), which is geotechnically similar to the study area, is considered here. Therefore, the following information regarding the seismic load should be entered as model inputs (Table 3.6).

Table 3.6: Earthquake parameters for seismic load

Parameters	**Value**
Peak Ground Acceleration (PGA)	0.25
Spectral Acceleration Coefficient at Period 1.0 sec (S1)	0.2
Spectral Acceleration Coefficient at Period 0.2 sec (Ss)	0.5
Site Factor, F_{pga}, at Zero-Period on Acceleration Spectrum	1.45
Site Factor, F_a, for Short Period Range of Acceleration Spectrum	1.7
Site Factor, F_v, or Long Period Range of Acceleration Spectrum	3.2

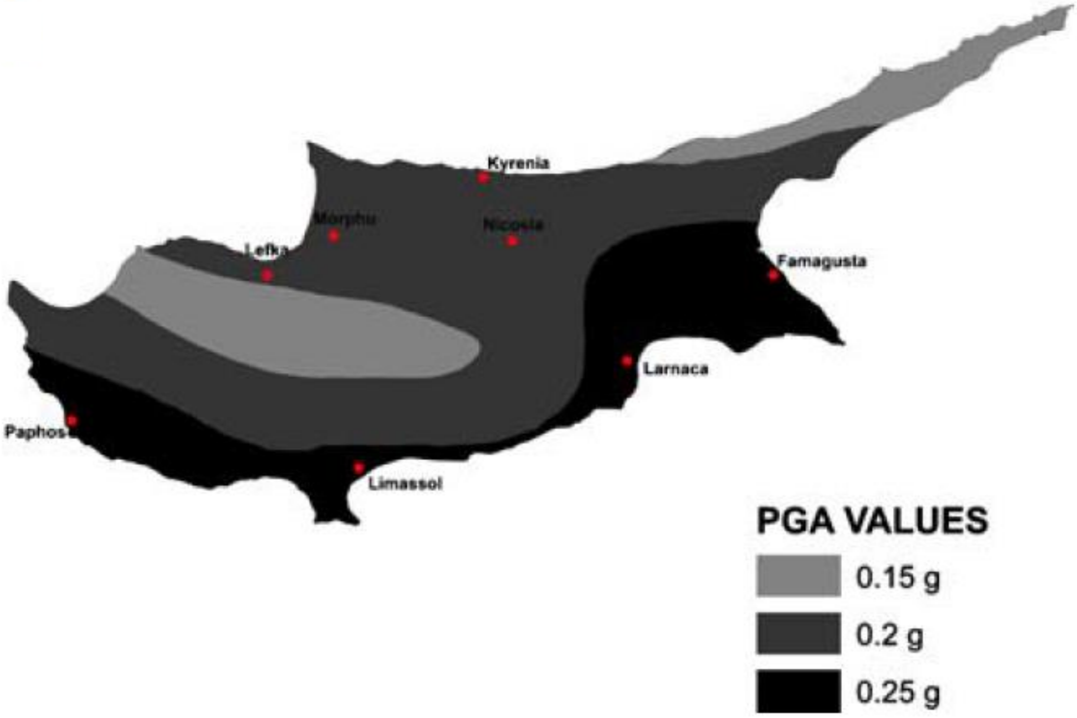


Figure 3.25: Seismic hazard maps for Cyprus (CEN 2007)

3.5 Load factors and combinations

The American Load-Resistance Factor Design (LRFD) Bridge Design Specifications make some allowance for ductility, redundancy and the operational maintenance (AASHTO 2012). The required LRFD design condition for each limit state is:

$$\sum \eta_i \gamma_i Q_i \leq \emptyset R_n \qquad (3.3)$$

Where, η_i is load modifier, $\gamma_i$is load factor, Q_i is load effect, $\emptyset$ is resistance factor, and R_n is nominal resistance.

The load modifier is obtained by combining three factors relating to ductility η_D, redundancy, η_R, and operational classification, η_I:

$$\eta_i = \eta_D \eta_R \eta_I \geq 0.95 \qquad (3.4)$$

Considering that in this research only dead, live, wing and seismic loads are studied and the bridge design is limited to pedestrian ones, then seven load combinations should be examined. The load factors and combinations are given in Table 3.7.

Table 3.7: Load combinations (AASHTO LRFD Bridge Design Specifications, 2012)

Load Combinations	**Explanation**	**DD**	**PL**	**WS**	**EQ**
Strength I	Basic load combination relating to the normal vehicular use of the bridge without wind.	1.25	1.75	0	0
Strength III	Load combination relating to the bridge exposed to wind velocity exceeding 55 mph.	1.25	0	1.4	0
Service I	Load combination relating to the normal operational use of the bridge with a 55 mph wind and all loads taken at their nominal values. Also related to deflection control in buried metal structures, tunnel liner plate, and thermoplastic pipe, to control crack width in reinforced concrete structures, and for transverse analysis relating to tension in concrete segmental girders. This load combination should also be used for the investigation of slope stability.	1	1	0.3	0
Service II	Load combination intended to control yielding of steel structures and slip of slip-critical connections due to vehicular live load.	1	1.3	0	0
Service III	Load combination for longitudinal analysis relating to tension in prestressed concrete superstructures with the objective of crack control and to principal tension in the webs of segmental concrete girders.	1	0.8	0	0
Service IV	Load combination relating only to tension in prestressed concrete columns with the objective of crack control	1	0	0.7	0
Extreme event I	Load combination including earthquake. The load factor for live load γEQ, shall be determined on a project-specific basis.	1.25	0.5	0	1

3.6 Cost estimation of bridges

In order to accurately calculate the cost of the whole bridge structure, all design variables should be considered. Cost of materials usually includes cost of material purchase, fabrication, transportation to the construction site and erection. In order to estimate the cost, all of these variables should be considered in the unit cost of each material. Taking the above-mentioned points into consideration, the total cost can be estimated by the following equation (each cost element consists of cost of all aforementioned factors):

$$C_T = C_{dc} + C_r + C_{fw} + C_{sw} \tag{3.5}$$

where, C_{dc}, C_{dr}, C_{fw}, C_{sw} are cost of concrete bridge deck (and stairs), deck (and stairs) reinforcement, formwork and steelwork.

The cost of deck (stair) concrete can be estimated by

$$C_{dc} = \sum_{spans} L_s . b_s . h_s . c_{concs} + \sum_{columns} L_c . b_c . h_c . c_{concc} \tag{3.6}$$

where, L_s is the total length of the bridge span (stairs); b_s is the width of the deck (stairs), h_s is the depth of the deck (stairs), and c_{conc} is the cost of the deck (stairs) concrete per unit volume, L_c is the length (height) of the column; $b_c$and h_c are column dimensions, and c_{concc} is the cost of the column concrete per unit volume,

The cost of reinforcement can be estimated by

$$C_r = \sum_{spans} A_{st} . L_s . \gamma . c_{rs} + \sum_{columns} A_{st} . L_c . \gamma . c_{rs} \tag{3.7}$$

where A_{st} is the cross sectional area of the reinforcing steel, L_s and L_c are the lengths by single span (stairs), columns, respectively, γ is the density of the reinforcing steel, c_{rs} is the unit cost of the steel.

The cost of formwork is determined by

$$C_{fw} = \sum_{spans} c_{fw}.L_s.b_s + \sum_{columns} c_{fw}.L_c.P_c \tag{3.8}$$

where, c_{fw} is the unit cost of formwork, L_s is the total length of the bridge span (stairs); b_s is the width of the deck (stairs), L_c is the length (height) of the column and P_c is the perimeter of the column.

The cost of steel work of the superstructure is calculated by

$$C_{sw} = W_{st}.c_{st} \tag{3.9}$$

where, W_{st} is the weight of the steel superstructure, c_{st} is the unit cost of the steel. The cost of connections is included in c_{st}.

Chapter 4

ANALYSIS AND RESULTS

The analyses were carried out using SAP2000 version 16.0.0 software. After completion of the modeling steps, analyses were carried out with required modifications being applied to the models until the expected results and optimum designs were achieved. At the final step, a comparison was also carried out to find out which type of bridges give the best results in terms of structural behavior, cost, material requirement and aesthetics.

4.1 Results

The objective of this section is to repeatedly modify the models so that the optimum cross sections for all design alternatives are obtained. It is obvious that the cross sections should be selected in a way that they can satisfy design requirements and at the same time be economical. One of the important serviceability checks during the design process is deflection. Deflection was checked according to AASHTO guidelines. Furthermore, progressive collapse analysis was carried out using SAP2000 to study the behavior of the design alternatives when a sudden column loss occurs.

4.1.1 Design details and cross sections

In this section the optimum cross sections obtained from the modeling procedure are represented. The details of the models and the design procedure are given in the following Figures 4.1 to 4.7 and Tables 4.1 to 4.8.

- Reference Bridge (A1)

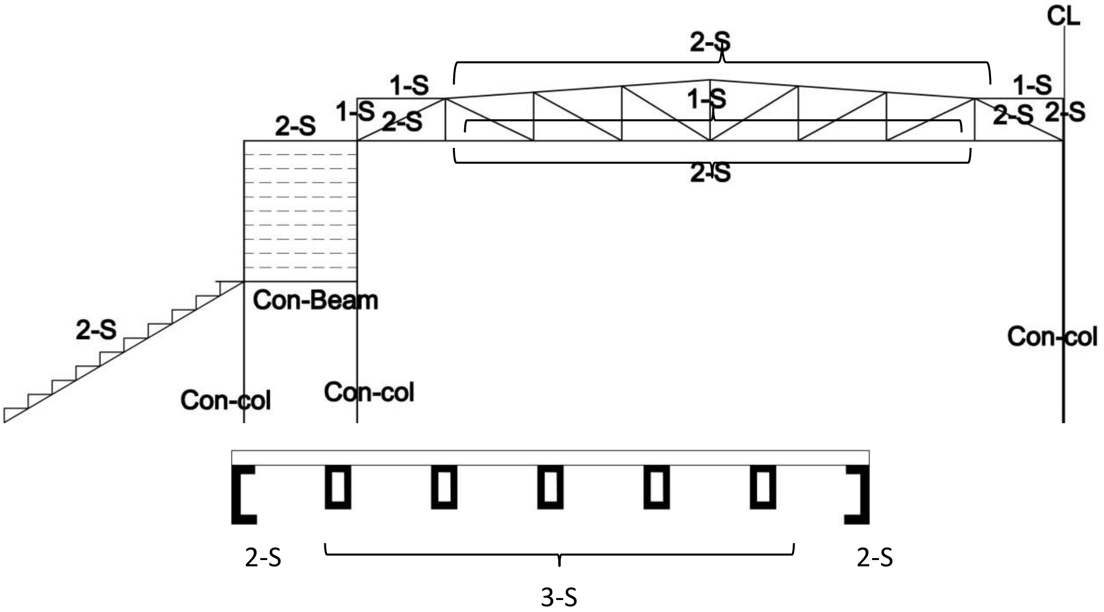
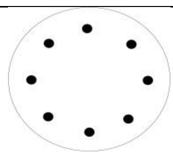
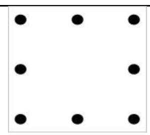


Figure 4.1: Section ID for elements of reference bridge (A1)

Table 4.1: Section types for the reference bridge (A1)

Section ID	**Section Type**
1-S	U10x5
2-S	U18x7
3-S	Box 10x10
Col	 Diameter= 35 cm Bar size = Ø10 mm
Con-beam	 75x75cm Bar size = Ø10 mm

- Single span arch bridge (variable section) (A2)

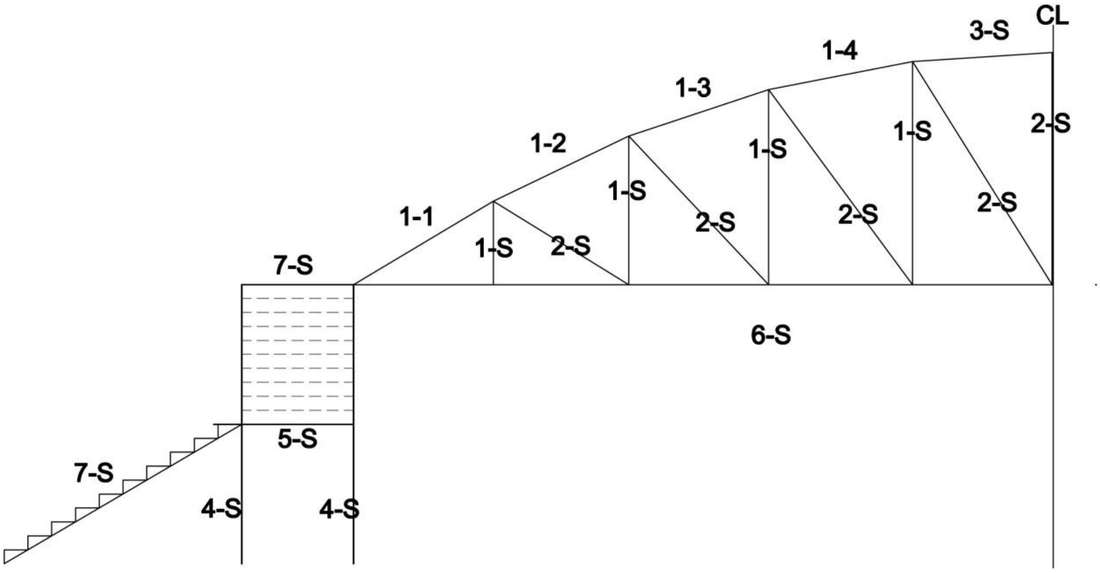


Figure 4.2: Section ID for elements of single span arch bridge with variable sections (A2)

Table 4.2: Section types for single span arch bridge with variable sections (A2)

Section ID	**Start Section**	**End Section**
1-1	HSS8x8x0.375	HSS7x7x0.375
1-2	HSS7x7x0.375	HSS6x6x0.375
1-3	HSS6x6x0.375	HSS5x5x0.375
1-4	HSS5x5x0.375	HSS4x4x0.375
2-1	HSS7x7x0.375	HSS8x8x0.375
2-2	HSS6x6x0.375	HSS7x7x0.375
2-3	HSS5x5x0.375	HSS6x6x0.375
2-4	HSS4x4x0.375	HSS5x5x0.375

