Dynamic Response of a Tied Arch Bridge to a Choice of Loading & Operation Conditions

A case study of the Urmia Lake Bridge

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Preface

This research has been carried out at the Division of Structural Design and Bridges, Department of Civil and Architectural Engineering, the Royal Institute of Technology (KTH) in Stockholm, Sweden.

The main idea for this study came up to my mind when I used to work as a design coordinator for Sadra, the EPC contractor of the Urmia Lake Causeway project. The Bridge was designed by a well known Norwegian bridge designer, Aas-Jakobsen, according to the state-of-the-art practices. We also did our best to build the bridge to the highest quality ever been achieved in Iran. Nowadays, the bridge is in the final stages of construction. However, I was always concerned of the prospective deviations in the utilization circumstances of the bridge. These could be, among others, exceeding the speed limits, overlooking the maintenance intervals, and overloading the bridge.

This study was conducted under supervision of Professor Raid Karoumi, who confirmed the idea and encouraged me for scrutinizing the possible problem areas of the bridge. I would like to express my sincere admiration and appreciation for his valuable advices and comments. I also wish to thank the management of Sadra and Aas-Jakobsen companies for their consent and granting me the authorization to extract input data from the design reports.

And Finally, I would like to thank my wife, Mozhgan, my daughter, and my Mother for their love, support and encouragements.

Stockholm, October 2009

Amir Mahan
Abstract

This unique arch bridge is the central part of the Urmia Lake Bridge in North West part of Iran. The bridge spans 100 meter and accommodates both road and railway traffic. A single track un-ballasted railway passes through arches and the outer cantilever parts are intended for double lane vehicle/road traffic in each direction. The main load carrying structure is made of steel box sections. A monolith 100 x 30 m free sliding concrete slab transfers traffic loads to steel structure through 134 numbers of buffer bearings.

In this study the dynamic behaviour of the bridge was analyzed to investigate its sensitivity to different loading conditions and different speeds. Consequently, dynamic amplification factors have been calculated directly from the results. For the purpose of scrutinizing the effects of bearings’ stiffness on dynamic response of the bridge, flexible and stiffer bearings were used in the model to simulate bearings condition if the maintenance period is overlooked. Finally, the effect of added mass, which is normally considered by bridge designers for the sake of stiffeners and connection plates in the steel structures, was investigated.

Most of the analyses have been performed using both mode superposition and direct integration methods with different time steps to compare the differences in analyses methods. In addition to the mode shapes and frequencies in every analysis, among all the results, deflections, accelerations, and bending moments in the bridge mid point and bridge lower chord have been studied. Moreover, for one-sided road traffic, bridge tilting and its torsion and bending moments were investigated.

From the results, the following conclusions are drawn. First, in the modal analyses high number of modes should be requested to get similar results to direct integration method. Time steps should be taken as short as possible. Second, Dynamic Amplification Factors of the bridge lay within a limited range when they are calculated in different ways. Third, the bridge is sensitive to stiffness of bearings and internal forces are increased insignificantly but, damping effect is irrelevant. Forth, there is a limitation for added mass to consider the results reliable otherwise; the maximum values of quantities will be missed in the design.

Keywords:
Arch Bridge, Dynamic Response, Dynamic Amplification Factor, Bearing’s Stiffness, Urmia Lake.
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1 Introduction

1.1 General

Design of the arch bridges is in principle influenced by two factors including geotechnical conditions and assembly methods. The geographical conditions and available facilities and machinery at every construction site play important roles in deciding the construction methods. Normally, these factors are determined prior or during the early stages of every bridge project and taken into consideration by designers.

Tied arch bridges have almost the lowest average moment and as well as lowest material consumption in comparison to other types of arch bridges. They are also of the greatest buckling capacity, which allows for narrower members and less weight for large spans. In poor ground conditions, e.g. soft clays, which is not solid enough to deal with the horizontal forces, the so-called tie beams are used to exert only vertical reactions at the bridge supports,[1]. In fact, instead of relying on the foundation to restrain the horizontal forces, the structure itself absorbs the loads by tying both ends of the arches together.

Arch bridges have also a tendency of flattening under loads. The tie beams that connect the arch chords prevent such an occurrence. However; under vertical loads, the lower chords tend to elongate in tension and the cross beams, which are attached to the lower chords, would move along with the points of attachment. Therefore; an area of concern is the tensile stresses in the concrete deck which need to be limited to prevent introducing cracks in highly stress areas, [2].

In steel arch bridges, hangers are installed vertically or inclined. In Nielsen-type steel arch bridges hangers are installed inclined and in two different directions. This arrangement substantially reduces bending moments in arch chords and the tie beams and thus, enables the arch and the deck to be made particularly slender, [3].

The bridge which is being introduced in this report, incorporate key aspects of the Urmia Lake Bridge. This Nielsen type tied arch bridge is the central part of the Urmia Lake Bridge in North West part of Iran. The bridge spans 100 meter and accommodates both road and railway traffic. A single un-ballasted railway track passes through arches and the outer cantilever parts are intended for double lane road traffic in each direction. The height of the arch bridge is 20.1 m from bottom axis. Arches are part of a circle with 72.2 m radius and inclined toward each other with a 5.7° angle. The main load carrying structure is made of steel box sections. A monolith 100 x 30 m free sliding concrete slab transfers traffic loads to steel structure through 134 numbers
of buffer bearings. The bridge deck has a width of 30 m and accommodates two traffic lanes with 7.30 m width on each cantilever side of the bridge, a 5 m railway path in the mid, and totally four shoulders which separated by median barriers.

A full description of the bridge is presented in Appendix A.

1.2 Aims and Scope

Structural analysis can be classified as static or dynamic. While statics deals with time-independent loadings, dynamics considers any load where the magnitude, direction, and position vary with time. Typical dynamic loads for a bridge structure include vehicular motions and wave actions such as winds, stream flow, and earthquakes, [8].

The main objective of this thesis is to study the dynamic behaviour of the tied arch bridges and scrutinize their sensitivity to a choice of vehicular motions at different speeds. The speed of moving loads has great influence on the dynamic response of bridges. Consequently, dynamic amplification factors are calculated directly from the results. In order to create a real and practical model, the geometry and sectional properties have been taken from the Urmia Lake Arch Bridge.

A number of other dynamically determinant factors have been also taken into consideration. For the purpose of scrutinizing effects of bearings’ stiffness on dynamic response of the bridge, flexible and stiffer bearings were used in the model to simulate bearings condition if the maintenance period is overlooked. Finally, the effect of added mass, which is normally accounted by bridge designers for the sake of stiffeners’ weight in the steel structures, was investigated.

Almost all analyses have been performed using both so-called “modes superposition” and “direct integration” methods with different time steps to compare the different analyses methods. In addition to the mode shapes and frequencies in every analysis, among all the results, deflections, accelerations, and bending moments in the bridge mid point and bridge lower chord have been plotted and compared. Moreover, for one-sided road traffic, bridge tilting and its torsion and bending behaviour were investigated.

The bridge loads have been limited to the self weight of structure, superimposed dead load from pavements and safety provisions installed on the bridge, moving truck load, and moving train load. Moreover, a moving point load has been used for basic modal studies of the bridge.

1.3 Thesis Outline

The present thesis comprises to nine chapters including the introductory Chapter. An appendix is also included to provide supporting data for model creation.
The introductory part summarizes the main features of the study bridge. It describes the basic idea and principle, gives a brief history of bridge, and discusses the aim and scope of the thesis.

In Chapter 2, accuracy of the conducted studies has been discussed. The structural model, boundary conditions, vehicle types, and types of analyses have been described and compared to the actual design performed by the bridge designer.

In Chapter 3, effects of added masses in the design procedure have been investigated. Several contingency factors have been assumed for steel weight for compensation of the weight of stiffeners and connection plates in the structure. Results have been compared and discussed.

In Chapter 4, stiffness of buffer bearings for floating slab have been changed to simulate malfunction of bearings in different conditions. This could be either due to damage to the bearings in early stages of bridge life or overlooking maintenance intervals in later stages. Results have been plotted and discussed.

In Chapter 5, a moving point load was run over the bridge to find out the required number of mode shapes for convergence of results in the modal analyses. Two different types of modal analyses were used by Eigenvectors and Ritz vectors. Finally, results have been compared to the results from direct integration method.

In Chapter 6, a full train ran over the bridge in two different speeds to simulate static and dynamic responses of the bridge. Results were obtained in both modal analyses (taking the outcome of Chapter 5 into consideration) and direct integration methods and were discussed.

In Chapter 7, the bridge was studied under train and truck loadings in different speeds so that they impose symmetric and asymmetric loads to the bridge. Bending and torsion behaviour of the bridge have been investigated using direct integration method. Six points in bridge mid section have been studied.

In Chapter 8, dynamic amplification factors of the bridge were extracted using direct integration method and applying several loading scenarios including; train, truck, and combined traffic all with different speeds.

In Chapter 9, effect of ballasted rail track on bridge behaviour has been studied to have an estimation of the bridge response to likely overloading in future. In this section direct integration method was used and damping effect of the ballast was taken into consideration.

Appendix A is the last part of this report. A summary of the actual design reports where were applicable for this study and necessary for creating an accurate model was provided.
Chapter 2. Accuracy of Analyses

2 Accuracy of Analyses

2.1 General

This report is a case study of an existing bridge and therefore, uppermost care has been taken into account to create a model almost similar to the designer’s one. Even though, there have been some limiting factors in making an exactly identical model.

In this chapter, specifications of the actual bridge are reviewed briefly and then key features of the study model are presented. The aim is to show how reliable the model is and how accurate are the results.

2.2 Vehicles and Structural Model

2.2.1 Vehicles

Of the 6 vehicle types in the actual design only two vehicle types have been used for this study; A 400 kN truck load for road way and a 1000 kN train (Locomotive) load for the railway, see Figures 2.1 and 2.2. However; various configurations of these loads with different running speeds were applied to the bridge in the current study. In simulation of moving loads the parts of loads which are in the form of uniform loads are disregarded.

The highway and railway loads have been taken from the AASHTO “Bridge Design Specification”, Edition 1998 including Interim 2002, and the ENV 1991-3, Eurocode 1-Part 3 “Traffic Loads on Bridges” respectively. The loading arrangements are in consistency with the Iranian regulations where almost the same loading arrangements have been adopted. In the Appendix A, section A.4.1 all the vehicle types, according to Iranian Regulations For Bridge Design have been described.