Table 4.3: Section types for single span arch bridge with variable sections (A2)

Section ID	**Section Type**
1-S	HSS4x4x0.500
2-S	HSS2x2x0.1875
3-S	HSS4x4x0.375
4-S	HSS9x7x0.1875
5-S	HSS1-1/2x1-1/2x0.125
6-S	HSS7x5x0.375
7-S	W6x15
8-S*	HSS4x3x0.375
9-S**	S3x5.7

*used for column bracing
**used for horizontal bracing of the deck

- Single span arch bridge (simple section) (A3)

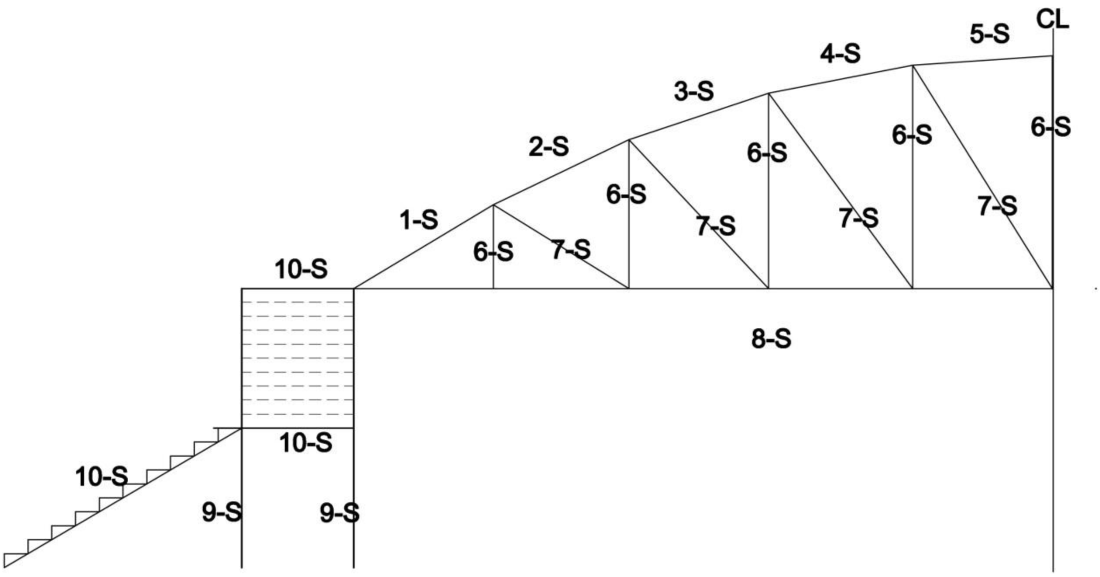


Figure 4.3: Section ID for elements of single span arch bridge with simple sections (A3)

Table 4.4: Section types for single span arch bridge with simple sections (A3)

Section ID	Section Type
1-S	HSS7x7x0.350
2-S	HSS6x6x0.375
3-S	HSS5x5x0.375
4-S	HSS4x4x0.375
5-S*	HSS4x4x0.250
6-S	HSS4x4x0.125
7-S	HSS3x3x0.125
8-S	HSS7x5x0.375
9-S	HSS5x5x0.1875
10-S	W8x31
11-S	HSS2-1/2x2-1/2x0.250
12-S**	S3x5.7

*used for column bracing
**used for horizontal bracing of the deck

- Double span arch bridge (A4)

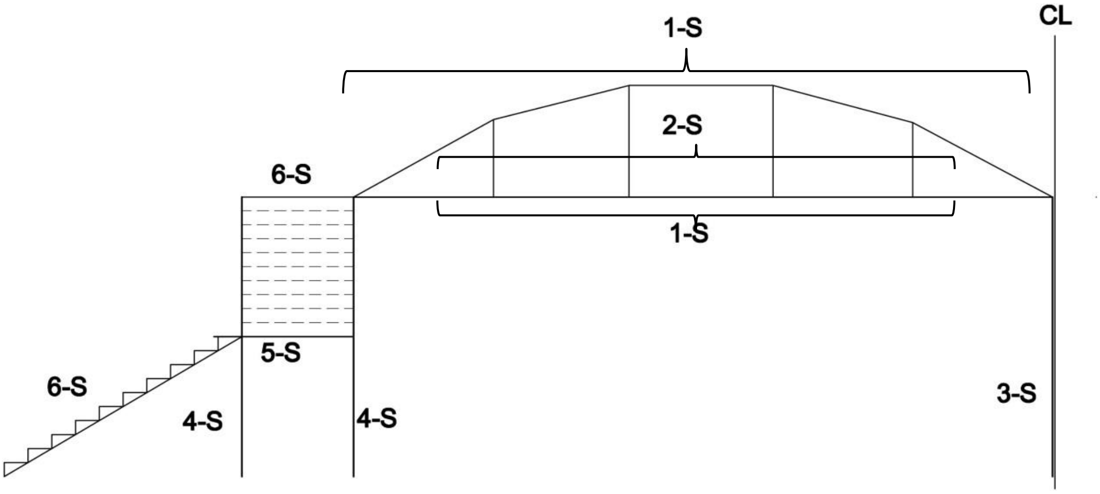


Figure 4.4: Section ID for elements of double span arch bridge (A4)

Table 4.5: Section types for double span arch bridge (A4)

Section ID	Section Type
1-S	HSS4x4x0.250
2-S*	HSS3x3x0.250
3-S	HSS10x10x0.3125
4-S	HSS9x7x0.1875
5-S	HSS2-1/2x2-1/2x0.250
6-S	W6x20
7-S**	S3x5.7

*used for column bracing
**used for horizontal bracing of the deck

- Single span truss bridge (A5)

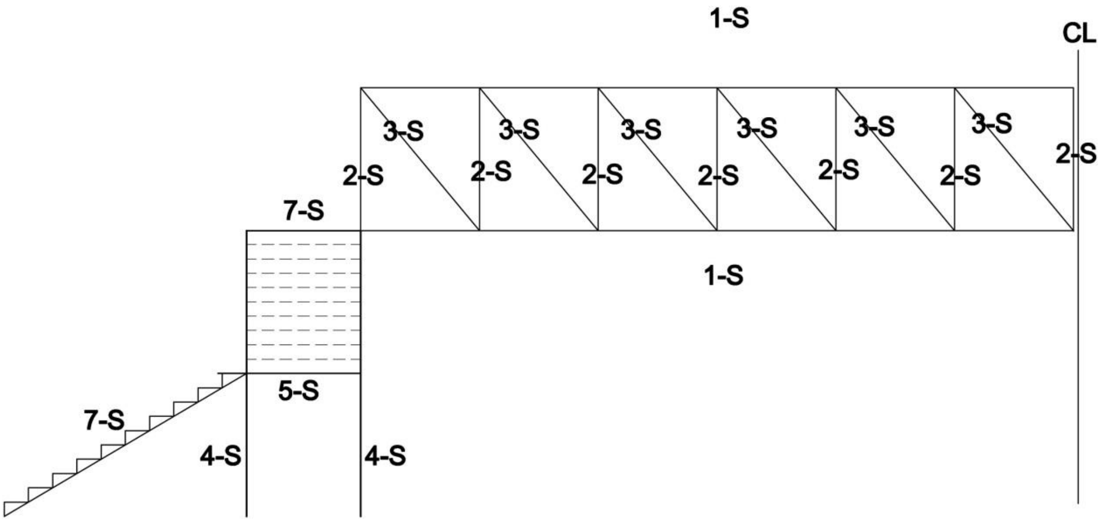


Figure 4.5: Section ID for elements of single span truss bridge (A5)

Table 4.6: Section types for single span truss bridge (A5)

Section ID	**Section Type**
1-S	HSS6x6x0.3125
2-S	HSS3x3x0.375
3-S	HSS3x3x0.250
4-S	HSS9x7x0.3125
5-S	HSS5x5x0.375
6-S*	HSS5x2x0.125
7-S	W8x31
8-S**	S3x7.5

*used for column bracing
**used for horizontal bracing of the deck

- Double span truss bridge (A6)

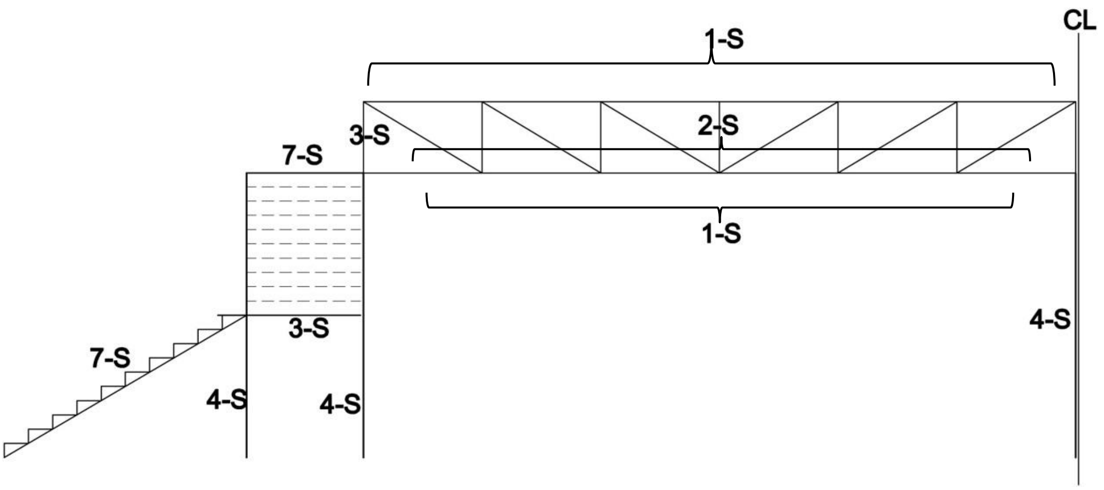


Figure 4.6: Section ID for elements of double span truss bridge (A6)

Table 4.7: Section types for double span truss bridge (A6)

Section ID	**Section Type**
1-S	HSS6x6x0.125
2-S	HSS3x3x0.250
3-S	HSS3x3x0.375
4-S	HSS9x7x0.1875
5-S	HSS5x5x0.375
6-S*	HSS5x2x0.125
7-S	W6x15
8-S**	W6x9

*used for column bracing
**used for horizontal bracing of the deck

- Single span truss bridge (Capstone project) (A7)

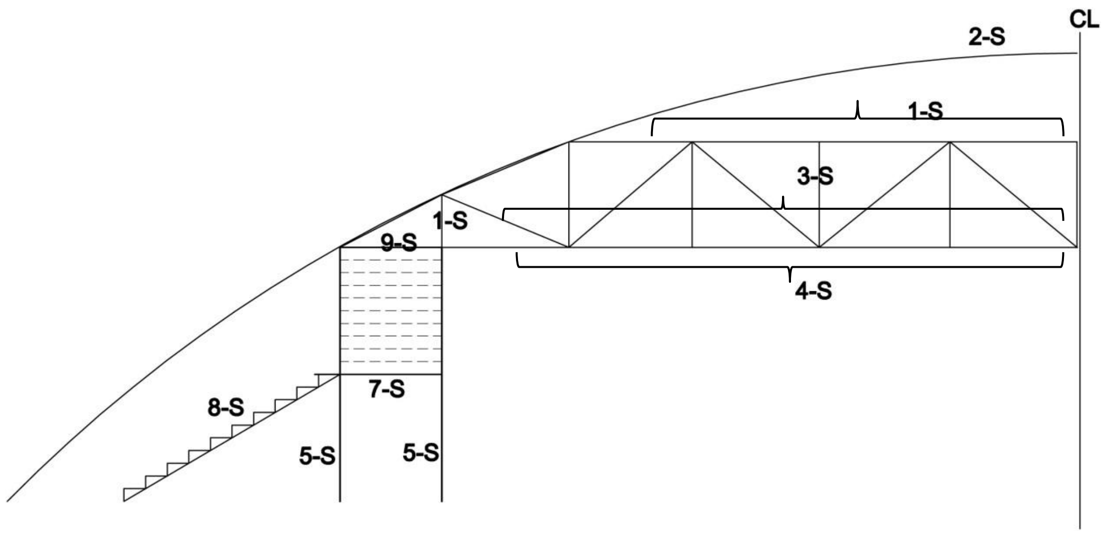


Figure 4.7: Section ID for elements of single span truss bridge-(Capstone project) (A7)

Table 4.8: Section types for single span truss bridge (Capstone project) (A7)

Section ID	**Section Type**
1-S	HSS5x5x0.500
2-S	HSS5x5x0.375
3-S	HSS4x4x0.375
4-S	HSS6x6x0.250
5-S	HSS7x7x0.1875
6-S*	HSS4x4x0.250
7-S	HSS2-1/2x2-1/2x0.250
8-S	W6x15
9-S**	W4x13

*used also for column bracing
**used for horizontal bracing of the deck

4.1.2 Deflection

To verify serviceability, the maximum deflection values should be analyzed. Deflection was checked according to AASHTO LRFD for the design of pedestrian bridges, which states that "deflections should be investigated at the serviceability limit state using combination Service I". According to this guideline, for spans other than cantilever arms, the displacement of the bridge caused by the unfactored pedestrian live loading, should not exceed L/360 (L is the length of span). Additionally, the horizontal deflections under unfactored wind loading should also be checked. This type of deflection should not exceed L/360 (L is the length of span).

Therefore, vertical displacements in load combination of Service I and horizontal displacement in the direction of the wind load have to be taken into account. Table

4.9 shows the above-mentioned displacements for all design alternatives. In this table, each alternative is scored in ordinal way. The overall scores highlight that double span bridges have better performances than single span ones. The best performance is related to the double span truss bridge.