![Figure 2.1: Type number 1- Highway ordinary loads](image)
2.2.2 Bridge Model

The 3D beam program RM2000 has been used by the designer for steel structural model of the bridge. Sections were calculated by the program SECTION and controlled in MathCad for worst case scenarios. Moreover, the program STAAD PRO was utilized for structural modelling of concrete deck. In the slab model, the steel cross beams and lower chords were modelled and diagonal points were replaced by vertical springs with accurately calculated stiffness. The results from the slab model have been taken to steel structure model and applied as joint loads to account for the concrete slab in the real bridge. Whereas the bridge slab is free floating a concrete buffer has been used in one end to anchor the deck to the steel structure. This is required for earthquake and train braking events. Slab has got haunches over the steel cross beams where there are several numbers of pad bearings to guarantee free floating behaviour of the bridge deck. The bridge has been fully designed and checked against the AASHTO regulations. The steel plates which have been used in the design and, as well as in practice, are of grade S355N and concrete class is of type C55.

In the current study, to some extent similar structural concept has been utilized. The major difference is the number of structural models. Here, only one complete model has been created which seemed to be sufficient for intended study. All the structural members have been defined as frame elements and concrete deck segments were adapted to frame sections to have same weight and stiffness as for shell elements in a separate analysis, similar to what has been done by the designer. Concrete deck is connected to the steel structure using 134 numbers of springs, i.e. two springs on every cross beam. The stiffness of springs was taken from the existing design reports.
In creating the model, exactly same numbers of sections, members and joints have been defined. However, there exist some minor differences in geometry of sections which is not of high importance for this size of structure and the chosen areas of study. Boundary conditions are completely identical to the actual design and hence, there should not be a problem area for further discussion. The structural model which is created for the study bridge is illustrated in Figures 2.4 to 2.8.

**Figure 2.4:** Full FEM model for the Arch Bridge

**Figure 2.5:** Steel structure of the Arch Bridge in the FEM model

**Figure 2.6:** Concrete deck of the Arch Bridge in the FEM model
2.2. VEHICLES AND STRUCTURAL MODEL

For dead weights and superimposed dead load calculations similar unit weights have been used. However; among all other types of the loads, just two sets of vehicle loads were taken and studied. The major differences here are as follows:

- The chosen vehicles were run in different speeds and configurations,
- Unit weight of steel is un-factored (see Chapter 3 for more detail), and
- No specific load combinations and load factors were used.

Structural model was first made in the SAP2000 program but, finally, created and run in the program ABAQUS. The ABAQUS is a powerful engineering simulation programs, based on the finite element methods that can solve problems ranging from relatively simple linear analyses to the most challenging nonlinear simulations.
2.3 Analyses Methods

In the actual design, all load and load cases have been incorporated, where applicable, in various types of analyses including; static, dynamic, accidental and train derailment, fatigue, and construction phase analyses.

In the current study, from various types of analyses the linear static and the dynamic analyses have been carried out. Dynamic analyses limited to the time history analyses. To perform this type of analyses modal and direct integration methods have been utilized.

The time history method is a numerical step-by-step integration of equations of motion. It is usually required for critical, important or geometrically complex bridges, [8].

A modal analysis is always linear but, modal time history analyses could be linear or non-linear. They are carried out either based on eigenvector or Ritz vectors. An eigenvector analysis determines the un-damped free-vibration mode shapes and frequencies of the system. Ritz vector analysis seeks to find modes that are excited by a particular loading. Ritz vectors can provide a better basis than eigenvectors when used for time history analyses that are based on modal superposition, [9].

Totally, 52 sets of analyses in 9 different categories have been performed and results of selected joints and elements have been tabulated in Excel sheets for further discussions and deriving of conclusions. The results are mostly presented as various types of charts in this report.

2.4 Verification of the Model & Discussion

To have a comparison of the conducted study with the actual design work which has was performed by the designer; modal responses of the models have been compared. Moreover, a 2nd study model was created by eliminating the concrete deck elements from the main study model. The new model, steel-only arch bridge, was compared to the similar arch bridges which have been subject to study by researchers. Results indicate that there is less than 10% difference in the modal behaviour of the study model with existing literature.

The first 10 modal shapes for free vibration of the bridge are introduced in the Table 2.1. Moreover, some of the mode shapes have been illustrated in Figures 2.9 to 2.18.

Comparing the structural models, boundary conditions, load cases and analyses types indicate that the study model is sufficiently accurate and the results are reliable to make conclusions. The model also could be used for further studies.
Table 2.1: Modes shapes and eigenfrequencies for free vibration of the Arch Bridge

<table>
<thead>
<tr>
<th>Mode Number</th>
<th>Frequency, 1/s</th>
<th>Vibration Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.376</td>
<td>Transversal movement 1 in plan</td>
</tr>
<tr>
<td>2</td>
<td>0.595</td>
<td>Twist 1</td>
</tr>
<tr>
<td>3</td>
<td>0.870</td>
<td>Twist 2</td>
</tr>
<tr>
<td>4</td>
<td>0.890</td>
<td>Vertical movement 1</td>
</tr>
<tr>
<td>5</td>
<td>0.993</td>
<td>Transversal movement 2 in plan</td>
</tr>
<tr>
<td>6</td>
<td>1.147</td>
<td>Vertical movement 2</td>
</tr>
<tr>
<td>7</td>
<td>1.186</td>
<td>Twist 3</td>
</tr>
<tr>
<td>8</td>
<td>1.273</td>
<td>Transversal movement 3 in plan</td>
</tr>
<tr>
<td>9</td>
<td>1.392</td>
<td>Longitudinal movement 1</td>
</tr>
<tr>
<td>10</td>
<td>1.452</td>
<td>Vertical movement 3</td>
</tr>
</tbody>
</table>

Figure 2.9: Some of the mode shapes of the Arch Bridge
Figure 2.10: Mode shape no.1; movement in plan

Figure 2.11: Mode shapes no.2 & 3; twisting

Figure 2.12: Mode shape no.4; 1st vertical movement

Figure 2.13: Mode shape no.5; movement in plan
2.4. Verification of the Model & Discussion

Figure 2.14: Mode shape no.6; 2\textsuperscript{nd} vertical movement

Figure 2.15: Mode shape no.7; twisting

Figure 2.16: Mode shape no.8; movement in plan

Figure 2.17: Mode shape no.9; movement in longitudinal direction
Figure 2.18: Mode shape no. 10; 3\textsuperscript{rd} vertical movement
3 Effect of Added Mass

3.1 General

In creating structural models it is almost impossible to include all connectors, stiffeners, brackets, bolts & welds, and all other secondary type elements into the model. These types of elements are analysed and designed separately based on resultant forces from the main models. They could also be designed manually or modelled independently. If the connection points or faces to the main structural members are of high importance it will be possible to make new models and study the interactions locally. Usually, the main structural members are in the form of frame and shell or plate elements. Very accurate modelling techniques are required to make combined models of those elements and consequently that would need much more effort or CPU time to solve the problem.

In a large scale structure like a bridge hundreds of secondary members are used. Although, they are being ignored in the structural models but, their weights might have considerable effect on the final results. As a normal practice, to take care of all secondary members which are not included in the study models, a contingency factor is applied to the unit weights of the main structural elements. A rough estimation of such factors is done by designer based on simple calculation or experience. Depending on the type of bridge or industrial structure, importance of structures, material, accuracy of calculations, and so many other criteria which might be important a factor of 1.05 to 1.3 might be used.

It seems that the contingency factors would affect the dynamic behaviour of structures. When mass of a structure increases, its reflections and movements become sluggish. At the very first glance it appears to be affirmative to a more stable structure but, at the other hand, from structural design point of view, it means that the structure is supposed more stable than real state. In other word; dynamic behaviour of the structure is overlooked with this method. Taking the basic formula; \( F = m \cdot a \) into account, under a constant load, increasing the mass would diminish accelerations. If the mass is increased unrealistically then, the designer will count on smaller accelerations and other quantities as well. Consequently, he will underestimate the resultant forces.

In this part, unit weight of steel is increased by applying different factors and then, the bridge response is observed. The aim is to scrutinize the effect of added mass on the bridge response.
3.2 Analyses

A point load of 400 kN moves, over the rail track, on the bridge. The speed is 50 m/s. Modal time history analyses using time steps of 0.01 s is utilized to study this part. Since the aim is to have a rough estimation of the mass factors therefore, only 10 mode shapes based on eigenvectors are calculated. The analysis has been repeated 5 times for weight factors of 1, 1.1, 1.15, 1.20, and 1.25.

3.3 Results

The maximum values of deflections and accelerations in the mid point of bridge, joint No. 39, have been taken out of the results and illustrated in the below figures. Figures 3.1 and 3.2 show the differences in the results for 20% increment in the steel weight.

Figure 3.1: Bridge mid point deflection
(point load 400 KN, speed 50 m/s)

Figure 3.2: Bridge mid point acceleration
(point load 400 KN, speed 50 m/s)
3.3. RESULTS

To have a comparison of all the results they are also plotted for the absolute maximum values of deflections and accelerations for all the studied weight factors, see Figures 3.3 and 3.4.

![Graph of Deflection vs Steel Mass Factor](image1)

Figure 3.3: *Absolute maximum deflection at mid point (point load 400 KN, speed 50 m/s)*

![Graph of Acceleration vs Steel Mass Factor](image2)

Figure 3.4: *Absolute maximum acceleration at mid point (point load 400 KN, speed 50 m/s)*

As it could be seen in the Figures 3.3 and 3.4, up to 10% of increase in the mass, has negligible effect on deflections and accelerations but, the quantities unexpectedly fell for larger weight factors by 10% for the deflections and 50% for the accelerations.

Figure 3.5 shows how the bridge mode shapes affected by changes in the weight factors. In this figure only the first and the last mode shapes are illustrated.
3.4 Conclusion

Reviewing the results indicate that the bridge is sensitive to changes in the dead weight, as it is expected. However, it seems that there is a limit for variations in the bridge response to appear. For this bridge the limit is about 10% and it means that weight factor of 1.10 could be used without having any concern about resulting design loads.

For other types of bridges the same procedure might be carried out to have an estimation of the sensitivity limits. As it is seen from the figures, bigger factors will have considerable effects on final results and therefore, must be decided carefully. Moreover, the mode shapes indicate that vibrations and forces in some of the members will be overlooked. It should be noted that from dynamic point of view, 50% of differences in the accelerations means a different structural behaviour.

In sensitive structures like bridges, it would be worthwhile to have the weight of secondary members carefully estimated before applying a weight factor to the unit weights.

Figure 3.5: Bridge response to changes in dead weight
4 Effect of Bearings’ Stiffness

4.1 General

Elastomeric pad bearings are providing connection between the floating slab and steel structure. They are normally made from rubber, in rectangular or circular shape and serve bridges both as vertical bearing devices for service loads and as lateral isolation devices for horizontal loads. This requires that the bearings to be stiff with respect to vertical loads but relatively flexible with respect to lateral seismic loads. Sliding-type isolation bearings reduce the forces transferred from the superstructure to the supporting substructure. In the case of an earthquake excitations or huge horizontal loads they allow the superstructure to slide on a low friction surfaces which usually made from stainless steel-PTFE. The maximum friction between the sliding surfaces limits the maximum force that can be transferred by the bearing. The friction between the surfaces will also dissipate energy, [8].

In practice, elastomeric bearings are used as spring elements and carry many tons of mass. Along with a low spring stiffness of the elastomeric bearings the requested low tuning frequencies are achieved. The stiffness of elastomeric bearings increases with increasing frequency of the vibrations affecting them. This effect is called “Dynamic Hardening” and should be as small as possible, because otherwise the effectiveness of bearings with higher frequencies is impaired, [4].

Nowadays bearing suppliers are introducing maintenance free products but, it should be noted that severe environmental conditions, as applicable for the study bridge, most likely will change this criterion. Moreover, every bearing has its own service life and would need to be replaced at specified intervals. At the other hand, if an earthquake happens within the life time of bearings or should the bridge experiences an unusual horizontal load like train brake load, they might get excessive deformation. Therefore, basic properties of the bearings will be changed and consequently, the stability of the bearings and the superstructure they support would be at risk.

The bridge deck is supported by 134 numbers of sliding elastomeric pad bearings. The bearings have been defined in the bridge model as spring elements to take vertical service loads and to allow for slightly sliding of the deck in horizontal direction. The axial stiffness of every spring represents stiffness of bearing perpendicular to its plane and the shear stiffness of spring is an indicator of in-plane friction for that bearing.

Ideally, the sliding pad bearings shall be functional throughout service life of the bridge. This means that they must be inspected, cleaned and, where required, replaced according to the maintenance manual. The bridge is located in a high risk earthquake
region and the environmental condition is extremely harsh. At the other; to date, a comprehensive bridge maintenance program has not been developed by the local road authorities. Therefore, the risk of overlooking the maintenance intervals is high.

To scrutinize the adverse effects of stiffness changes on the dynamic behaviour of the bridge, a wide range of bearings stiffness have been studied. Included in this range, are some smaller factors which represent softening cases of bearings. In practice these cases may happen because of damage in the bearings. Some exaggerated factors have also been used to find limits of the variations.

### 4.2 Analyses

A combination of three vehicles including two trucks and a train has been considered for this analysis. They start to run over the bridge at the same time. Trucks are running with speed of 25 m/s (90 km/h) in two sides of arches, on the traffic lanes R1 and L1, in two reverse directions. The train moves with speed of 44.4 m/s (160 km/h) through arches on the rail track. Modal time history analyses using time steps of 0.05 s have been performed. Mode shapes have been calculated based on eigenvectors and are limited to 10 in number. This seems to be enough for this study as it is aimed to have a feeling of the change trend and a rough comparison of the results. This analysis was repeated 9 times for different coefficients of bearings’ stiffness; 0.2, 0.5, 1, 2, 5, 10, 20, 50 & 100 X.

### 4.3 Results

Fluctuation of deflections, accelerations, and bending moments are plotted against changes in the stiffness of bearings and illustrated in Figures 4.1 to 4.3. These graphs are valid for the mid point of bridge, joint No. 39.
As it could be seen from the Figure 4.1, for a wide range of stiffness, from 0.2 X to 100 X, the magnitude of deflections in the mid point of bridge varies from 0.004592 m to 0.003396 m. The most probable cases are 1 X and 10 X which corresponds to changes in deflections from 0.003786 m to 0.003516 m.
a)

![Graph showing acceleration versus time for different stiffness factors.]

b)

![Graph showing maximum accelerations for all stiffness factors.]

Figure 4.2: Bridge response to changes in the bearings stiffness;  
a) Accelerations versus time in the mid point for chosen stiffness factors  
b) Maximum accelerations in the mid point for all stiffness factors

In the Figure 4.2, for a wide range of stiffness, from 0.2 X to 100 X, the magnitude of accelerations in the mid point of bridge varies from 0.03265 m/s$^2$ to 0.02681 m/s$^2$. The most probable cases are 1 X and 10 X which corresponds to changes in deflections from 0.03244 m/s$^2$ to 0.02780 m/s$^2$. 
4.3. Results

Figure 4.3: Bridge response to changes in the bearings stiffness; 
a) Bending moments versus time in the mid point for chosen stiffness factors 
b) Maximum bending moments in the mid point for all stiffness factors

Figure 4.3 shows that, for a wide range of stiffness, from 0.2 X to 100 X, the magnitude of moments in the mid point of bridge varies from 694.8 kN.m to 450.4 kN.m. The most probable cases are 1 X and 10 X which corresponds to changes in deflections from 570.4 kN.m to 523.1 kN.m.

Variations of mode shape frequencies against changes in the stiffness of bearings have been illustrated in the Figure 4.4. This has been plotted for three mode shapes to show sensitivity degrees for them. Reviewing the graphs it is seen that the first mode shape frequency is largely affected by variation in the stiffness of bearings. Even tough, this is not negligible for the other mode shapes.
Chapter 4. Effect of Bearings’ Stiffness

Figure 4.4: Bridge response to changes in the bearings stiffness; Variation in mode shape frequencies

4.4 Conclusion

The results of this study are not applicable for main bearings of the bridge.

Dynamic hardening of sliding pad bearings has no major effect on deflections, accelerations, and bending moments of the bridge. However, softening which may occur in the damaged bearings will increase these quantities by 10% roughly.

Frequencies which are the main indicators of dynamic behaviour of bridge are largely affected by changes in the bearings stiffness. This would mean a different vibration pattern and magnitude for bridge members. In other word, the basic assumptions for designing of this bridge are changed to a large extent. Also, one could say that the bridge mid point is not a good point for this type of study.

Maintenance intervals including inspection of bearings should be carefully taken into account for important bridges like this bridge. Otherwise, bridge may experience different loads and deflections than those presumed in the design and thus major failures might be happened in the bridge.
5 Moving Point Load

5.1 General

In early stages of bridge projects usually simple models of structures are made to realize the possible problem areas. Analyzing large models are time consuming and difficult to check. However, simplification of models must not jeopardize the fundamental properties of structures and shall ensure that accurate results are obtained and evaluated.

Generally, a vehicle is modelled as a number of moving point loads over the bridge and the dynamic response of bridges are influenced by speed of moving loads. Among other things, accuracy of the results will depend on the time steps defined for a study.

“Moving point loads method” is a common approach to find dynamic response of a bridge to design traffic loads. In next sections, train and truck loads will be defined as series of moving point loads and bridge response to those loads at different speeds and time steps will be studied. Time history analyses based on mode superposition and direct integration methods will be utilized to calculate the bridge response. In mode superposition method, the accuracy of results will depend on the number of modes which are used for calculations. The more numbers of vibration modes would result in more accurate results.

Taking all of the aforementioned points into consideration, in this chapter a simple analyses model of the bridge using only a moving point load was created to find the required numbers of mode shapes for sufficiently accurate results. In fact, this chapter is a pre analyses stage for studying train and truck loadings on the bridge. In this part, modes superposition method based on Ritz vectors and eigenvectors are used to find the number of modes needed for convergence of the accelerations and deflections. The results are finally compared to the results from direct integration method.

5.2 Analyses

A moving point load of 400 kN with speed of 50 m/s passes over the bridge. A complete set of modal time history analyses using both types of eigenvectors and Ritz vectors have been conducted. In order to check the accuracy of the results, a set of time history analysis based on direct integration method has been also performed.
For the modal analyses the requested numbers of modes have been gradually increased to study convergence of the results. From the results it was found that minimum of 100 mode shapes are needed for very accurate values of deflections. The maximum number of mode shapes which have been studied was limited to 200. Based on the results, there was no significant difference between resultant deflections from 100 and 200 mode shapes. Almost the same conclusion is valid for the accelerations obtained from the modal analyses using Ritz vectors. However, the analyses using eigenvectors require more mode shapes to converge.

The loading effects have been studied for every time step of 0.01 s until the load leaves the bridge. Time steps were chosen based on a separate set of analyses which were carried out to ensure accuracy of the results. There have been 14 sets of analyses for this part of study totally.

### 5.3 Results

#### 5.3.1 Modal Analyses

Deflections and accelerations in the mid point of the bridge have been plotted versus time for several numbers of mode shapes, see Figures 5.1 to 5.4. Note that in the figures below E and R are representatives of Eigen and Ritz vectors and the numbers in the right indicate the total number of mode shapes utilized for calculations.

![Figure 5.1: Bridge mid point deflection using Ritz vectors](image-url)

*Figure 5.1: Bridge mid point deflection using Ritz vectors (point load 400 KN, speed 50 m/s)*
5.3. RESULTS

Figure 5.2: Bridge mid point deflection using Eigen vectors (point load 400 KN, speed 50 m/s)

Figure 5.3: Bridge mid point acceleration using Ritz vectors (point load 400 KN, speed 50 m/s)
Figure 5.4: Bridge mid point acceleration using Eigen vectors (point load 400 KN, speed 50 m/s)

In order to study convergence of the results the absolute maximum values of the deflections and accelerations from the modal analyses are taken and plotted in Figure 5.5.

Figure 5.5: Convergence of the results (point load 400 KN, speed 50 m/s)

a) absolute maximum values of deflections

b) absolute maximum values of accelerations
5.3.2 Direct Integration

Although it is not necessary to use direct integration method in this part but, it is carried out to have a feeling about range of variations and to check the accuracy of the obtained results. Figures 5.6 and 5.7 illustrate fluctuation of deflections and accelerations in the mid point of bridge versus time using direct integration method.