The overall score in Table 4.9 is calculated using Ordered Weighted Averaging (OWA) aggregation operator (See the Appendix). To apply this operator it is required to form the scores matrix (Step 1). Then, this operator sorts the scores for individual alternatives in descending order (Step2). This operator then, assigns order weights to the sorted scores. Thus these weights are needed to be calculated (Step 3) and then multiplied by the sorted scores (Step 4). Finally, the OWA operator calculates the overall score by summing up the weighted scores (Step 5). The procedure for calculation of the overall score in Table 4.9 is as the following:

$$
Step\ 1)\quad
\begin{array}{c}
Alternative \\ Reference \\
Single\ Span\ arch\ (Variable\ section) \\
Single\ span\ arch\ (simple\ section) \\
Double\ span\ arch \\
Single\ span\ truss \\
Double\ span\ truss \\
Single\ span\ truss\ (Capstone)
\end{array}
\quad
\begin{array}{cc}
Score_1 & Score_2 \\
\end{array}
\begin{bmatrix}
6 & 7 \\
3 & 1 \\
4 & 4 \\
7 & 3 \\
2 & 6 \\
5 & 6 \\
1 & 2
\end{bmatrix}
$$

$$
\xrightarrow{Step\ 2)\ Sorting}
\begin{bmatrix}
7 & 6 \\
3 & 1 \\
4 & 4 \\
7 & 3 \\
6 & 2 \\
6 & 5 \\
2 & 1
\end{bmatrix}
$$

Step 3) The order weights for aggregation of two scores (*n=2*) are calculated using (step3) $w_j = \left(\frac{j}{n}\right)^{\left(\frac{1}{\beta}\right)-1} - \left(\frac{j-1}{n}\right)^{\left(\frac{1}{\beta}\right)-1}$ are w_1=0.2495, w_2=0.7505 (*j=1,…,n,* for the values of β see the Appendix).

$$\xrightarrow[\text{Multiplication of 2nd column by } w_2]{\text{Step 4) Multiplication of 1st column by } w_1} \begin{bmatrix} 1.74 & 4.50 \\ 1.75 & 0.75 \\ 0.99 & 3.00 \\ 1.74 & 2.25 \\ 1.49 & 1.50 \\ 1.49 & 3.75 \\ 0.49 & 0.75 \end{bmatrix} \xrightarrow{Step5)\ Aggregation} \begin{bmatrix} 6.25 \\ 1.50 \\ 4.00 \\ 3.99 \\ 2.99 \\ 5.25 \\ 1.25 \end{bmatrix}$$

Table 4.9: Maximum vertical and horizontal deflections

Alternatives	**Deflection (cm)**		**Overall**
	Vertical (score)	**Horizontal (score)**	**score (Rank) ***
Reference Bridge	1.18 (6)	0.05 (7)	6.25 (1)
Single span arch bridge (variable section)	1.86 (3)	0.33 (1)	1.50 (6)
Single span arch bridge (simple section)	1.81 (4)	0.14 (4)	4.00 (3)
Double span arch bridge	1.01 (7)	0.17 (3)	3.99 (4)
Single span truss bridge	3.14 (2)	0.11 (6)	2.99 (5)
Double span truss bridge	1.47 (5)	0.11 (6)	5.25 (2)
Single span truss bridge (based on Capstone project)	3.55 (1)	0. 28 (2)	1.25 (7)

* The overall scores are calculated using Ordered Weighted Averaging (OWA) aggregation operator. For more details see Appendix.

4.1.3 Progressive collapse analysis

"A progressive collapse is a chain reaction type of failure, which follows damage to a relatively small portion of a structure", (Ellingwood and Leyendecker 1978). When a local failure of a primary component of a structure results in failure of connecting members and then, the failure of partial or whole systems occurs, this is called progressive collapse. This process is dynamic and usually leads to large deformations until the structural system finds alternative load paths to survive. It is important to know that the final damage due to progressive collapse is not proportional to the initial damage (Kim et al., 2009). Progressive collapse analysis is done by removing one of several major load bearing elements and studying the behavior of the damaged structure. According to GSA Progressive Collapse Guidelines (GSA, 2003), removal of only one primary load bearing element at a time is sufficient.

In this section progressive collapse analysis is done in order to study the effect of member removals on the bridge structure. Progressive collapse and dynamic behavior are evaluated using time-history analysis in SAP2000 as the following steps:

1. From the previous analyses and design given in this chapter, the most critical column of each bridge alternative is determined and the internal forces are obtained.
2. For each alternative, a new model was created, in which the critical column was removed. The removed column equivalent forces were applied to simulate the presence of the removed column.
3. A time history analysis was carried out to simulate the removal of the column. In this process, the equivalent forces should be reduced to zero in a period of time which matches the time in which the column was removed. This procedure was done by applying opposite loads to the equivalent ones which should be increased from zero to the value equal to the equivalent column loads.

Dynamic analysis for progressive collapse usually is carried out under initial conditions (Buscemi and Marjanishvili, 2005). Initial conditions mean deformed shape of the structure under normal loading before any damage imposed to it. Therefore, the dynamic analysis for progressive collapse should be conducted when the displacements under normal conditions are applied.

4.1.3.1 Time History Functions

A time history analysis can be linear or non-linear and is used to evaluate the dynamic response of a structure under loadings that are defined by specified time functions. In time history analysis the following dynamic equilibrium is solved:

$$[M]\ddot{d} + [C]\dot{d} + [K]d = F(t) \qquad (4.1)$$

where, $[M]$ is mass matrix, $[C]$ is damping matrix, $[K]$ is stiffness matrix, $F(t)$ is vector of nodal loads, and d is an unknown vector of nodal displacements (Kurowski, 2014).

In the time history analysis, two linear time functions were defined: the first one was used to set the initial conditions and the second one to simulate the column removal. Firstly, the load combination together with the equivalent column load was applied using a single time-history load case which was defined by the first time function that gradually ramps these loads to their supposed values. Figure 4.8 shows the function graph. As it can be seen, the ramp time of the first time function is 2 seconds. This means that the load combination reaches its full values in 2 seconds, and then, the structure experiences its displacements and deformed shape before any damage and column removal.

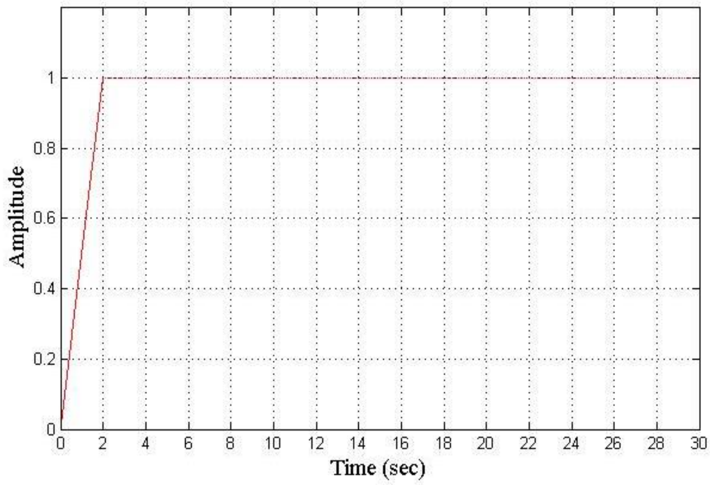


Figure 4.8: Time function graph for applying the load combination

The initial conditions lasts until 6th second and then a separate time function applies the column removal load with a later arrival time (6th second). This function is a ramp type one which ramps a load in the direction opposite to the column equivalent load and equal to it in 2 seconds. This function is applied to the load pattern which defines the column equivalent load, and thus, its amplitude value should be -1 (Figure 4.9)

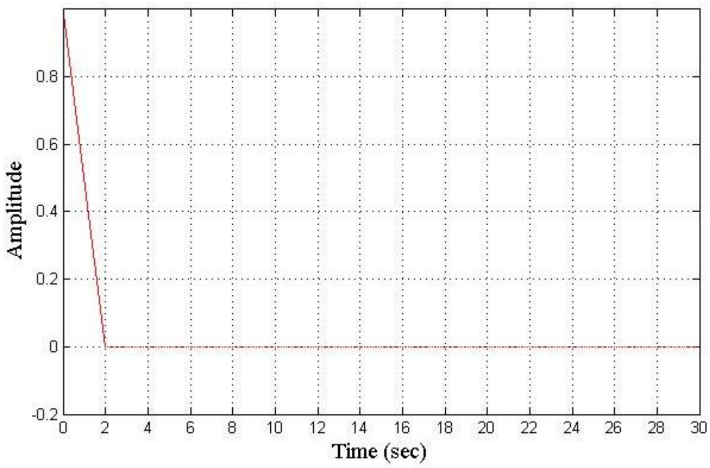


Figure 4.9: Time function graph for simulation of column removal

4.1.3.2 Assumptions

There are some assumptions in this analysis that should be considered in the evaluation of results. Choosing the load combination for the progressive collapse analysis is done based on the load combinations of GSA and UFC for dynamic analyses:

- GSA: $(DL + 0.25LL)$
- UFC: $(1.2DL + 0.5LL + 0.2W)$

As it is obvious, the UFC adopts a more conservative approach than GSA to analyze a building, and takes the wind load into consideration. According to the load combination of UFC and considering that there are no guidelines for progressive collapse analysis of bridges, limit state Service I of the AASHTO was chosen. Limit state guidelines is related to the normal use with a 88.5 km/h (55 mph) wind $(1DL + 1LL + 0.3W)$. The load coefficients of live and wind load in Service I are more than those in UFC. This may lead to more conservative results but allows us to be on the safe side. Finally, it was assumed that removal of one column may affect the function of the braces connected to it. Therefore, such braces are also removed together with the column.

The outputs of the progressive analysis are presented in Figures 4.10 to 4.30.

- Reference Bridge

In this model, the middle column was removed since from the primary model it was revealed that this column carries more forces than the others. Figure 4.10 shows the deformed shape of the structure as a result of column removal process. The results show that joint J-2 has the highest displacement. It experiences a displacement about

10.36 cm (Figure 4.11). The results also indicate that immediately after the column removal the other middle column bears the highest axial force when compared to the other columns (300.80 KN, Figure 4.12).

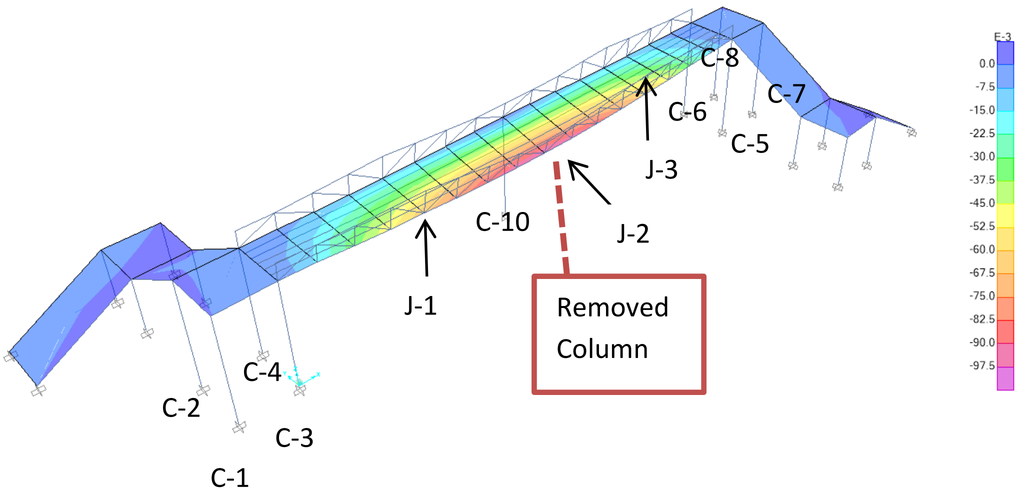


Figure 4.10: Deformed shape after column removal in the reference bridge

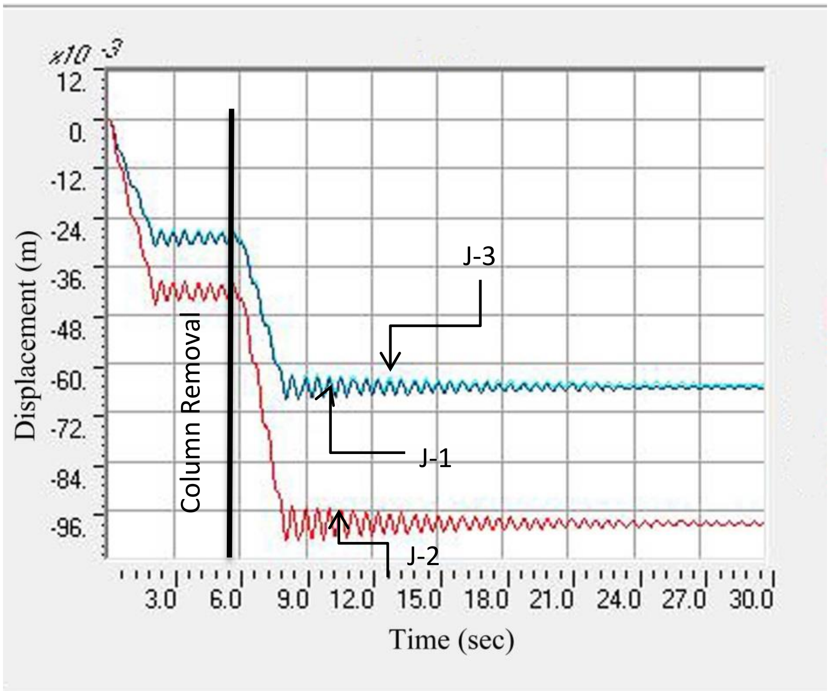


Figure 4.11: Vertical displacement of joints in the reference bridge

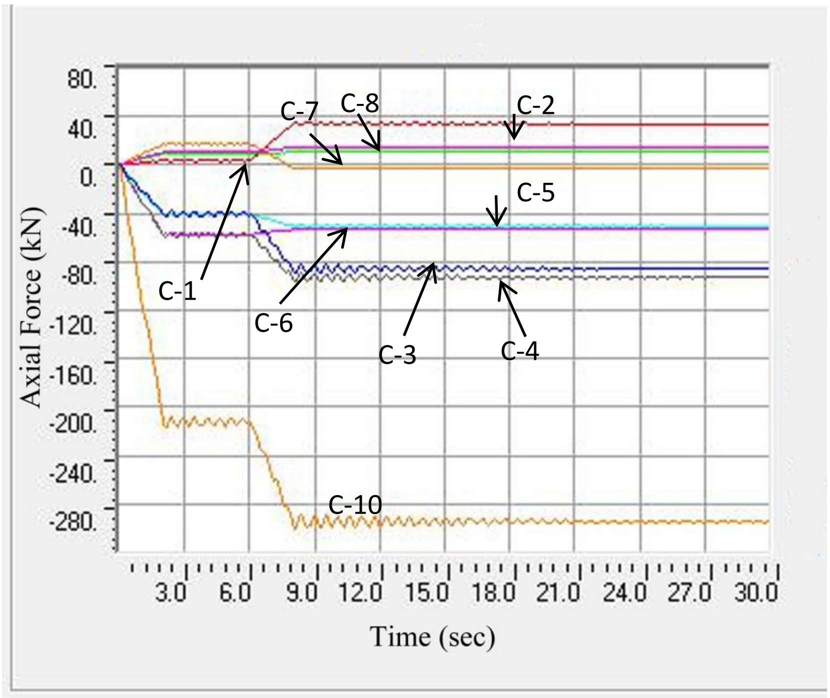


Figure 4.12: Column axial forces in the reference bridge

- Single span arch bridge (variable section)

Figure 4.13 demonstrates which column is removed. The maximum vertical displacement is related to joint J-2 and is about 2.83 cm (Figure 4.14). Figure 4.15 illustrates the axial force changes in the remaining columns. The axial force in column C-3 increases to 214.70 kN after column removal.

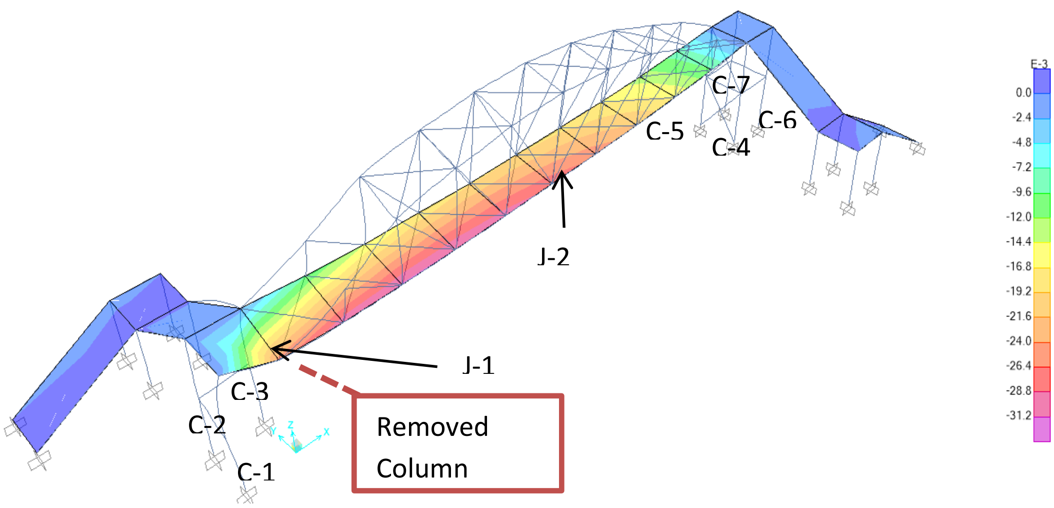


Figure 4.13: Deformed shape after column removal in single span arch bridge with variable sections

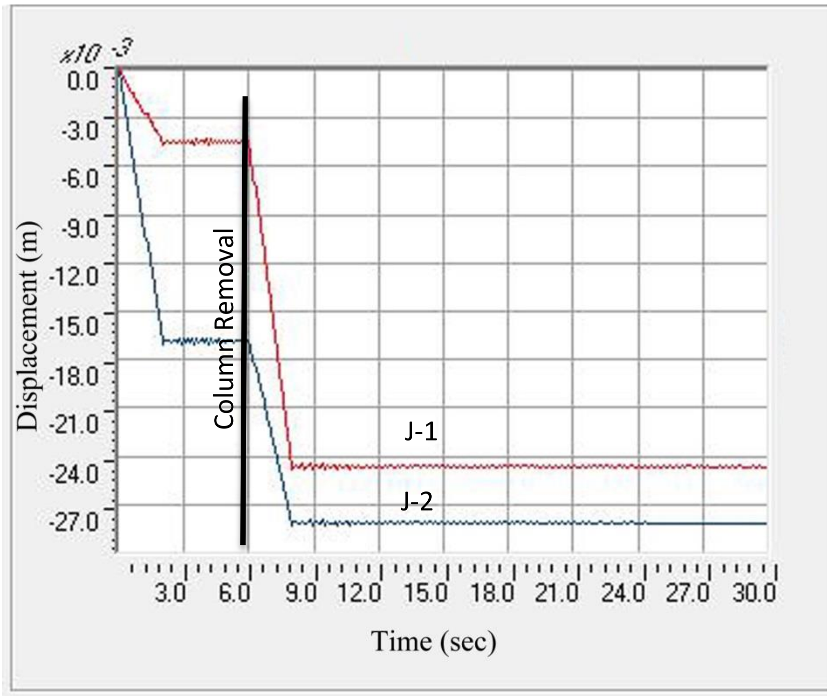


Figure 4.14: Vertical displacement of joints in the single span arch bridge with variable sections

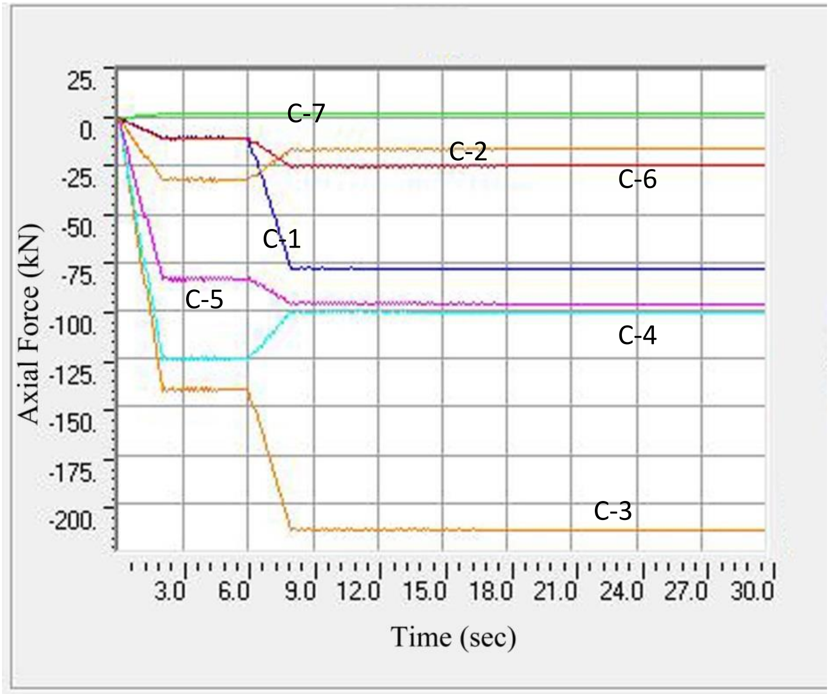


Figure 4.15: Column axial forces in single span arch bridge with variable sections

- Single span arch bridge (simple section)

The deformed shape of this alternative is represented in Figure 4.16 Maximum vertical displacement in this bridge is related to joint J-1. It can be seen form Figure 4.17 that this displacement is about 4.64 cm. Maximum axial force is in column C-3, about 190.40 kN(Figure 4.18).

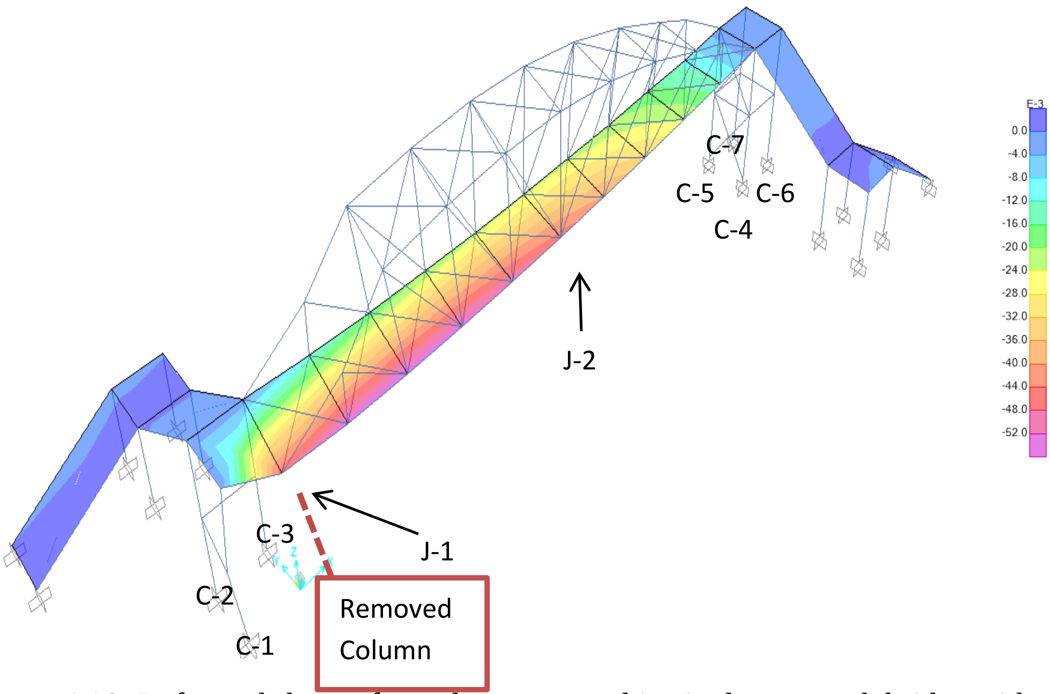


Figure 4.16: Deformed shape after column removal in single span arch bridge with simple sections

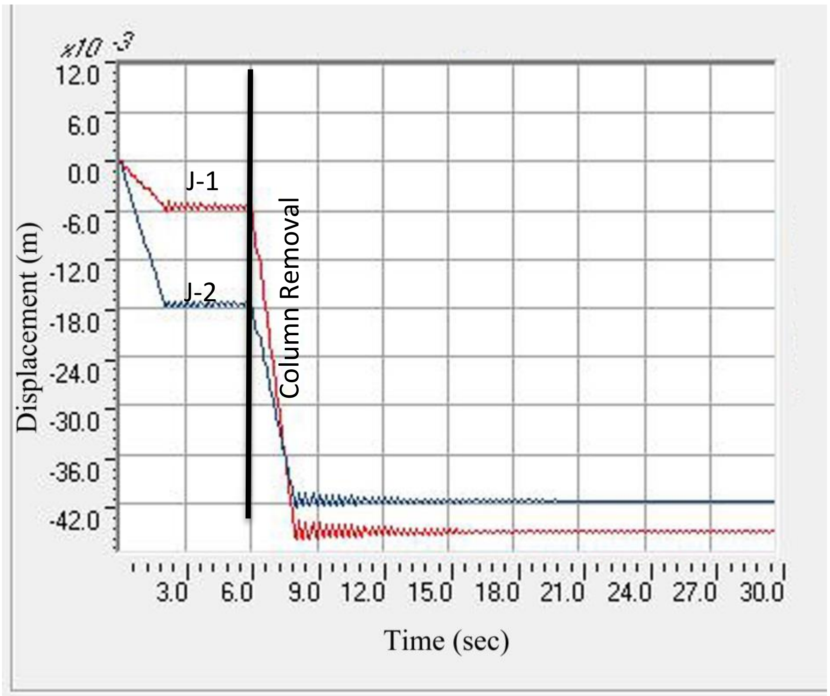


Figure 4.17: Vertical displacement of joints in the single span arch bridge with simple sections

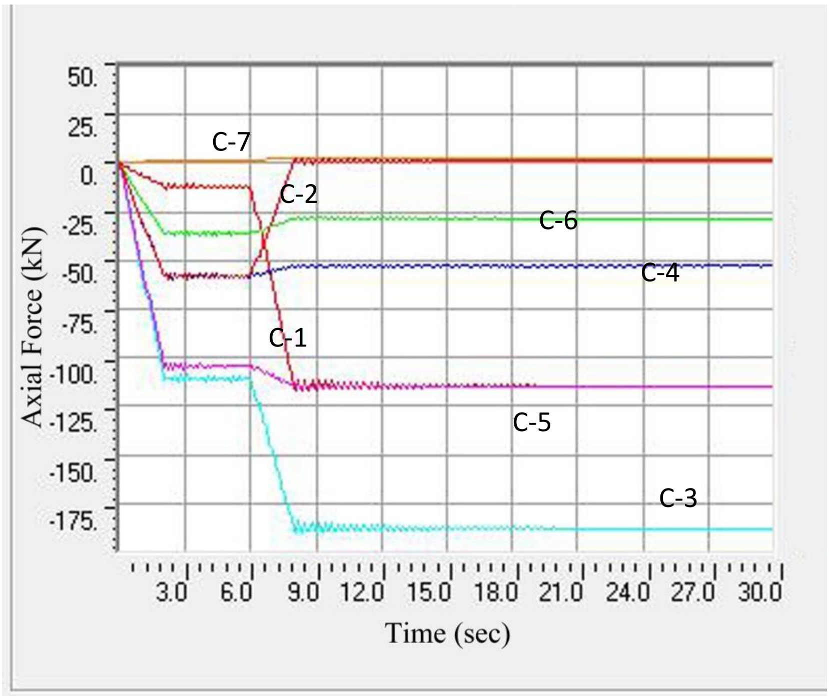


Figure 4.18: Column axial forces in single span arch bridge with simple sections

- Double span arch bridge

In this alternative one of the middle columns is removed to run the progressive collapse analysis. Figure 4.19 illustrates the deformed shape of the bridge at the end of the removal process. As it can be seen from Figure 4.20, joint J-3 has the most vertical displacement (8.57 cm). Column C-5 (the other middle column) was subjected to an axial force of 200.20 kN after column removal (Figure 4.21).