Figure 5.6: Bridge mid point deflection using DI method
(point load 400 KN, speed 50 m/s)

Figure 5.7: Bridge mid point acceleration using DI method
(point load 400 KN, speed 50 m/s)
5.3.3 Summary of Results

Summary of the Figures 5.1 to 5.7 are presented in the Table 5.1

<table>
<thead>
<tr>
<th>Analysis Method</th>
<th>Minimum number of Modes for convergence</th>
<th>Maximum Deflection, m</th>
<th>Maximum Acceleration, m/s²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modal Ritz</td>
<td>100*</td>
<td>0.00197</td>
<td>0.1041</td>
</tr>
<tr>
<td>Modal Eigen</td>
<td>100</td>
<td>0.00200</td>
<td>0.0781</td>
</tr>
<tr>
<td>Direct integration</td>
<td>-</td>
<td>0.00200</td>
<td>0.5051</td>
</tr>
</tbody>
</table>

* Acceleration calculated based on 200 mode shapes.

5.4 Conclusion

According to the results in the different parts of this chapter, it could be concluded that the accuracy of the calculations for a bridge response to a moving point load is increased by choosing large number of modes. This is especially of high importance for obtaining the accelerations. For the deflections it might be enough to consider limited numbers of mode shapes.

It is to mention that the modal analyses were continued up to 300 modes. According to the results, magnitude of the accelerations, using the eigenvectors, slightly increased but, still they were lower than the values obtained from the analyses using Ritz vectors in 200 modes. Deflections in both set of the modal analyses and the accelerations in Ritz vector based analyses do not show any significant difference between the results from 200 and 300 numbers of mode shapes. So, results up to combination of 200 modes have been presented in the figures.

The results indicate that 100 numbers of mode shapes are enough for convergence of the deflections. They are accurate enough when are compared to the results form direct integration method. However, more than 200 modes is needed for accelerations, in eigenvector based modal analyses, to be converged.

When results are being compared to results from direct integration method it is seen that the modal time history analyses based on Ritz vectors are relatively accurate than the analyses based on Eigenvectors.
6.1 General

Dynamic traffic load effects on bridges have increased, especially for railway bridges, as the result of increasing vehicle speed, increasing axle loads to improve transport efficiency, larger $M_{\text{vehicle}}/M_{\text{bridge}}$ ratio as the result of lighter and more slender bridges. Modern bridges, e.g. ballast-less railway bridges, have lower energy dissipation and consequently negligible damping ratios. The dynamic increment of traffic loads must be determined accurately in order to guarantee the planned life time and economical assessments, [6]. The bridge deflections, stresses and vertical accelerations roughly rise with increasing vehicle speed. The positions and amplitudes of local picks depend on the complex dynamic interaction of the bridge with the moving train, [5].

In this chapter the dynamic effects of train speed on bridge response are studied using different methods. Running the train in full design speed corresponds to the dynamic loading state. The results are compared to the static loading state from the train. Among all the results, deflections, bending moments, and accelerations in the mid point and maximum deflections along lower chord of the bridge are extracted from the outputs and plotted in various graphs. The aim is to find out the dynamic increment of different quantities and likely any abnormality.

6.2 Analyses

A train moves over the bridge very slowly with a speed of 1.39 m/s (corresponding to 5 km/h) and very quickly with speed of 44.4 m/s (corresponding to 160 km/h). Duration of loadings is 73.5 s and 2.3 s respectively. Two sets of time history analyses based on mode superposition method, using Ritz vectors, and also two more sets based on direct integration method have been conducted. Due to limited CPU capacity, time steps were set to 0.05 s and in the modal analyses the total number of the vibration modes was limited to 50 modes only. This was assumed to be accurate enough to study dynamic increment of the abovementioned quantities.

6.3 Results

Results including deflections, accelerations, and bending moments have been extracted from output files for mid point of the bridge, joint 39. For all the joints along lower
chord of the bridge only deflection were studied. Variation of abovementioned quantities are illustrated in Figures 6.1 to 6.4.

Figure 6.1: Bending moment at mid point under train load
MS method:
a) Speed 5 km/h, b) Speed 160 km/h
DI method:
c) Speed 5 km/h, d) Speed 160 km/h
6.3. RESULTS

Figure 6.2: Deflection at mid point under train load

MS method:
a) Speed 5 km/h, b) Speed 160 km/h

DI method:
c) Speed 5 km/h, d) Speed 160 km/h
Figure 6.3: Acceleration at mid point under train load

**MS method:**
a) Speed 5 km/h, b) Speed 160 km/h

**DI method:**
c) Speed 5 km/h, d) Speed 160 km/h
Figure 6.4: Bridge lower chord deflection under train load

**MS method:**
a) Speed 5 km/h, b) Speed 160 km/h

**DI method:**
c) Speed 5 km/h, d) Speed 160 km/h
As it could be seen in the Figures 6.1 to 6.4 there are considerable differences in the obtained results from different methods. Maximum values of bending moments and deflections for the mid point and lower chord are summarized in the Tables 6.1 to 6.3. Generally, dynamic amplification factors are calculated based on magnitude of deflections and bending moments. It was found that comparison of the accelerations could not lead us to a clear decision and therefore; it was ignored.

Table 6.1: Maximum bending moment at mid point under train load

<table>
<thead>
<tr>
<th>Bending Moment (kN.m)</th>
<th>MS Method</th>
<th>DI Method</th>
<th>Variation Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static</td>
<td>830.8</td>
<td>1041.1</td>
<td>1.25</td>
</tr>
<tr>
<td>Dynamic</td>
<td>925.7</td>
<td>1131.7</td>
<td>1.22</td>
</tr>
<tr>
<td>DAF</td>
<td>1.11</td>
<td>1.09</td>
<td>0.98</td>
</tr>
</tbody>
</table>

Table 6.2: Maximum deflection at mid point under train load

<table>
<thead>
<tr>
<th>Deflection (m)</th>
<th>MS Method</th>
<th>DI Method</th>
<th>Variation Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static</td>
<td>0.00395</td>
<td>0.00419</td>
<td>1.06</td>
</tr>
<tr>
<td>Dynamic</td>
<td>0.00423</td>
<td>0.00437</td>
<td>1.03</td>
</tr>
<tr>
<td>DAF</td>
<td>1.07</td>
<td>1.04</td>
<td>0.97</td>
</tr>
</tbody>
</table>

Table 6.3: Maximum deflection along lower chord under train load

<table>
<thead>
<tr>
<th>Deflection (m)</th>
<th>MS Method</th>
<th>DI Method</th>
<th>Variation Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static</td>
<td>0.00395</td>
<td>0.00412</td>
<td>1.04</td>
</tr>
<tr>
<td>Dynamic</td>
<td>0.00460</td>
<td>0.00518</td>
<td>1.13</td>
</tr>
<tr>
<td>DAF</td>
<td>1.16</td>
<td>1.26</td>
<td>1.09</td>
</tr>
</tbody>
</table>

6.4 Conclusion

In Chapter 5, it was found that, as expected, for a single moving point load the direct integration method gives more accurate results than mode superposition method. At the other hand, the modal time history analyses based on Ritz vectors is more accurate than analyses using eigenvectors. In this chapter, it is seen that for a series of moving point loads there is evidently 3 to 25% variation between the results of the two methods. The results from direct integration method are more crucial than those from mode superposition method.

The train speed has a great influence on magnitude of deflections, accelerations, and bending moments. It increases the deflections by about 5% and bending moments by about 10% in the mid point of the bridge.
Figure 6.3 illustrates variations of the accelerations for dynamic and static loading states using two different analyses methods. While dynamic increment of the accelerations in direct integration method is about 30%, the results from the modal analyses fail to present a reliable and clear picture. This could be because of insufficient number of the chosen mode shapes and time steps. So, any decision based on the accelerations is ignored.

Reviewing the deflections along the lower chord of the bridge indicate almost similar increments for each point under the dynamic loading state but, one can compare the maximum size of deflections in different points along the lower chord and consequently could expect large increments. As it is seen in Figure 6.4 and Table 6.3 both the analyses methods confirm larger increments for dynamic loading state along the lower chord. This approach is one of the several state-of-the-art methods for computation of dynamic amplification factors for bridges which will be discussed with more details in Chapter 8.
7 Bending & Torsional Behaviour

7.1 General

The three quantities which are being examined in this report, deflections, bending moments, and accelerations are continuously changing size and direction until the applied dynamic loads leave the bridge and it backs to a steady state. The study bridge is a huge and complex structure and can not be treated like a long single beam. The special geometry of bridge with extended cantilever load carrying members of the roadways encourages researchers for scrutinizing the bridge behaviour under different loading types. It seems that for this type of bridge structures evaluation of the quantities in a single point which, here is the mid point, could not be a proper and decisive criterion for bridge response. We may need to evaluate the bridge mid section entirely to be in the same level of assessments as for a mid point of a beam bridge.

Figure 7.1: Joints subject to study in the mid section of bridge

Figure 7.1 shows the mid cross section of the bridge. The joint no.39 which has already been examined for different loading and operation states is located on the lower chord. Five more joint have been shown in the figure. All the joints are chosen symmetrically. In this chapter the bridge response in all of the specified joints will be evaluated. Under a random and transient vehicle load the deflections, accelerations and bending moments are sometimes in the same direction and sometimes in opposite. Therefore, to have an imagination of bridge response pairs of symmetric points are evaluated with together and their average or divergence values are presented as the bridge mid section response to the applied loads. The loads are applied to the bridge in different speeds. Trucks are run over the lane L1 (see Figure 2.8 for lanes definition) to find the
maximum deviations of the symmetric joints on cantilever parts. The average values of quantities are known as bending values and the differing values are called torsional values. The torsional behaviour cause twisting or tilting of the bridge.

7.2 Analyses

For this part of study time history analyses using direct integration method has been utilized. Train was first run with crawling speed of 1.39 m/s (5 km/h) corresponding to the static state. Then the speed was increased to 44.4 m/s (160 km/h) to simulate the dynamic state of loading.

The same procedure has been followed for truck loading. Firstly, it was run with speed of 1.39 m/s (5 km/h) corresponding to the static state and secondly, with speed of 25 m/s (90 km/h) corresponding to the dynamic state. To have a feeling of the bridge response to trucks with higher speeds than the maximum allowed speed it was increased to 33.3 m/s (120 km/h). All the trucks are run over the lane L1 separately. The bridge behaviour was studied for every time steps of 0.01 s until the vehicle leaves the bridge. There have been 5 sets of analyses for this part of study totally.

7.3 Results

From the output files results of deflections, accelerations and bending moments for the study joints have been evaluated. The results were then processed further and tabulated in the Tables 7.1 to 7.10 in this chapter.

Average and differential values of all the quantities have been calculated from the absolute values of deflections in all the 6 joints. The results for deflections are presented in Tables 7.1 and 7.2. All values in these tables were calculated based on movements of the joints in same directions. For instance; maximum average value of deflections for joints 188 and 1107 represents average downward movement of those joints and the differential value indicates the differences in downward movements of those joints.

<table>
<thead>
<tr>
<th>Joint</th>
<th>Quick, V=44.4 m/s</th>
<th>Deflection, m</th>
<th>Slow, V=1.39 m/s</th>
<th>Joint</th>
</tr>
</thead>
<tbody>
<tr>
<td>39</td>
<td>1.6E-05 2.0E-06</td>
<td>Min 1.0E-04</td>
<td>7.0E-06</td>
<td>646</td>
</tr>
<tr>
<td></td>
<td>4.2E-03 1.3E-04</td>
<td>Max 4.1E-03</td>
<td>4.4E-05</td>
<td></td>
</tr>
<tr>
<td>188</td>
<td>7.3E-05 1.5E-05</td>
<td>Min 2.1E-04</td>
<td>4.6E-05</td>
<td>1107</td>
</tr>
<tr>
<td></td>
<td>4.5E-03 5.7E-04</td>
<td>Max 3.1E-03</td>
<td>1.4E-04</td>
<td></td>
</tr>
<tr>
<td>380</td>
<td>2.8E-05 1.0E-05</td>
<td>Min 9.3E-05</td>
<td>4.0E-06</td>
<td>586</td>
</tr>
<tr>
<td></td>
<td>3.7E-03 8.5E-05</td>
<td>Max 3.5E-03</td>
<td>2.6E-05</td>
<td></td>
</tr>
</tbody>
</table>
### Table 7.2: Average & differential deflections in symmetric joints of bridge mid section under truck load

<table>
<thead>
<tr>
<th>Abs. Joint</th>
<th>V-Quick, V=33.3 m/s Average</th>
<th>Diff.</th>
<th>Quick, V=25.0 m/s Average</th>
<th>Diff.</th>
<th>Slow, V= 1.39 m/s Average</th>
<th>Diff.</th>
<th>Joint</th>
</tr>
</thead>
<tbody>
<tr>
<td>Min 39</td>
<td>7.8E-04</td>
<td>1.5E-03</td>
<td>6.8E-04</td>
<td>1.2E-03</td>
<td>5.7E-04</td>
<td>1.0E-03</td>
<td></td>
</tr>
<tr>
<td>Max 188</td>
<td>2.4E-03</td>
<td>4.3E-03</td>
<td>2.3E-03</td>
<td>4.3E-03</td>
<td>2.1E-03</td>
<td>4.1E-03</td>
<td>646</td>
</tr>
<tr>
<td>Min 380</td>
<td>4.3E-03</td>
<td>6.9E-03</td>
<td>3.8E-03</td>
<td>5.2E-03</td>
<td>3.6E-03</td>
<td>5.7E-03</td>
<td>1107</td>
</tr>
<tr>
<td>Max</td>
<td>1.1E-02</td>
<td>2.1E-02</td>
<td>1.0E-02</td>
<td>1.9E-02</td>
<td>9.4E-03</td>
<td>1.9E-02</td>
<td>586</td>
</tr>
<tr>
<td>Min 188</td>
<td>2.4E-04</td>
<td>4.7E-04</td>
<td>2.5E-04</td>
<td>3.7E-04</td>
<td>9.3E-05</td>
<td>1.6E-04</td>
<td></td>
</tr>
<tr>
<td>Max 380</td>
<td>1.6E-03</td>
<td>2.6E-03</td>
<td>1.6E-03</td>
<td>2.8E-03</td>
<td>1.5E-03</td>
<td>2.7E-03</td>
<td></td>
</tr>
</tbody>
</table>

Units: meter

Differential deflections or tilting could be calculated based on movements of symmetric joints in opposite directions. In such cases the maximum degrees of bridge tilting are obtained, see Tables 7.3 and 7.4.

### Table 7.3: Maximum differential deflections or “Tilting” in symmetric joints of bridge mid section under train load

<table>
<thead>
<tr>
<th>Joint</th>
<th>Quick, V=44.4 m/s</th>
<th>Slow, V=1.39 m/s</th>
<th>Joint</th>
</tr>
</thead>
<tbody>
<tr>
<td>39</td>
<td>4.28E-03</td>
<td>4.27E-03</td>
<td>646</td>
</tr>
<tr>
<td>188</td>
<td>4.87E-03</td>
<td>3.37E-03</td>
<td>1107</td>
</tr>
<tr>
<td>380</td>
<td>3.81E-03</td>
<td>3.63E-03</td>
<td>586</td>
</tr>
</tbody>
</table>

Units: meter

The size of tilting or movements in opposite directions for joints 188 and 1107 is 4.87 mm for the dynamic train loading state while it was 0.57 mm for differential movements of the joints in the same direction.

### Table 7.4: Maximum differential deflections or “tilting” in symmetric joints of bridge mid section under truck load

<table>
<thead>
<tr>
<th>Joint</th>
<th>V-Quick, V=33.3 m/s</th>
<th>Quick, V=25.0 m/s</th>
<th>Slow, V=1.39 m/s</th>
<th>Joint</th>
</tr>
</thead>
<tbody>
<tr>
<td>39</td>
<td>6.1E-03</td>
<td>5.7E-03</td>
<td>5.2E-03</td>
<td>646</td>
</tr>
<tr>
<td>188</td>
<td>2.9E-02</td>
<td>2.6E-02</td>
<td>2.5E-02</td>
<td>1107</td>
</tr>
<tr>
<td>380</td>
<td>3.4E-03</td>
<td>3.5E-03</td>
<td>3.0E-03</td>
<td>586</td>
</tr>
</tbody>
</table>

Units: meter

With the same procedure was described above, average and differential values of accelerations calculated using the absolute values of the all 6 joints. The results of accelerations are presented in Tables 7.5 and 7.6. Note that average accelerations are known as bending accelerations and the differential accelerations are known as torsion accelerations.
7.3. RESULTS

Table 7.5: bending & torsional accelerations in symmetric joints of bridge mid section under train load

<table>
<thead>
<tr>
<th>Joint</th>
<th>Quick, V=44.4 m/s</th>
<th>Acceleration, m/s²</th>
<th>Slow, V=1.39 m/s</th>
<th>Joint</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bend.</td>
<td>Tors.</td>
<td>Min</td>
<td>Bend.</td>
</tr>
<tr>
<td>39</td>
<td>0.829</td>
<td>0.012</td>
<td>Min</td>
<td>0.819</td>
</tr>
<tr>
<td>188</td>
<td>0.906</td>
<td>0.039</td>
<td>Max</td>
<td>1.537</td>
</tr>
<tr>
<td>380</td>
<td>0.716</td>
<td>0.357</td>
<td>Min</td>
<td>0.737</td>
</tr>
</tbody>
</table>

Table 7.6: Bending & torsional accelerations in symmetric joints of bridge mid section under truck load

<table>
<thead>
<tr>
<th>Abs. Joint</th>
<th>V-Quick, V=33.3 m/s</th>
<th>Quick, V=25.0 m/s</th>
<th>Slow, V= 1.39 m/s</th>
<th>Joint</th>
</tr>
</thead>
<tbody>
<tr>
<td>Min</td>
<td>0.309</td>
<td>0.028</td>
<td>0.338</td>
<td>0.010</td>
</tr>
<tr>
<td>Max</td>
<td>0.319</td>
<td>0.032</td>
<td>0.344</td>
<td>0.024</td>
</tr>
<tr>
<td>Min</td>
<td>2.174</td>
<td>0.121</td>
<td>2.605</td>
<td>0.696</td>
</tr>
<tr>
<td>Max</td>
<td>2.327</td>
<td>0.187</td>
<td>2.760</td>
<td>0.704</td>
</tr>
<tr>
<td>Min</td>
<td>0.315</td>
<td>0.033</td>
<td>0.361</td>
<td>0.051</td>
</tr>
<tr>
<td>Max</td>
<td>0.325</td>
<td>0.057</td>
<td>0.399</td>
<td>0.053</td>
</tr>
</tbody>
</table>

Units: m/s²

Average and differential values of bending moments calculated using the absolute values of all the 6 joints. The results are presented in Tables 7.7 and 7.8. Average and differential moments are known as bending and torsional moments respectively.

Table 7.7: Bending & torsional moments in symmetric joints of bridge mid section under train load

<table>
<thead>
<tr>
<th>Joint</th>
<th>Quick, V=44.4 m/s</th>
<th>Moments, kN.m</th>
<th>Slow, V=1.39 m/s</th>
<th>Joint</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bend.</td>
<td>Tors.</td>
<td>Min</td>
<td>Bend.</td>
</tr>
<tr>
<td>39</td>
<td>1146.53</td>
<td>5.00</td>
<td>Min</td>
<td>1044.51</td>
</tr>
<tr>
<td>188</td>
<td>0.00</td>
<td>0.00</td>
<td>Max</td>
<td>0.00</td>
</tr>
<tr>
<td>380</td>
<td>6.45</td>
<td>0.19</td>
<td>Min</td>
<td>10.56</td>
</tr>
<tr>
<td></td>
<td>6.45</td>
<td>0.19</td>
<td>Max</td>
<td>10.56</td>
</tr>
</tbody>
</table>
Table 7.8:  Bending & torsional moments in symmetric joints of bridge mid section under truck load

<table>
<thead>
<tr>
<th>Joint</th>
<th>V-Quick, V=33.3 m/s</th>
<th>Quick, V=25.0 m/s</th>
<th>Slow, V=1.39 m/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>Min</td>
<td>534.63</td>
<td>329.38</td>
<td>510.04</td>
</tr>
<tr>
<td>Max</td>
<td>561.45</td>
<td>353.41</td>
<td>515.59</td>
</tr>
<tr>
<td>Min</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Max</td>
<td>188.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Min</td>
<td>62.56</td>
<td>4.78</td>
<td>58.86</td>
</tr>
<tr>
<td>Max</td>
<td>380.00</td>
<td>4.78</td>
<td>58.86</td>
</tr>
</tbody>
</table>

Units: kN.m

7.4 Conclusion

A comparison between the results from this part of the study to other parts of the report indicate that the average values of quantities for joints 39 and 646 are inline with calculated values for joint 39 in the other parts and the maximum variation is about 4% for deflection under the dynamic loading state.