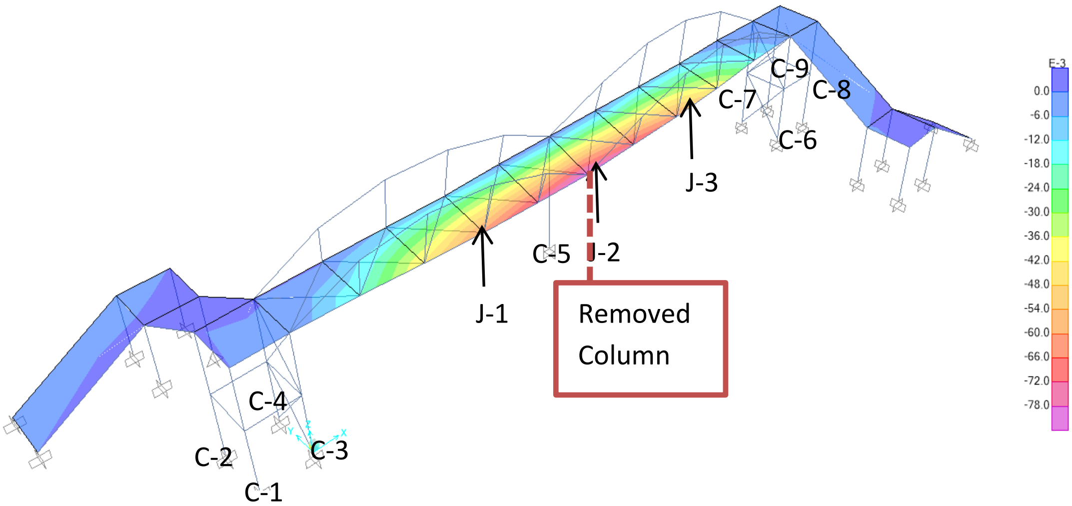


Figure 4.19: Deformed shape after column removal in double span arch bridge

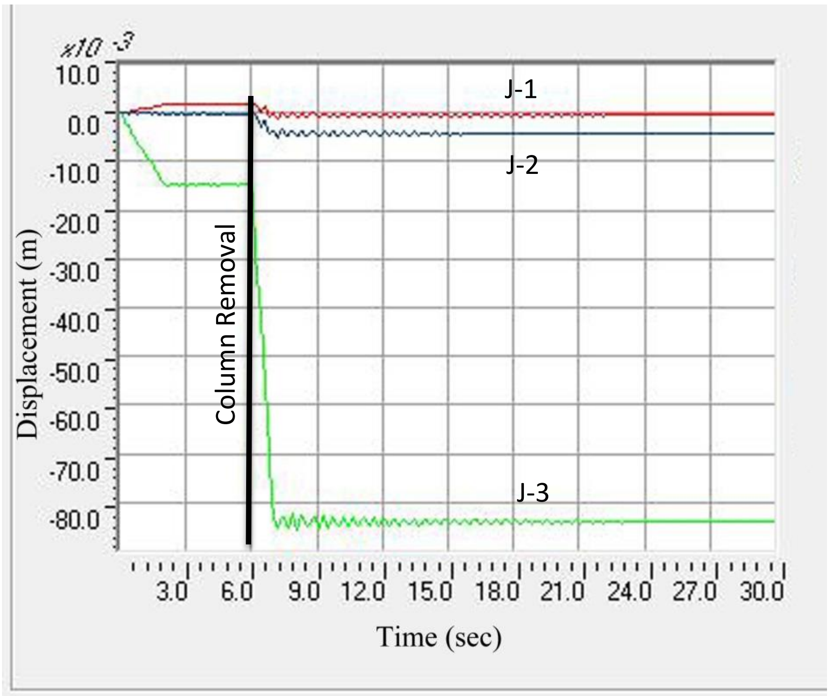


Figure 4.20: Vertical displacement of joints in double span arch bridge

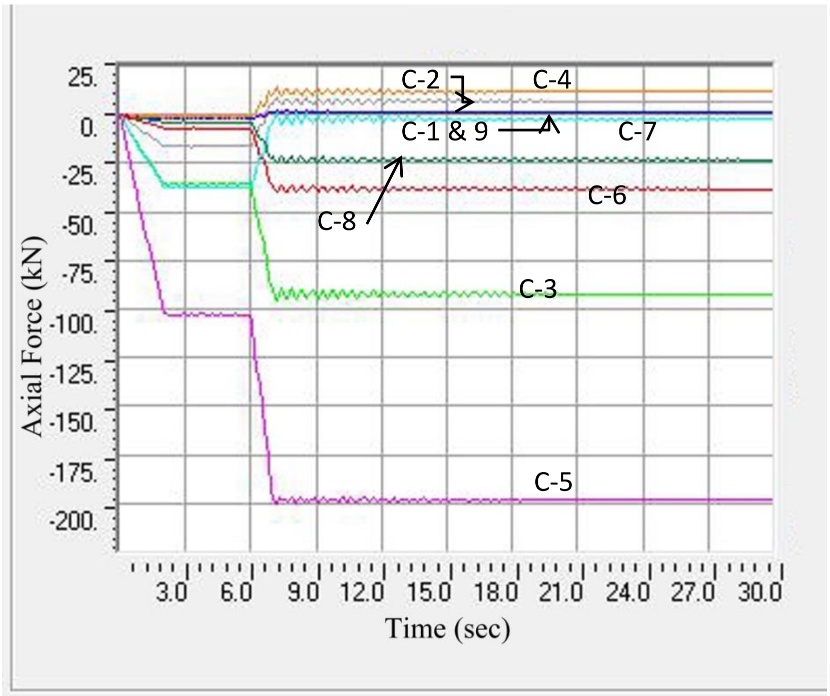


Figure 4.21: Column axial forces in double span arch bridge

- Single span truss bridge

Figure 4.22 shows which column is removed in this alternative. The deformed shape implies that joint J-2 has the maximum vertical displacement (4.41 cm, Figure 4.23). Maximum axial force was in column C-3, about 240.50 KN.

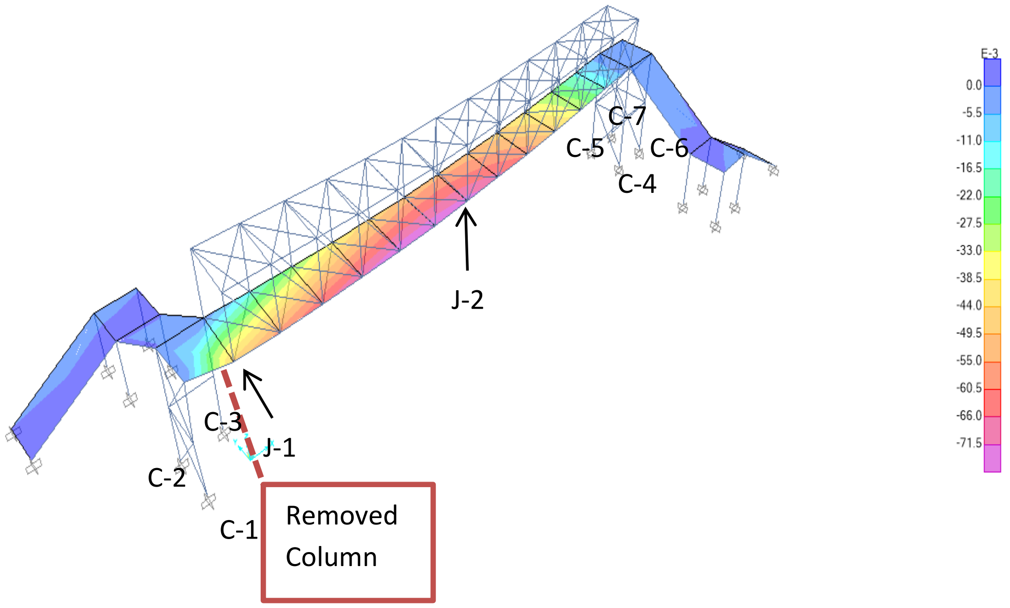


Figure 4.22: Deformed shape after column removal in single span truss bridge

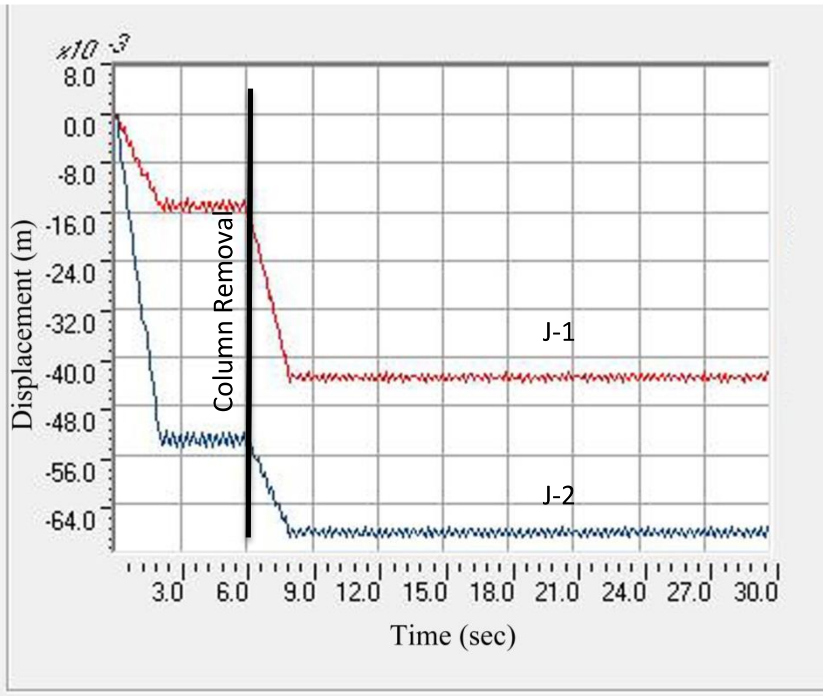


Figure 4.23: Vertical displacement of joints in single span truss bridge

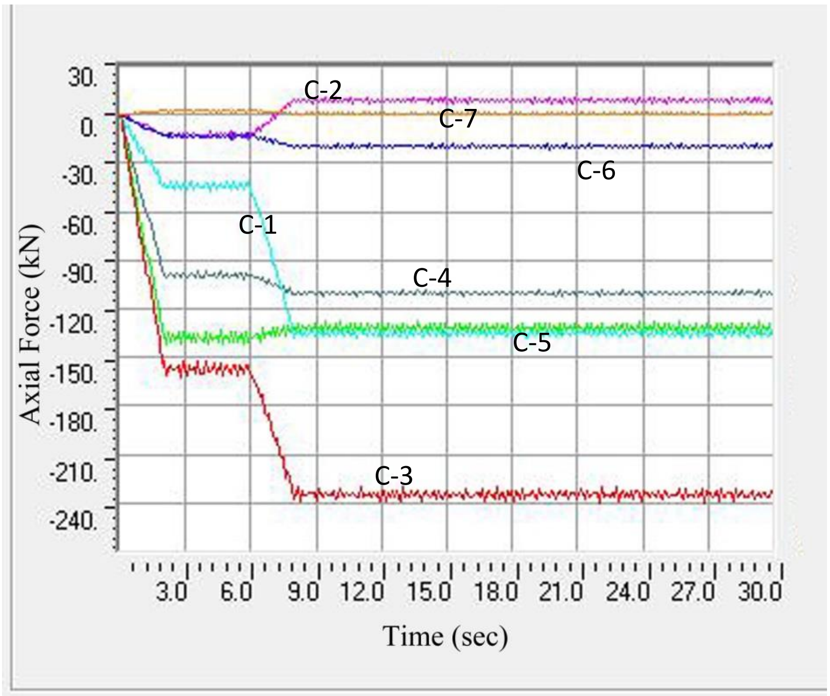


Figure 4.24: Column axial forces in single span truss bridge

- Double span truss bridge

In double span truss bridge alternative, one of the middle columns was removed and the deformed shape was as shown in Figure 4.25. The maximum vertical displacement occurred in joint J-1. Figure 4.26 shows that, the highest displacement was about 7.56 cm and the maximum axial force was about 261.00 kN in column C-5 (Figure 4.27).