The high speed of the train has a great influence on deflections of cantilever parts of the bridge. As it could be seen in the Table 7.1 the deflections increase by 50% in average for joints 188 and 1107 under the dynamic loading state. The bridge mid point, joint 39, and the other joints as well are influenced just by 5% in maximum, as observed in the previous chapters. These results confirm that for a large scale bridges like this bridge focusing on a single point could potentially lead to erroneous conclusions.

Reviewing Table 7.2 shows that under truck load the average deflection is about 2.4 mm on lower chord while it is about 11 mm in the cantilever parts. So, the so called bridge mid point is not a suitable point for evaluation of influences from truck loads.

Under the train load which moves along the centreline of the bridge it will experience a differential movements in symmetric joints in every cross section. For the mid cross section it is about 3.5 to 4 mm for study joints and Table 7.3 indicate that cantilever parts of the bridge are sensitive to train speed.

Differential deflections or bridge tilting is about 25 mm under a single truck load. This value has been calculated for the two far most joints on the bridge section, joints 188 and 1107. The titling size is relatively small for the joints on the chords as shown in Table 7.4. The higher speed of truck increase deflections by about 20%.

The dynamic increment of accelerations under the train load is about 10% for joints 39 and 646. For the other joints, it is diminished by about 20%. Similarly, the nearby joints to truck load experience increment and the far joint get decrement. The difference for the truck load case is the magnitude of accelerations for high dynamic
loading case. The increment size for joint 188 and 1107 is more than twice and the reduction for the other joints is about 10%.

Due to the geometry of bridge and nature of arch bridges only the joints on the lower chords are evaluated for the bending moments. Dynamic increment of bending moments is about 10% for high speed train and about 20% for high speed truck. These are inline with the results of previous chapters. However, bridge experiences significant torsional moment under a truck load which is running over the lane L1. The dynamic increment of torsional moment is about 10%.
8 Dynamic Amplification Factor

8.1 General

The Dynamic Amplification Factor (DAF) of a structure is simply defined as the ratio of the absolute maximum dynamic response to the absolute maximum static response of that structure. The important parameters that influence the dynamic response of bridges are vehicle speed, road or rail surface roughness, characteristics of the vehicles such as number of axles, axle spacing, axle load, natural frequencies, and damping and stiffness of the vehicle suspension, number of vehicle and their travel paths, characteristics of the bridge structure, such as the bridge geometry, support conditions, bridge mass, and stiffness and natural frequencies. DAF in railway bridges is a function of track irregularities, load distribution pattern due to ballast or slab tracks, damping from the suspension system of the train, the mass of train, and the train speed and some other parameters. Because of the simplicity of DAF expression, it can not characterize the effect of all the above listed parameters. Moreover, as it is originally developed for regular bridges. It is believed that for long spans and complicated bridges the additional dynamic loads should be determined in more accurate way, [7].

The DAF is a unique but, uncertain property of each bridge and depends on the vehicle types for which the bridge is designed. The extreme static load effect is a function of the life time exposure of the bridge to traffic, but the extreme dynamic load effect will be a property of the bridge, [6].

There are mainly two alternative methods for calculation of DAF; formulas according to bridge design codes and performing dynamic analyses, where are possible. The method which has been adopted in this report is the simple dynamic analysis with moving concentrated loads. In the dynamic analyses DAF is calculated as follows;

- Only based on displacements (DAF_d)
- Based on displacements (DAF_d) and moments (DAF_m)
- Max $y_{dyn}$ and $y_{stat}$ at the same point
- Max $y_{dyn}$ and $y_{stat}$ can be at different points
- Separate for upward and downward displacement
- Coupled to one axle of the train
8.2 Analyses

This study has been done based on time history analyses using direct integration method. Six sets of analyses were performed using various vehicle types with different speeds.

For the static loading state, corresponding to speed of 1.39 m/s (5 km/h), firstly a train, secondly a truck, and finally a combination of train and truck run over the bridge.

Likewise, same combinations of vehicle types were used for analyzing of the dynamic state. Train speed in this case was 44.4 m/s (160 km/h) and truck speed was 33.3 m/s (120 km/h). In the combined case, both of the vehicles start at the same time, however; train leaves the bridge early because of higher speed.

In these analyses the bridge behaviour has been studied for every time step of 0.02 s until the vehicles leave the bridge.

8.3 Results

Deflections and bending moments have been extracted from the results and presented in this section. Figures 8.1 to 8.3 illustrate deflections of the lower chord under various kinds of loadings.

Figure 8.1: Bridge lower chord deflection (train load)
Reviewing these figures it is seen that moving with high speed cause larger deflections in the bridge when it is compared to still loading from vehicles. The results from these sets of analyses have been summarized in the Table 8.1. Dynamic amplification factors were calculated as ratios of dynamic to static deflections. For the DAFs in the lower chord maximum deflections at different joints were considered.

**Table 8.1:** Dynamic amplification factors for different loading conditions calculated with different methods

<table>
<thead>
<tr>
<th>Vehicle</th>
<th>Speed, m/s</th>
<th>At Mid Point (Joint 39)</th>
<th>Along Lower Chord (Different Joints)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Defl., m</td>
<td>DAF</td>
</tr>
<tr>
<td>Train</td>
<td>1.39</td>
<td>0.00416</td>
<td>1.016</td>
</tr>
<tr>
<td></td>
<td>44.4</td>
<td>0.00423</td>
<td>1.051</td>
</tr>
<tr>
<td>Truck</td>
<td>1.39</td>
<td>0.00416</td>
<td>1.074</td>
</tr>
<tr>
<td></td>
<td>33.3</td>
<td>0.00447</td>
<td>1.074</td>
</tr>
<tr>
<td>Mix</td>
<td>1.39</td>
<td>0.00398</td>
<td>1.181</td>
</tr>
<tr>
<td></td>
<td>44.4 &amp; 33.3</td>
<td>0.00470</td>
<td>1.181</td>
</tr>
</tbody>
</table>
In the Chapter 7, a comprehensive set of analyses have been performed for various loading states. If the absolute values of deflections and bending moments for the bridge mid point taken from there, it will be possible to calculate the DAF for almost all those cases. In order to have a comparison between DAFs calculated in this part of study with other DAFs which come from other loading cases they are presented in Table 8.2 for train loading and Table 8.3 for truck loading.

<table>
<thead>
<tr>
<th>Train Speed</th>
<th>Slow, V=1.39 m/s</th>
<th>Quick, V=44.4 m/s</th>
<th>DAF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mom., kN.m</td>
<td>1036.11</td>
<td>1141.28</td>
<td>1.102</td>
</tr>
<tr>
<td>Defl., m</td>
<td>0.00417</td>
<td>0.00426</td>
<td>1.022</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Truck Speed</th>
<th>Slow, V=1.0 m/s</th>
<th>Quick, V=25.0 m/s</th>
<th>DAF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mom., kN.m</td>
<td>797.88</td>
<td>879.35</td>
<td>1.102</td>
</tr>
<tr>
<td>Defl., m</td>
<td>0.00416</td>
<td>0.00444</td>
<td>1.067</td>
</tr>
</tbody>
</table>

8.4 Conclusion

In the mid point of the bridge the DAF_d, for the deflections, is 1.02 for train load and is 1.07 for truck load. For the bending moments DAF_m is about 1.1 for both loading cases. When the DAF_d is calculated in different points along the lower chord it gives significantly large values especially for load cases with components of train load.

Mixed traffic consisting of train and truck loads will lead to higher DAFs. This indicates that in practice, the dynamic amplification factor of the bridge is in the higher ranges than the calculated magnitudes in this report. In fact, the bridge will be in service for full load and several combinations of loading cases and thus; its real and largest DAF should be calculated using full scale analyses. The DAF_d which has been calculated here was based on a truck over roadway and a train over railway and results indicate that it is about 1.18 for the mid point and 1.3 for different points along the lower chord.

It seems that for complicated bridges like the study bridge, calculation of the dynamic amplification factors only in one point could be erroneous and therefore, they should be calculated based on dynamic and static responses at different points and using different load cases.
9 Overloading by Ballast

9.1 General

The railway of the study bridge is intended to be placed without ballast, i.e. a slab track solution has been adopted in the design. Apart from the technical advantages of un-ballast rail tracks the main concern for this bridge was the adverse effects of superstructure weight. It was of vital importance to reduce the weights because of the special geotechnical conditions of the site and high risks of earthquakes in the region. The superimposed dead loads on the bridge include 27 kN/m for the railway and 29.6 kN/m for the roadways while the load from ballast will be in excess of 52 kN/m. The dead weight of bridge comprises to 127.5 kN/m and 290 kN/m from steel and concrete parts respectively.

Overloading the bridge will result in higher deflections and stresses. Accelerations are likely diminished with increasing mass. However, the vertical accelerations for a medium span bridge like the study bridge under various loading conditions may attain considerable amounts and could be dangerous for ballast destabilization, [5].

The damping coefficient accounts for all energy-dissipating mechanisms of the structures, for instance ballasted railway, and can be estimated only by experimental methods. Damping does not play an important quantitative role except for resonant responses in structural responses and it is common to use average damping ratios based on the types of construction materials, [8]. Modal and proportional dampings are two distinct types of damping. Modal damping is used for modal time history analyses. It is given as a fraction of critical damping for each mode in the structure. In this study a constant value of damping has been used for all modes.

The basic properties of the bridge like eigenfrequencies and vibration modes may also be altered by a heavier structure. Railway bridges with ballasted rail tracks have greater values of damping ratios and energy absorbing mechanisms. It is desired to have this matter to be scrutinized and any problem areas be recognized.

9.2 Analyses

A train moves on the bridge with crawling speed of 1.39 m/s (corresponding to 5 km/h) and very quickly with speed of 44.4 m/s (corresponding to 160 km/h). Time history analyses using direct integration method have been conducted. Damping factors have been set to 0%, 0.5%, and 0.8% in three sets of analyses for both loading
cases that means 6 set of analyses in total. Time steps are set to 0.05 s. A modal analysis has also been performed to study variations in the mode shapes under ballasted rail track. For this analysis only 10 mode shapes were requested to have a rough estimation of frequency changes under additional ballast loading.

9.3 Results

Results are illustrated for the deflections along the lower chord for the three states of assumed damping factors, see Figures 9.1 to 9.3.