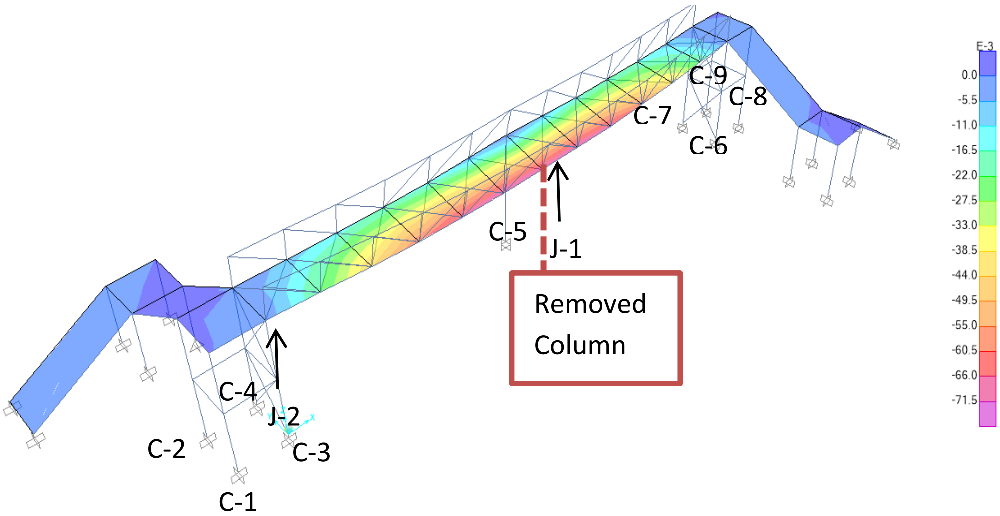


Figure 4.25: Deformed shape after column removal in double span truss bridge

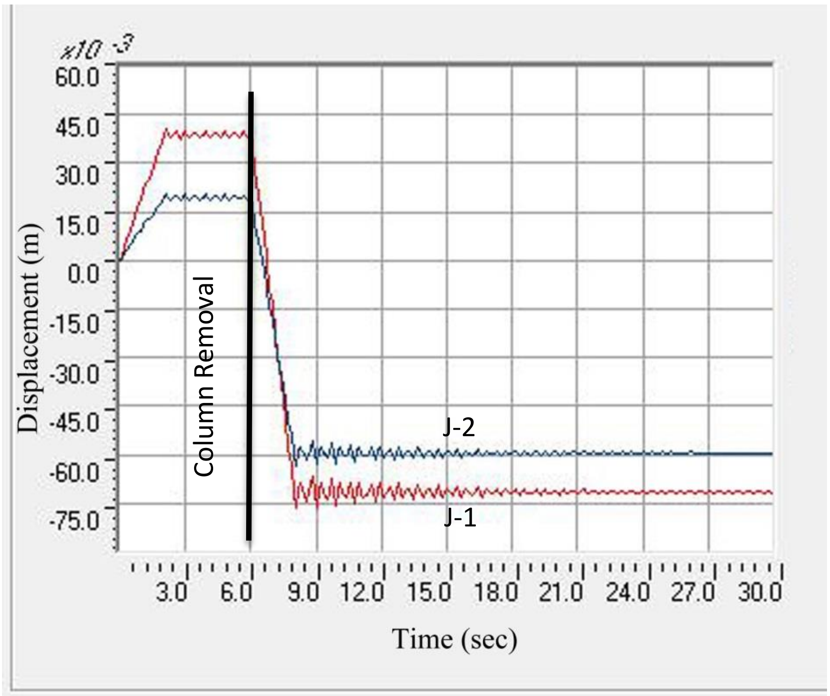


Figure 4.26: Vertical displacement of joints in double span truss bridge

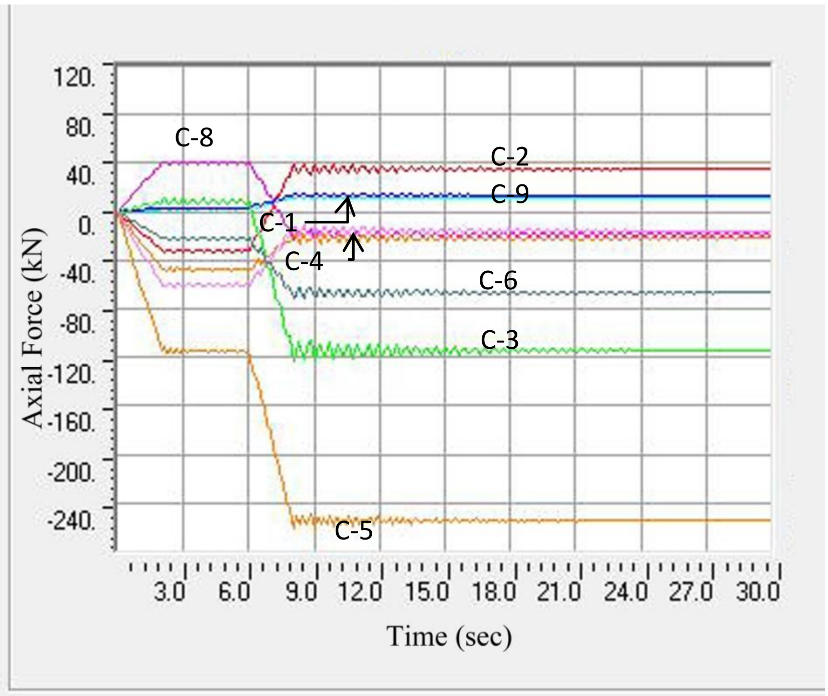


Figure 4.27: Column axial forces in double span truss bridge

- Single span truss bridge (capstone)

Figure 4.28 shows the deformed shape after column removal. The maximum displacement belongs to joint J-2 with 4.36 cm of deflection (Figure 4.29). The maximum axial force induced after column removal is about 149.00 kN for column C-3, which is the lowest maximum axial force among all alternatives (Figure 4.30).

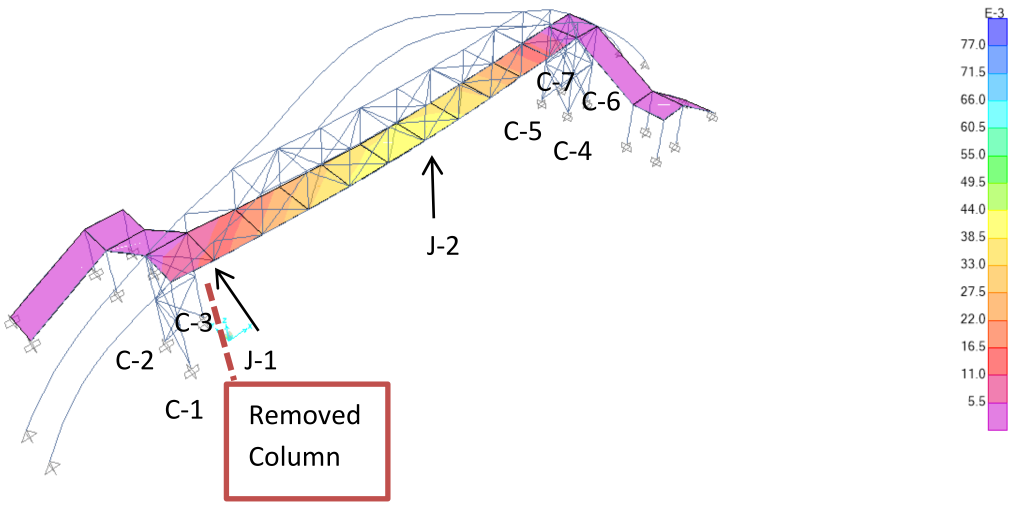


Figure 4.28: Deformed shape after column removal in single span truss bridge (Capstone project)

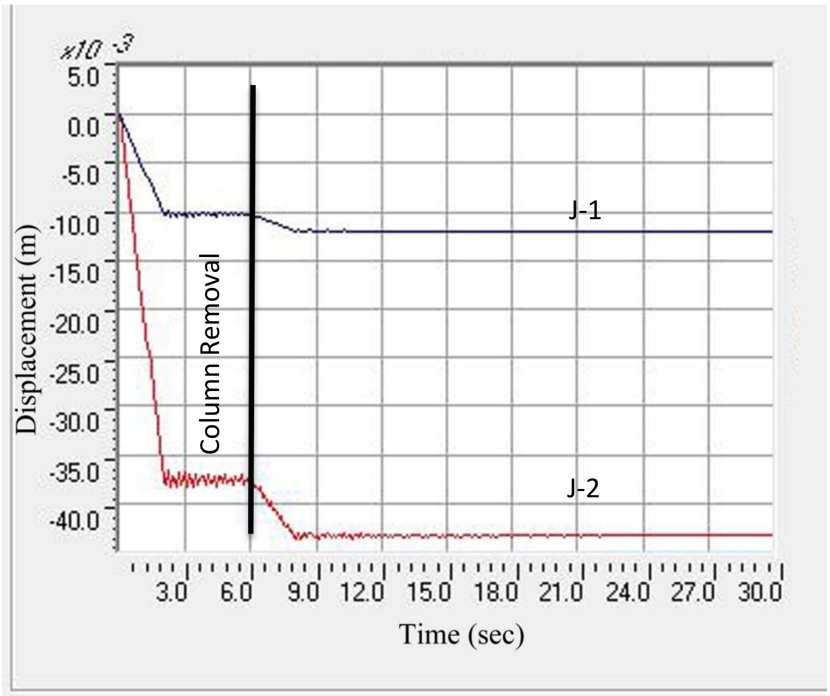


Figure 4.29: Vertical displacement of joints in single span truss bridge (Capstone project)

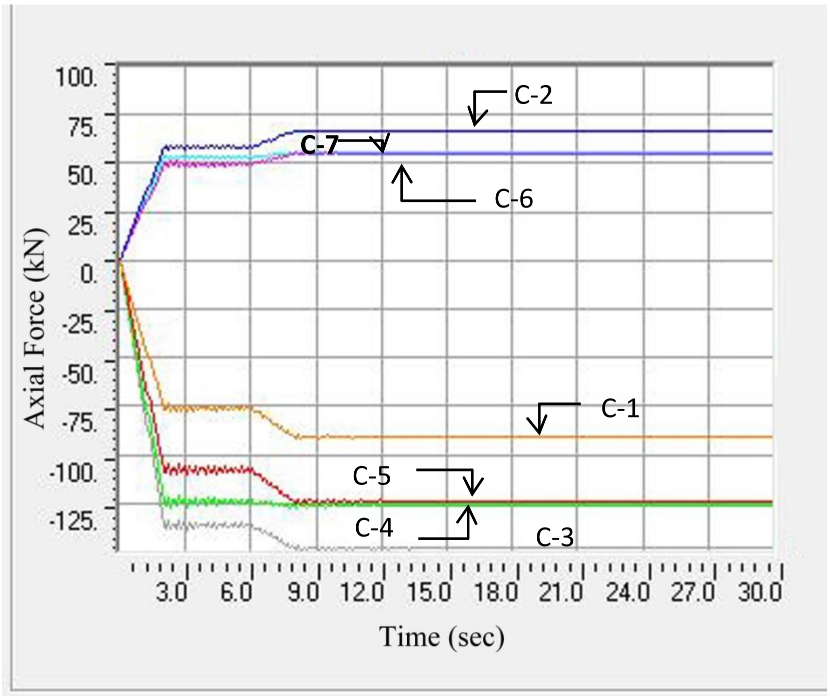


Figure 4.30: Column axial forces in single span truss bridge (Capstone project)

To make the comparisons easier the results are summarized in Table 4.10. As the steel sections used for columns in different alternatives are not identical. Therefore, using axial force for comparison may not provide reliable results. For this reason, the maximum compressive stresses induced to the columns were considered for the comparison of alternatives in a progressive collapse situation.

Table 4.10: Summary of the results of progressive collapse analyses

Alternative	**Maximum vertical displacement (cm) [score]**	**Maximum axial force (kN)**	**Maximum compressive stress (N/mm^2) [score]**	**Overall performance score [rank]***
Reference bridge	10.36 [1]	300.80	94.19 [6]	2.247 [6]
Single span arch bridge (variable section)	2.83 [7]	214.70	170.51 [3]	3.998 [4]
Single span arch bridge (simple section)	4.64 [4]	190.40	102.92 [4]	4.000 [3]
Double span arch bridge	8.57 [2]	200.20	396.58 [1]	1.249 [7]
Single span truss bridge	4.41[5]	240.50	69.99 [7]	5.499 [1]
Double span truss bridge	7.56 [3]	261.00	301.06 [2]	2.249 [5]
Single span truss bridge (based on Capstone project)	4.36 [6]	149.00	120.78[5]	5.349 [2]

* The overall scores are calculated using Ordered Weighted Averaging (OWA) aggregation operator. For more details see Appendix.

The results show that double span bridges experience more displacements than single span ones after column removal. The reason is that in these alternatives the middle column which bears the most internal loads in the primary model is removed in progressive collapse model. This removal induces more displacement to the bridge than the column removal in double span ones. Furthermore, higher axial forces occur in the members of single span bridges. According to the results of progressive collapse analysis single span truss bridge appears to be the best alternative. This bridge is followed by single span truss bridge (Capstone project). Double span arch bridge obtains the lowest score in these analyses (Table 4.10). In Table 4.10, the numbers in brackets show the ordinal score that alternatives have obtained according to their performances. In the last column the overall score for each bridge design is calculated, and accordingly, a rank is assigned.

4.2 Comparison of structures

As previously described, seven alternatives are designed in this research. However, each of them has its own advantages and disadvantages with respect to different aspects. The present section points out differences with regard to structural behavior, economical, material requirement, and aesthetical aspects.

4.2.1 Structural behavior

To evaluate the alternatives from the structural behavior point of view, the analysis results for deflection and progressive collapse analyses should be taken together into consideration. Considering the rankings obtained by each alternative, the final ranking is as the Table 4.11. It can be seen that, single span arch bridge has the best performance in terms of structural behavior, while the worst ranking is assigned to double span arch bridge.

Table 4.11: Ranking and scores of alternatives with respect to deflection and progressive collapse analysis results

Alternatives	**Ranking**		**Total Rank**
	Deflection	**Progressive collapse**	
Reference Bridge	1	6	4
Single span arch bridge (variable section)	6	4	5
Single span arch bridge (simple section)	3	3	1
Double span arch bridge	4	7	7
Single span truss bridge	5	1	2
Double span truss bridge	2	5	3
Single span truss bridge (based on Capstone project)	7	2	6

* The overall scores are calculated using Ordered Weighted Averaging (OWA) aggregation operator. For more details see Appendix.

4.2.2 Material usage

Recent advances in technology and construction techniques allow use of variety of materials. Material usage in any structure is of high importance. Firstly, the total weight of materials used for any structure, affects the structural behavior as well as the foundation design. Heavier structures require bigger foundations. The most important issue regarding material usage is usually cost. However, it should be noted that many resources and materials used in construction might have potential impacts on the environment (Liu et al., 2013). Construction industry is among top contributors to the environmental impacts such as greenhouse gas emissions, resource depletion, land change, waste production etc. Thus, material usage is important from not only economic point of view, but also environmental one. Table 4.12 summarizes the total weight of the whole structure, concrete and steel used. Table 4.12 shows that double span arch bridge has the lowest weight than the others and requires the minimum amount of steel and concrete. The reference bridge demands more materials among these seven alternatives.