Figure 9.1: Bridge lower chord deflection (train load with ballast, d=0)

Figure 9.2: Bridge lower chord deflection (train load with ballast, d=0.5%)

Figure 9.3: Bridge lower chord deflection (train load with ballast, d=0.8%)
Should these figures be compared to the Figure 6.4 of Chapter 6 which, has been produced with same method and time steps, it will be possible to derive the following summary table of results, See Table 9.1.

Table 9.1: Maximum absolute values of bending moments, accelerations & deflections in bridge mid point under train load

<table>
<thead>
<tr>
<th>DI Method, Δt=0.05 s</th>
<th>Train Speed, m/s</th>
<th>Damping Coefficient</th>
<th>No Ballast</th>
<th>0.5%</th>
<th>0.8%</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>0</td>
<td>0.5%</td>
<td>0.8%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Deflection, m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.39</td>
<td>0.00409</td>
<td>0.00414</td>
<td>0.00404</td>
</tr>
<tr>
<td></td>
<td></td>
<td>44.4</td>
<td>0.00437</td>
<td>0.00451</td>
<td>0.00444</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Acceleration, m/s²</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.39</td>
<td>0.3052</td>
<td>0.42239</td>
<td>0.00300</td>
</tr>
<tr>
<td></td>
<td></td>
<td>44.4</td>
<td>0.2148</td>
<td>0.21740</td>
<td>0.09800</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Moment, kN.m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.39</td>
<td>1041.07</td>
<td>1036.4</td>
<td>997.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>44.4</td>
<td>1131.73</td>
<td>1175.2</td>
<td>1155.0</td>
</tr>
</tbody>
</table>

The results from the modal analyses are presented in the Table 9.2. The mode shapes have been compared to the those are obtained from un-ballasted model.

Table 9.2: Variation of the modal frequencies (in Hz) between ballasted & un-ballasted rail track

<table>
<thead>
<tr>
<th>Mode shape</th>
<th>w/o ballast</th>
<th>w ballast</th>
<th>Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.376</td>
<td>0.370</td>
<td>0.98</td>
</tr>
<tr>
<td>2</td>
<td>0.595</td>
<td>0.595</td>
<td>1.00</td>
</tr>
<tr>
<td>3</td>
<td>0.870</td>
<td>0.870</td>
<td>1.00</td>
</tr>
<tr>
<td>4</td>
<td>0.891</td>
<td>0.884</td>
<td>0.99</td>
</tr>
<tr>
<td>5</td>
<td>0.993</td>
<td>0.990</td>
<td>1.00</td>
</tr>
<tr>
<td>6</td>
<td>1.147</td>
<td>1.140</td>
<td>0.99</td>
</tr>
<tr>
<td>7</td>
<td>1.186</td>
<td>1.186</td>
<td>1.00</td>
</tr>
<tr>
<td>8</td>
<td>1.273</td>
<td>1.260</td>
<td>0.99</td>
</tr>
<tr>
<td>9</td>
<td>1.393</td>
<td>1.373</td>
<td>0.98</td>
</tr>
<tr>
<td>10</td>
<td>1.453</td>
<td>1.449</td>
<td>1.00</td>
</tr>
</tbody>
</table>

9.4 Conclusion

As it is seen in the Table 9.1 all the three quantities are increased by overloading the bridge. Energy absorbing properties of the ballast helps the bridge to have its vibrations diminished to a large extent. Although, more or less the same size of reduction could be achieved using new technologies by special slab track systems.
In crawling speeds which corresponds to the static loading state the higher damping of
the ballasted tracks have led to smaller deflections and bending moments than the
ballast-less bridge which shall be simply ignored.

The 10% additional weight from ballast has no considerable effect on basic vibration
modes and frequencies of the bridge.
Bibliography


A Appendix A-Design Summary

A.1 Introduction

This appendix has been summarized from the design reports provided by the designer, Aas-Jakobsen Engineering Consultants. Data are provided here have been limited to the requirements of this study for creating an accurate structural model. After a general introduction to the whole of Urmia Lake Bridge the Arch Bridge is described.

A.1.1 General

The Urmia Lake with an approximated area of 4500 to 6000 square kilometres and with an average water depth of 6 metres is situated in the North West Mountainous zone of Iran. The lake’s narrowest wide is 15.4 km where a non-completed rock-fill causeway has been constructed from both shores.

The construction operation of the causeway started in 1979 and within a decade almost 14 km (10.85 km from west and 3.25 km from east coast) has been established. However; about 1.3 km in the middle section of the causeway left open and it was decided to build a bridge to link the two side embankments to each other.

The current bridge concept is 1276 m long, including 19 spans. The configuration consists of a 100 m main span and 9 spans in both sides. The main span is in the form of an overhead tied arch structure and the side spans will be in form of composite box girders, see Figures A.1 & A.2.

![Figure A.1: A 3D view of the Urmia Lake Bridge](image-url)
In transition zones of this bridge to embankment, the causeway gets considerable settlements. Two approach bridges are designed to accommodate flexibility between the bridge and the settling embankments. These bridges are designed in a pre-cast post-tensioned system and are built span by span. Total length of the bridge including the main bridge structure, approach bridges on the embankment, and the transition spans is 1710 m.

![Figure A.2: Longitudinal section of the bridge](image)

The superstructure accommodates both road and railway traffic and consists of three parallel box girders in viaducts. The central girder is intended for single track railway and the outer ones are for double lane road traffic, see Figure A.3. In the opening; the superstructure is supported by concrete piers with a circular shape and the foundations consist of submerged concrete caissons with long driven friction piles. On the embankments; two abutment structures are established on driven piles and piers of approach bridges are provided as slabs directly on the embankment and columns are of wall type.

![Figure A.3: Cross section of the viaduct bridges](image)

A.1.2 The Arch Bridge

The desired arch bridge is the central part of the Urmia Lake Bridge that spans 100 meter. The height of the arch is 20.1 m from bottom axis. In general; box girders are used as structural elements in different parts of the bridge. The concrete slab is supported on free sliding bearings directly on the transverse beams. The bridge will accommodate both road and railway traffic. A single track railway will pass through the arch and the outer cantilever parts are for double lane road traffic in each direction. The bridge design life is taken as 100 years. See Figures A.1, A.2 & A.3.
A.1. INTRODUCTION

Figure A.4: *Longitudinal section of the Arch Bridge*

Figure A.5: *Cross section of the Arch Bridge*

Figure A.6: *A 3D view of the Arch Bridge*
Appendix A. Appendix A-Design Summary

A.2 Material

The Arch Bridge is a steel arch frame with a concrete slab covering both railway and highway areas. Material properties are summarized in Table A.1.

<table>
<thead>
<tr>
<th>Material</th>
<th>Grade</th>
<th>E-modulus, MPa</th>
<th>G-modulus, MPa</th>
<th>Density, kN/m³</th>
<th>Tem. Coeff., 1/°C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>S355</td>
<td>2.00E5</td>
<td>0.769E5</td>
<td>77.0</td>
<td>1.2E-5</td>
</tr>
<tr>
<td>Concrete</td>
<td>C55</td>
<td>3.55E4</td>
<td>1.479E4</td>
<td>25.5</td>
<td>1.1E-5</td>
</tr>
</tbody>
</table>

A.3 Structural Definition

A.3.1 Structural Model

The Arch Bridge is build up of steel box-girders in the longitudinal direction and steel box-girders with 4.5 m c/c in the transverse direction. The concrete plate on the railway surface on the Arch Bridge is a so-called free-floating slab and is modelled with correct dimensions and is supported on spring elements with correct shear stiffness. The cross-beams for the train near axis 11, See Figure A.7, where the brake loads from the train acts, are box sections. For cross-sections with both concrete and steel parts, concrete properties are transformed to equivalent steel properties.

The concrete slab is free floating & supported on free sliding bearings directly on the transverse beams. The concrete slab is about 400 mm thick & continuous and behaves like a stiff plate. It is fixed in the horizontal plane in one discrete point near axis 11. The connection is established by means of studs between the concrete and the steel beams. The slab is vertically supported on neoprene blocks that transfer just small shear forces in the longitudinal directions caused by difference in temperature and shrinkage deformations. The positioning of the neoprene blocks was determined following a careful trial and error process in order to eliminate uplift. Transversal force at axis 11 is taken by studs between the concrete and the steel beams. The remaining concrete slab is supported transversally on each side of the slab between lower chords with horizontal supports approximately in 30 m c/c. The supports between the concrete slab and the steel brackets are made of Teflon. By this solution the concrete plate is given the possibility to slide in the longitudinal direction without transferring any forces in this direction.

The design of the concrete slab is based on static analyses. The slab model comprises both the steel cross beams and lower chord. The diagonal points are modelled as vertical springs. The stiffness of these springs is found by iterations until it resulted in the same deflections as from the Arch Bridge model.

Reviewing the design reports indicates that the following strategy has been taken into consideration in modelling of the arch bridge;
• Arches (upper chords) are made of 44 segments,
• Tie beams (lower chords) include 66 segments,
• Diagonals defined in 1 segment,
• Outer cross beams (cantilever beams for roadway) divided into 3 segments,
• Inner cross beams (middle railway beams) divided into 4 segments,
• Upper & lower chords are connected using a special segment at either end, and
• Concrete slab is monolith & sits on sliding pad bearings on the cross beams.