Table 4.12: Material weight of the alternatives

Bridge Alternative	**Concrete (tonnes)**	**Steel* (tonnes)**	**Total (tonnes)**	**Rank**
Reference bridge	82.87	10.27	93.14	7
Single span arch bridge (variable section)	51.45	14.87	66.32	5
Single span arch bridge (simple section)	51.45	12.88	64.33	4
Double span arch bridge	51.45	10.23	61.68	1
Single span truss bridge	51.45	10.78	62.23	2
Double span truss bridge	51.45	11.2	62.65	3
Single span truss bridge (based on Capstone project)	51.45	18.12	69.57	6

* Includes reinforcement steel

4.2.3 Cost

Table 4.13 presents an estimation of total required budget for each alternative. Even though cost of some items may not be up to date due to fluctuations in North Cyprus market and foreign currency exchange but they would give a reasonably accurate idea on the cost comparison for the bridge types that have been investigated in this study. The cost estimation is based on material and labor costs obtained from the market investigation in North Cyprus. It should be noted that the cost of material purchase, fabrication, transportation to the site, erection and labor are included in these prices.

Cost of steel (including material purchase, fabrication, transportation to the construction site and erection) is approximately 3500-4000 TL/ton in Cyprus. In order to include the connections in steel price it is needed to increase the total cost of steel by about 20%. The price for concrete pouring is about 150 TL/m^3. The cost of formwork is considered about 20 TL/m^2.

As explained in chapter 3, total cost, in this study, is calculated by the following equation:

$$C_T = C_{dc} + C_r + C_{fw} + C_{sw} \qquad (4.2)$$

where, C_{dc}, C_{dr}, C_{fw}, C_{sw} are cost of concrete bridge deck (and stairs), deck (and stair) reinforcement, formwork and steelwork.

Table 4.13 shows the calculated values for each item. From Table 4.13, it can be seen that double span arch bridge could be constructed with lower budget than the other bridge types.

Table 4.13: Estimated costs (all values are in 1000TL)

Alternative	C_{dc}	C_{dr}	C_{fw}	C_{sw}	**Total Cost**	**Rank**
Reference bridge	12.43	9.30	8.70	60.05	90.48	5
Single span arch bridge (variable section)	7.72	3.42	6.00	74.35	91.48	6
Single span arch bridge (simple section)	7.72	3.42	6.00	64.40	81.53	4
Double span arch bridge	7.72	3.42	6.00	51.15	68.28	1
Single span truss bridge	7.72	3.42	6.00	53.90	71.03	2
Double span truss bridge	7.72	3.42	6.00	55.5	72.63	3
Single span truss bridge (based on Capstone project)	7.72	3.42	6.00	90.60	107.73	7

4.2.4 Aesthetics and appearance

Besides structural and economic aspects, appearance of the bridge and aesthetics are also important. The aesthetic aspects of pedestrian bridges are often neglected. These bridges bear comparatively small loadings and their widths are often narrow. This characteristic makes it easier and more flexible to design a footbridge (Yang and Huang, 1997).

Among bridge forms, arch is most natural one and it is considered as one of the most aesthetically appealing forms (MDT, 1995). Moreover, footbridges are slender structures and these kinds of structures with long spans are visually more attractive than deeper structures with short spans (Bridge Aesthetics Sourcebook, 2010). Therefore, aesthetically speaking, single span bridges are highly preferred to those with double spans. Thus, we can rank the alternatives in terms of aesthetics in Table 4.14. The first rank is assigned to the single span truss bridge (Capstone project) which has an architectural curve above the structure which contributes to its beauty and is a single span bridge.

Table 4.14: Alternative rankings from aesthetic point of view

Bridge Alternative	**Aesthetic score (Rank)**
Reference bridge	3 (5)
Single span arch bridge (variable section)	6 (2)
Single span arch bridge (simple section)	5 (3)
Double span arch bridge	4 (4)
Single span truss bridge	2 (6)
Double span truss bridge	1 (7)
Single span truss bridge (based on Capstone project)	7 (1)

4.3 Selection of the most appropriate alternative

In previous sections the best alternatives are selected from different perspectives. However, it is important to adopt a final decision regarding the most suitable alternative. If we assume that alternative selection is a decision making problem in presence of different decision makers with various preferences and criteria, then, this selection problem can be considered as a group decision making (GDM) problem. Then, this group decision making will have four decision makers:

1. Structural behavior decision maker, who prefers the alternative with best structural performance,
2. Material usage decision maker, who prefers the design with minimum material requirement,
3. Cost decision maker, who seeks the alternative that requires minimum budget,
4. Aesthetic decision maker, who looks for the most aesthetically appealing design.

The objective of these four decision makers might be in conflict with each other. For instance, choosing the most aesthetically attractive design may require higher budgets. Nevertheless, they need to attend a decision making session and bargain over the alternatives, until they select the one which satisfies them to proximate levels.

To solve a group decision making problem there are different tools to use. One of the recently developed methods is Fallback Bargaining (FB) (Brams and Kilgour, 2001).

4.3.1 Decision making using Fallback Bargaining

FB was introduced by Brams and Kilgour (2001) and it is a bargaining approach that predicts the bargaining outcome in a decision making session. In a decision making session decision makers or bargainers pursue their own goals and objectives and each one insists on their most preferred alternative. However, their preferences are usually not in accordance with others and they may have conflicts with each other. For this reason, reaching a compromise requires falling back from the most preferred alternative to the less preferred ones until all decision makers or bargainers have an agreement on it and give sufficient support. The outcome of FB is a subset of alternatives called Compromise Set.

There are different branches for Fallback Bargaining:

- Unanimity Fallback Bargaining: this method determines the alternative(s) with unanimous support at highest possible depth of bargaining. This method maximizes the minimum satisfaction among bargainers.
- *q*-Approval Fallback Bargaining: this method allows for selection of an alternative which receives support of q number of bargainers at the highest possible level.
- Fallback Bargaining with Impasse: in this method bargainers could determine an impasse in their rankings. This means that bargainers would prefer not to make any agreement on decision below the impasse level.

In order to solve this decision making problem, four decision makers or bargainers are considered. Each bargainer is a representative of one aspect (structural behavior, material usage, cost, and aesthetics) and pursues its own objectives to maximize the outcome from the final decision. As in this study the satisfaction of all bargainers (aspects) is important, unanimity fallback bargaining is utilized.

Firstly, it is needed to form the decision making matrix:

$$
\begin{array}{ll}
\mathit{Structural\ Behaviour} & \rightarrow \\
\mathit{Cost} & \rightarrow \\
\mathit{Material\ usage} & \rightarrow \\
\mathit{Aesthetics} & \rightarrow
\end{array}
\begin{bmatrix}
\boxed{A_3} & A_5 & A_6 & A_1 & A_2 & A_7 & A_4 \\
A_4 & A_5 & A_6 & \boxed{A_3} & A_1 & A_2 & A_7 \\
A_4 & A_5 & A_6 & \boxed{A_3} & A_2 & A_7 & A_1 \\
A_7 & A_2 & \boxed{A_3} & A_4 & A_1 & A_5 & A_6
\end{bmatrix}
$$

where:

A_1 is the reference bridge

A_2 is single span arch bridge (with variable section)

A_3 is single span arch bridge (with simple section)

A_4 is double span arch bridge

A_5 is single span truss bridge

A_6 is double span truss bridge

A_7 is single span truss bridge (capstone project)

In the above matrix, each raw represents that ranking order of each bargainer. As can be seen in the matrix, A_3 is the first preference of the first bargainer, the third preference of the fourth bargainer, and finally, the fourth preference of the second and third bargainers. Therefore, until the fourth level of bargaining, A_3 is among top preferences of all bargainers and can be considered as the most appropriate choice which satisfies all aspects at highest possible level.

Thus, single span arch bridge with simple sections is selected as the best alternative. This design option shows relatively satisfactory structural behavior in terms of deflection and progressive collapse and earns the first rank. Regarding cost and material usage, it is the fourth, and from aesthetical point of view it follows the single span arch bridge with variable sections.

The Fallback Bargaining method assigns equal weights to the bargainers. This means that structural behavior, cost, material usage and aesthetics have equal weights of importance. However, in real situations the importance of these criteria may vary according to the project requirements. For this reason, another approach which could assign different power weights to the bargainers is employed in the next section.

4.3.2 Decision making by assigning power weights to the bargainers

In reality, when it is decided to construct a bridge, the structural behavior may have more significance than other parameters, such as, aesthetics. The purpose of this section is to solve the problem under the circumstance in which the parameters or bargainers have different power weights. For this aim, the approach proposed by Zarghami (2011), which is based on the OWA operator (called Borda-OWA), is used. This approach has the capability of assigning power weights to the bargainers. The steps of this method are as follows:

Step 1) The ranking of alternative *i* from the view point of bargainer *j* is determined for *n* number of bargainers (r_{ij}). For the current problem we have:

$$
\begin{array}{c} \\ A_1 \\ A_2 \\ A_3 \\ A_4 \\ A_5 \\ A_6 \\ A_7 \end{array}
\begin{array}{c} \begin{array}{cccc} SB & C & MU & A \end{array} \\ \begin{bmatrix} r_{11} & r_{12} & r_{13} & r_{14} \\ r_{21} & r_{22} & r_{23} & r_{24} \\ r_{31} & r_{32} & r_{33} & r_{34} \\ r_{41} & r_{42} & r_{43} & r_{44} \\ r_{51} & r_{52} & r_{53} & r_{54} \\ r_{61} & r_{62} & r_{63} & r_{64} \\ r_{71} & r_{72} & r_{73} & r_{74} \end{bmatrix} \end{array}
=
\begin{array}{c} \\ A_1 \\ A_2 \\ A_3 \\ A_4 \\ A_5 \\ A_6 \\ A_7 \end{array}
\begin{array}{c} \begin{array}{cccc} SB & C & MU & A \end{array} \\ \begin{bmatrix} 4 & 5 & 7 & 5 \\ 5 & 6 & 5 & 2 \\ 1 & 4 & 4 & 3 \\ 7 & 1 & 1 & 4 \\ 2 & 2 & 2 & 6 \\ 3 & 3 & 3 & 7 \\ 6 & 7 & 6 & 1 \end{bmatrix} \end{array}
$$

where, SB is structural behavior, C is cost, MU is material usage, and A is aesthetics.

Step 2) The value of each rank is reduced from the total number of alternatives (m) as $m - r_{ij}$. Then, we have:

$$\begin{array}{cc} & \begin{array}{cccc} SB & C & MU & A \end{array} \\ \begin{array}{c} A_1 \\ A_2 \\ A_3 \\ A_4 \\ A_5 \\ A_6 \\ A_7 \end{array} & \begin{bmatrix} 3 & 2 & 0 & 2 \\ 2 & 1 & 2 & 5 \\ 6 & 3 & 3 & 4 \\ 0 & 6 & 6 & 3 \\ 5 & 5 & 5 & 1 \\ 4 & 4 & 4 & 0 \\ 1 & 0 & 1 & 6 \end{bmatrix} \end{array}$$

Step 3) The values obtained in the previous step are normalized and rescaled to [0,1] using $(m - r_{ij})/m$. The decision matrix is changed as follows in this step:

$$\begin{array}{cc} & \begin{array}{cccc} SB & C & MU & A \end{array} \\ \begin{array}{c} A_1 \\ A_2 \\ A_3 \\ A_4 \\ A_5 \\ A_6 \\ A_7 \end{array} & \begin{bmatrix} 0.43 & 0.29 & 0.00 & 0.29 \\ 0.29 & 0.14 & 0.29 & 0.71 \\ 0.86 & 0.43 & 0.43 & 0.57 \\ 0.00 & 0.86 & 0.86 & 0.43 \\ 0.71 & 0.71 & 0.71 & 0.14 \\ 0.57 & 0.57 & 0.57 & 0.00 \\ 0.14 & 0.00 & 0.14 & 0.86 \end{bmatrix} \end{array}$$

Step 4) The normalized values are multiplied by the relative power weights of the bargainers (p_j) as $a_{ij} = \frac{p_j(m - r_{ij})}{m}$. In this problem it is assumed that the power weights for the bargainers of structural behavior, cost, material usage, and aesthetics are 3, 3, 3, and 1, respectively. Therefore, we have:

$$\begin{array}{cc} & \begin{array}{cccc} SB & C & MU & A \end{array} \\ \begin{array}{c} A_1 \\ A_2 \\ A_3 \\ A_4 \\ A_5 \\ A_6 \\ A_7 \end{array} & \begin{bmatrix} 1.28 & 0.85 & 0.00 & 0.28 \\ 0.85 & 0.42 & 0.85 & 0.71 \\ 2.57 & 1.28 & 1.28 & 0.57 \\ 0.00 & 2.57 & 2.57 & 0.43 \\ 2.1 & 2.14 & 2.14 & 0.14 \\ 1.71 & 1.71 & 1.71 & 0.00 \\ 4.28 & 0.00 & 0.42 & 0.85 \end{bmatrix} \end{array}$$

Step 5) The aggregated score of the alternative i is calculated using the following equation:

$$S_i = \frac{\sum_{j=1}^{n} a_{ij}(1-a_{ij})^{\frac{1}{\beta}-2}}{\sum_{j=1}^{n}(1-a_{ij})^{\frac{1}{\beta}-2}} \text{ (for the values of } \beta \text{ see the Appendix)} \tag{4.3}$$

By applying the above-mentioned equation to each raw of the matrix obtained in the previous step, the final matrix containing scores will be as:

$$\begin{array}{c} \\ A_1 \\ A_2 \\ A_3 \\ A_4 \\ A_5 \\ A_6 \\ A_7 \end{array} \begin{array}{c} Score \\ \begin{bmatrix} 0.61 \\ 0.71 \\ 1.43 \\ 1.39 \\ 1.64 \\ 1.28 \\ 0.43 \end{bmatrix} \end{array} \rightarrow \begin{array}{c} Rank \\ \begin{bmatrix} 6 \\ 5 \\ 2 \\ 3 \\ \boxed{1} \\ 4 \\ 7 \end{bmatrix} \end{array}$$

The results indicate that A5 (single span truss bridge) is the alternative that is more satisfactory and has better performance when the structural behavior, cost, and material usage have power weights three times more than aesthetics.

Chapter 5

CONCLUSION AND RECOMMENDATION FOR FUTURE WORK

5.1 General summary

This research is about investigation, evaluation, and comparison of different design alternatives for a footbridge crossing over Nicosia-Famagusta main road between the North and South Campuses of Eastern Mediterranean University. Footbridges are usually slender structures with relatively small loadings, aesthetically important, should be in harmony with the surroundings, provide a safe passage for pedestrians and of course should be cost-effective like any other construction project.

The main objective of this study was to investigate seven design alternatives consisting truss and arch bridges (single span and double span) and then decide on the most suitable alternative with regards to structural behavior, material usage, cost and aesthetics. The alternatives were designed and then modeled and analyzed in SAP 2000 v.16.0.0 Ultimate and then studied under different aspects.

5.2 Conclusions

The following are the conclusions drawn for the investigation of seven truss and arch footbridges considering structural behavior, material usage, cost and aesthetics.

1) Structural behavior:

   Deflection is one of the important checks during the design process of footbridges. Horizontal and vertical deflection of the design alternatives were checked according to AASHTO guidelines. The results showed that double span truss bridge has the best performance in terms of deflection. Moreover, because of the location of the studied bridge is over a road and risk of collision loads are high a progressive collapse analysis was conducted to study the behavior of the structures. For this purpose, the most critical column of each alternative was removed and the analysis was done. The results of progressive collapse analysis highlighted that single span arch bridge with simple sections has the best performance.

2) Material usage:

Material usage is important not only due to economic aspects, but also environmental issues. The less the material usage is, the less the environmental damage will occur. The material weight obtained from the SAP2000 software output (which excludes the connection material) shows that single span arch bridge requires less material than the other six alternatives.

3) Cost:

Cost is one the main factors, based on which the successful execution of a construction project is evaluated. In this thesis, cost of the bridge construction was defined as the summation of cost of material purchase, fabrication and manufacturing, transportation to the construction site, installation, and deck and column concrete. Cost estimation results showed that double span arch bridge is more cost-effective than the other alternative designs.

4) Aesthetics:

For a pedestrian bridge, aesthetics is very important. Footbridges offer flexibility to form. According to aesthetical considerations, arch is among the forms that effectively contribute to visual attractiveness. In addition, longer spans are more visually appealing. From this perspective, the single span truss bridge (capstone project) which has an arch element seems to be more attractive than other alternatives.

5) Overall performance:

   To determine which alternative has the best overall performance under the all aforementioned aspects, the Fallback Bargaining method was used. To utilize this method each aspect is considered as a bargainer to bargain over the alternative selection problem. The outcome of the Fallback Bargaining introduced the single span arch bridge with simple sections as the most appropriate alternative. Furthermore, the Borda-OWA method is used to give power weights to the bargainers. The results showed that when the structural behavior, cost, and material usage have a power which is three times more than the power of aesthetic, the single span truss bridge is the most appropriate alternative.

5.3 Recommendation for future work

Research on bridge structural evaluation is very wide and variety of analyses and method developments can be done in this regard. As further works, the following can be pursued.

- Different types of decks, for example cellular deck and box deck and can also be evaluated along with superstructures.
- The design of a pedestrian bridge can be formulated as an optimization problem considering different objective functions like structural behavior, cost, etc.

- The structural evaluation of pedestrian bridges can be done based on different design guidelines like Ontario Highway Bridge Design Code, British Standard, etc.
- A systematic decision making algorithm can be developed in order to select the most suitable bridge design.

REFERENCES

AASHTO, L. (2008). Bridge design specifications, customary US units, with 2012 interim revisions. American Association of State Highway and Transportation Officials, Washington, DC.

ACI 318 Committee. (2011). Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary. In American Concrete Institute (p. 503).

American Society of Civil Engineers (ASCE), 2010. Minimum Design Loads for Buildings and Other Structures, ASCE-7.10.

Astaneh-Asl, A. (2008, June). Progressive collapse of steel truss bridges, the case of I-35W collapse. *In Proceedings of 7th International Conference on Steel Bridges*, Guimarăes, Portugal

Bahmani, S., and AghajaniNamin, A. (June, 2010). Design of a Footbridge on Nicosia-Famagusta, Famagusta-Karpaz Road. Capston Project (undergraduate level project), Department of Civil Engineering, Eastern Mediterranean University. Gazimağusa, North Cyprus.

Bayraktar, A., Altunişik, A. C., Sevim, B., & Türker, T. (2009). Modal testing, finite-element model updating, and dynamic analysis of an arch type steel footbridge. *Journal of Performance of Constructed Facilities*, 23(2), 81-89.

Brams, S. J., & Kilgour, D. M. (2001). Fallback bargaining. *Group Decision and Negotiation* 10(4),287–316.

Buscemi, N., & Marjanishvili, S. (2005). SDOF model for progressive collapse analysis. In Metropolis and Beyond.

CEN (2007) National Annex to Eurocode 8, design ofstructures for earthquake resistance-part 1: generalrules, seismic actions and rules for buildings, CYS EN 1998-1:2005. Comite European de Normalisation, Brussels; in: Cagnan, Z., and Tanircan, G. B. (2010). Seismic hazard assessment for Cyprus. *Journal of seismology*, 14(2), 225-246.

Cheng, J. (2010). Optimum design of steel truss arch bridges using a hybrid genetic algorithm. *Journal of Constructional Steel Research*, 66(8), 1011-1017.

Chen, Z., Cao, H., Zhu, H., Hu, J., & Li, S. (2014). A simplified structural mechanics model for cable-truss footbridges and its implications for preliminary design. *Engineering Structures*, 68, 121-133.

CSI 2014, http://www.csiamerica.com/products/sap2000

Davison, B., & Owens, G. W. (Eds.). (2011). Steel designers' manual. John Wiley & Sons.

Department of Defense DoD. 2005. “Unified facilities criteria UFC, design of buildings to resist progressive collapse.” UFC 4-023-03, DoD.

Erhan, G. (2009). “Assessment of Liquefaction/Cyclic Failure Potential of Tuzla Soils”. Master’s Thesis, Eastern Mediterranean University.

Ellingwood, B. R., & Leyendecker, E. V. (1978). Approaches for design against progressive collapse. *Journal of the Structural Division*, 104(3), 413-423.

Fu, G. (2013). Bridge Design and Evaluation: LRFD and LRFR. John Wiley & Sons.

GSA, (2003). “Progressive Collapse Analysis and Design Guideleines for New Federal Office Buildings and Major Modernization Projects”. General Services Adminisration.

Hartel, R. A., Amrheim, W.J., Willson, K. E., Baughman, D.R., & Tkacs, J.J. Bridge Inspector’s Training Manual/90, FHWA-PD-91-015, Federal Highway Administration, Washington, D.C., 1990.

http://www.nzta.govt.nz/resources/urban-design/principles/pedestrian-bridge/docs/urban-design-principles-pedestrian-bridges.pdf

Jagadeesh, T. R., & Jayaram, M. A. (2004). Design of bridge Structures. PHI Learning Pvt. Ltd.

Kaewkulchai, G. & Williamson, E. (2003). “Dynamic behavior of planar frames during progressive collapse”. 16th ASCE Engineering Mechannics Conference, July 16-18, 2003, Seattle, WA.

Kassimali, A. (2009). Structural analysis. Fourth Edition. Stamford, CT: Cengage Learning.

Kim, H. (2006). Progressive collapse behavior of reinforced concrete structures with deficient details. ProQuest.

Kim, H. S., Kim, J., & An, D. W. (2009). Development of integrated system for progressive collapse analysis of building structures considering dynamic effects. Advances in Engineering Software, 40(1), 1-8.

Kurowski, P. (2014). Vibration Analysis with SolidWorks Simulation 2014. SDC Publications.

Lewis, W. J. (2012). A mathematical model for assessment of material requirements for cable supported bridges: Implications for conceptual design. Engineering Structures, 42, 266-277.

Liu, S., Tao, R., and Tam, C. M., (2013). “Optimizing cost and CO2 emission for construction projects using particle swarm optimization” Habitat International, 37, 155-162.

Long, Y. Q., Cen, S., & Long, Z. F. (2009). Advanced finite element method in structural engineering. Tsinghua University Press.

Malekly, H., Meysam Mousavi, S., & Hashemi, H. (2010). A fuzzy integrated methodology for evaluating conceptual bridge design. Expert Systems with Applications, 37(7), 4910-4920.

Martí, J. V., & González-Vidosa, F. (2010). Design of prestressed concrete precast pedestrian bridges by heuristic optimization. Advances in Engineering Software, 41(7), 916-922.

Martí, J. V., Gonzalez-Vidosa, F., Yepes, V., & Alcalá, J. (2013). Design of prestressed concrete precast road bridges with hybrid simulated annealing. Engineering Structures, 48, 342-352.

Minnesota Department of Transportation: Aesthetic guidelines for bridge design, MDT, Minnesota 1995

Miyachi, K., Nakamura, S., & Manda, A. (2012). Progressive collapse analysis of steel truss bridges and evaluation of ductility. Journal of Constructional Steel Research, 78, 192-200.

National Research Council (US). Transportation Research Board. Subcommittee on Bridge Aesthetics, & Transportation Officials. Subcommittee on Bridges. (2010). Bridge Aesthetics Sourcebook: Practical Ideas for Short-and Medium-span Bridges. AASHTO.

O'Connor, C. (1971). Design of bridge superstructures. In: Ryall, M. J., Parke, G. A., & Harding, J. E. (Eds.). (2000). The manual of bridge engineering. Thomas Telford.

Proske, D., van Gelder, P. H. A. J. M., & Van Gelder, P. (2009). Safety of historical stone arch bridges (Vol. 359). Springer.

Rao, S. S. (2010). The finite element method in engineering. Butterworth-heinemann.

Ryall, M. J., Parke, G. A., & Harding, J. E. (Eds.). (2000). The manual of bridge engineering. Thomas Telford.

Sandovič, G., & Juozapaitis, A. (2012). The analysis of the behaviour of an innovative pedestrian steel bridge. *Procedia Engineering*, 40, 411-416.

Str′ask′y, J. (2005) Stress ribbon and cable-supported pedestrian bridges. Thomas Telford Ltd, London, UK.

Transportation Officials. (2009). LRFD Guide Specifications for the Design of Pedestrian Bridges. AASHTO.

Transportation Officials. Subcommittee on Bridges. (2009). Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals. AASHTO.

Troitsky, M. S. (1994). Planning and design of bridges. John Wiley & Sons.

Wai-Fah, C., & Lian, D. (2000). Bridge Engineering Handbook.

Wai-Fah, C., & Lian, D. (2014). Bridge engineering handbook, Second Edition: Superstructure Design"

Welch, J. F., Alhassan, M. A., & Amaireh, L. K. (2012). Analysis and Design of Arch-Type Pedestrian Bridge for Static and Dynamic Loads. *Journal of Advanced Science & Engineering Research*, 2(3).

Wolff, M., & Starossek, U. (2009). Cable loss and progressive collapse in cable-stayed bridges. Bridge Structures, 5(1), 17-28.

Xanthakos, P. P. (1994). Theory and design of bridges. John Wiley & Sons.

Yager RR. (1988) On ordered weighted averaging aggregation operators in multicriteria decision making. *IEEE Transactions on Systems, Man and Cybernetics*; 18(1): 183–190.

Yang, S., & Huang, D. (1997). Aesthetic considerations for urban pedestrian bridge design. *Journal of Architectural Engineering*, 3(1), 3-8.

Zarghami, M. (2011). Soft computing of the Borda count by fuzzy linguistic quantifiers. *Applied Soft Computing*; 11(1): 1067-1073, 2011.

Zarghami, M., & Szidarovszky, F. (2009) Revision the operator for multi criteria decision making problems under uncertainty. *European Journal of Operational Research*; 198(1): 259-265.

APPENDIX

Ordered Weighted Averaging Aggregation Operator

The Ordered weighted Averaging (OWA) operator which is developed and introduced by Yager (1988) is a soft aggregation operator (Zarghami 2011) and has been used in many multi-criteria decision making problems in various fields such as engineering. Soft aggregation operators have parametric behavior and provide the capability of defining compound selection conditions. According to Zarghami (2011), OWA is an *n*-dimensional operator, that assigns a goodness measure, *C*, to each alternative i in a decision making problem, while satisfies the following condition:

$$C_i^{min}(a_{i1}, a_{i2}, \dots, a_{in}) \leq C_i^{OWA}(a_{i1}, a_{i2}, \dots, a_{in}) \leq C_i^{max}(a_{i1}, a_{i2}, \dots, a_{in}) \quad (1)$$

Where $C^{OWA}: I^n \rightarrow I$ as follows:

$$S_i^{OWA}(a_{i1}, a_{i2}, \dots, a_{in}) = \sum_{j=1}^{n} w_j g_{ij} = w_1 g_1 + w_2 g_2 + \cdots + w_n g_n, (i = 1, 2, \dots, m) \quad (2)$$

where g_j is the *j*th largest element in the set of inputs $\{a_{ij}\}$ for alternative i, which belongs to a unit interval. Here, a_{ij} represents the score of alternative i under criterion *j* calculated in subsection 4.1. The coefficient w_j , are the order weights such that $w_j \geq 0$, $\sum_{j=1}^{n} w_j = 1$. It should be noted that these order weights are not in accordance with particular criteria, and they are relevant to an ordered situation (Zarghami, 2011).

The order weights are pertinent to the degree of risk acceptance in the decision making session. According to Zarghami (2011) the relationship between optimism degree, α, and the order weights is as follows:

$$w_j = \left(\frac{j}{n}\right)^{\left(\frac{1}{\beta}\right)-1} - \left(\frac{j-1}{n}\right)^{\left(\frac{1}{\beta}\right)-1} \tag{3}$$

The parameter β expresses the decision maker's attitude regarding the decision making problem. For example, when he wants to achieve the solution which provides satisfaction for all criteria, he may adopt the amount of 0.001 for β. The following table represents the values for α and the corresponding decision maker's attitude.

Table 1: Decision maker's attitude and the corresponding value of α

Decision maker's attitude (Satisfaction of criteria)	$\boldsymbol{\beta}$
All of them	0.001
Most of them	0.091
Many of them	0.333
Half of them	0.500
Some of them	0.667
Few of them	0.906
Any of them	0.999

In this study, as OWA operator is used to aggregate two criteria, 0.333 is considered to be a suitable value for β.