A.3.2 Support Conditions

In the internal fixations of the Urmia Lake Bridge superstructure the Arch Bridge is fixed in longitudinal direction at pier 11, see Figure A.7. Assumptions and calculations for Arch Bridge supports and for the floating slab are described in section 3.5 of this appendix. Expansion Joints (EJ) are introduced to facilitate relative longitudinal displacements between different bridge sections due to load effects such as temperature, braking and earthquake. EJs related to Arch Bridge are located in the following locations;
• Viaduct/Arch axis 10/11
• Arch/Viaduct axis 12/13

![Figure A.7: Support conditions for Arch Bridge](image)

A.3.3 Steel Members

General descriptions of steel sections are provided in Table A.2. Various box types are depicted in Figure A.8.
Table A.2: Descriptions of steel sections

<table>
<thead>
<tr>
<th>Box Type/Size, mm</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Box1- 1450x1200x40x40x20</td>
<td>Upper chord</td>
</tr>
<tr>
<td>Box1- 1000x1000x25x40x20</td>
<td>Cross beam between upper chords</td>
</tr>
<tr>
<td>Box2- 1450x1200x25x40x17x139</td>
<td>Lower chord, left</td>
</tr>
<tr>
<td>Box2- 1450x1200x25x40x17x139</td>
<td>Lower chord, right</td>
</tr>
<tr>
<td>Box1- 1450x800x14x40x20</td>
<td>Cross beam for railway in axes 11&amp;12</td>
</tr>
<tr>
<td>Box1- 1450x800x14x40x20</td>
<td>Cross beam for railway in general</td>
</tr>
<tr>
<td>Box1- 540x800x12x40x20</td>
<td>Cross beam 1 for roadway in axes 11&amp;12</td>
</tr>
<tr>
<td>Box1- 880x800x12x40x20</td>
<td>Cross beam 2 for roadway in axes 11&amp;12</td>
</tr>
<tr>
<td>Box1- 880x800x12x40x20</td>
<td>Cross beam 3 for roadway in axes 11&amp;12</td>
</tr>
<tr>
<td>Box1- 1180x800x12x40x20</td>
<td>Cross beam 4 for roadway in axes 11&amp;12</td>
</tr>
<tr>
<td>Box1- 1180x800x12x40x20</td>
<td>Cross beam 5 for roadway in axes 11&amp;12</td>
</tr>
<tr>
<td>Box1- 1180x800x12x40x20</td>
<td>Cross beam 6 for roadway in axes 11&amp;12</td>
</tr>
<tr>
<td>Box1- 1450x800x12x40x20</td>
<td>Cross beam 1 for roadway in general</td>
</tr>
<tr>
<td>Box1- 1450x800x12x40x20</td>
<td>Cross beam 2 for roadway in general</td>
</tr>
<tr>
<td>Box1- 1450x800x12x40x20</td>
<td>Cross beam 3 for roadway in general</td>
</tr>
<tr>
<td>Box1- 1450x800x12x40x20</td>
<td>Cross beam 4 for roadway in general</td>
</tr>
<tr>
<td>Box1- 1450x800x12x40x20</td>
<td>Cross beam 5 for roadway in general</td>
</tr>
<tr>
<td>Box1- 1450x800x12x40x20</td>
<td>Cross beam 6 for roadway in general</td>
</tr>
<tr>
<td>Box3- 450x250x25</td>
<td>Diagonals</td>
</tr>
<tr>
<td>Box1- 1450x800x14x40x20</td>
<td>Beam near axis 11 where the slab is tied up</td>
</tr>
</tbody>
</table>

Figure A.8: Typical box types for Arch Bridge

A.3.4 Concrete Slab

Descriptions of concrete sections are presented in Table A.3 and various slab types are depicted in Figure A.9.
### Table A.3: Descriptions of concrete sections

<table>
<thead>
<tr>
<th>Slab Type/Size, mm</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab1- 10100x299x425</td>
<td>Slab for roadway</td>
</tr>
<tr>
<td>Slab2- 5600x355x425</td>
<td>Slab for railway</td>
</tr>
<tr>
<td>Slab3- 2150x327</td>
<td>Slab over the lower chord</td>
</tr>
<tr>
<td>Slab3- 1500x362</td>
<td>Dummy elements connecting the slabs</td>
</tr>
<tr>
<td>Slab4- 1500x338x500x1000x200</td>
<td>Dummy elements for haunches beneath the slabs, over the slab bearings</td>
</tr>
</tbody>
</table>

![Typical slab types for Arch Bridge](image)

### Figure A.9: Typical slab types for Arch Bridge

#### A.3.5 Bearings

The bearing arrangement has been subject to evaluation through several analyses models to reach a good balance between restrained axis to carry longitudinal forces from earthquake (EQ) and train braking, and unrestrained axis to allow for free temperature movements.

The critical force action for horizontal loads is in general earthquake and whereas it is the dominant force action, ductility in the bearing arrangement and overload capacity is appropriate. The selected bearing arrangement consists of:
- Pot bearings to carry main vertical loads
- Neoprene (elastomeric) steel plate reinforced pad bearings to carry horizontal loads

Longitudinal horizontal short term loads are taken in axis 11. The fixed bearings consists of a neoprene pad, vertically mounted in a steel frame on a concrete plinth on top of the concrete piers, reacting on a steel bracket extending down from the soffit of the steel box bridge deck. Each bearing has a thickness which allows for rotation and a sliding interface PTFE/stainless steel which allows for deformation in two directions.

For fixed bearing arrangement the movement is very small and only due to the effect of rotation of the bridge deck times distance from point of restraint to the pot bearing. This movement is in general very smaller than 10 mm. For sliding bearings the movement in axis 12 is in general large and determined by the requirements of minimum seat width according to AASHTO clause 4.7.4.4.

According to the project’s Design Basis the following temperature range is considered;
Appendix A. Appendix A-Design Summary

- 100 year return $T_{\text{min}} = -30 \, ^{\circ}\text{C}$
- 100 year return $T_{\text{max}} = +35 \, ^{\circ}\text{C}$

The reactions from bearings are according to the static load analyses for pot bearings and according to the dynamic load analyses for neoprene pad bearings. Deformation capacities were investigated specially for every case. The bearings have been designed according to AASHTO, Chapter 14.6 and 14.7.

**Main bearings - pot bearings for vertical loads**

All pot bearings have been designed in ultimate limit state (ULS) for loads. Movement capacity is, according to AASHTO 4.7.4.4, zone 4. Rotation capacity is 2% standard for this type of bearing.

**Main bearings - buffer bearings for horizontal transverse loads**

All buffer bearings for horizontal transverse loads are neoprene pads reinforced with steel plates. The capacities are according to serviceability limit state (SLS) which is traditional for this type of bearing used as typical bridge bearings to carry vertical loads with many repetitions. For these neoprene pad bearings which are used as buffer bearings for EQ, fatigue with many load cycles is not a design criterion. Instead the bearing must sustain a fairly large load just a few times in its life span and the extreme EQ load only once.

All these buffer bearings have designed as free sliding. This is to provide for small, free movements due to rotations on the main bearings. The design of the bearings also provide for:
- Tolerance at installation
- Exchange or adjustment at a later stage

The total lengths of the sliding surface for movable bearings correspond to the lengths provided for sliding pot bearings.

**Main bearings - buffer bearings for horizontal longitudinal loads**

All buffer bearings for horizontal longitudinal loads are neoprene pads reinforced with steel plates. The capacities are according to SLS for this type of bearing. EQ is the critical loading for all longitudinal forces and movements. Due to the nature of EQ movements, it was considered appropriate to provide nominal buffer bearings at certain places in the structure. In axis 12 such buffers have been provided at the end of the tie-beams to prevent the arch from falling off the bearing even if the analyzed movement was considerably smaller than the length of the sliding plate provided.
Neoprene Pad Bearings for Concrete Slab

The concrete slab in the arch bridge deck has been designed as a “floating” slab supported by neoprene bearing pads vertically on the cross beams and horizontally on the arch legs. Movement in the bearings is only due to shrinkage of concrete and differential temperature. The slab has been fixed in axis 11. The maximum differential movement between steel frame and concrete slab in axis 12 is:

- due to differential temperature expansion:
  \[ \Delta \epsilon = (13 - 10) \times 10^{-6} = 3 \times 10^{-6} \, ^{\circ} \text{C} \]
  \[ T = +35, \, L = 100 \, \text{m} \rightarrow \Delta L = 10.5 \, \text{mm} \]

- due to shrinkage
  \[ \epsilon_c = 0.03\% \]
  \[ \Delta L = 0.3 \times 10^{-3} \times 100 \times 10^3 = 30 \, \text{mm} \]

The bridge slab is supported by vertical bearings; 2 Nos. at 2.50 m c/c between the arches for the railway part, and 2 Nos. at 6.00 m c/c on each cantilever outside the arches for the roadway part. A layout of the bearing arrangement is shown in Figure A.10. The distances transversely between the bearings on each cross beam have been in general chosen to provide enough self weight reaction on each bearing to avoid uplift in ULS for unsymmetrical traffic loading.

The required size of the typical neoprene pad bearing was 200 x 300 mm with a permissible load of 750 kN in SLS, approximately 1500 kN in ULS. A deformation capacity of 20 to 40 mm was required. If this had to be taken as shear deformation in the bearing the required maximum size was 300 x 400 x 85 mm, and the minimum size was 200 x 300 x 52 mm. Alternatively, if the deformation had taken by sliding the bearing size was 200 x 300 x 19 mm.

The rotation required in the longitudinal direction of the bridge due to unbalanced traffic loads is of the order 0.05% which is much smaller than the capacity of the 200 x 300 x 19 mm pad, which is 0.35%. Furthermore, the 19 mm pad with sliding plate provides much more flexibility with respect to unexpected larger deformations and it was better detailed in the construction of the slab. The sliding bearing type was therefore adopted. The detail of the concrete slab including location of the slab support is shown in Figure A.11.
Figure A.11: *Pad bearings in the connection of slab and steel cross beams*

As a result of the analyses the restraint arrangement near axis 11 has been designed as composite connection between bridge slab and steel deck grid. The buffer is there to lock the slab longitudinally and transversally, see Figure A.12. Studs at main girder and cross beams take the forces which acts centre slab (vertically). The moment created from this is taken by the studs located on the side of the box sections.

*Figure A.12: Buffer/anchor near axis 11*

Main horizontal restraints of the floating bridge slab have been provided close to axes 11 & 12. In addition two secondary restraints have been provided at crossbeams Nos. 9 & 15. The main restraints have been located close to the bearings for transverse load between the arch bridge and foundation in the centreline of the bridge in axes 11 & 12 in order to minimize the horizontal load transfer in the steel arch structure. Near axis 11 the restraint is in the form of composite action between a thickened bridge slab and a strengthened steel grid in the end bay. Near axis 12 the restraint has been provided by two neoprene pad buffer bearings in the end bay. The design of the bearing support near axis 12 is shown in Figure A.13. The sliding capacity provided is + 50 mm.
A.4 Traffic Loads

A.4.1 Vehicle Types

Different vehicle types which have been used in the analyses of the Arch Bridge are illustrated in Figures A.14 to A.17. (Ref. Iranian Regulations For Bridge Design)

Figure A.13: Horizontal buffer bearing near axis 12

Figure A.14: Type number 1- Roadway ordinary loads

Figure A.15: Type number 2- Railway standard load
Figure A.16: Type number 3- Roadway tank load

Figure A.17: Type number 4- Roadway tank-carrying trailer load

Other load types comprise to;
- Type number 5; Brake load roadway, horizontal point load of 400 kN in each lane in any position.
- Type number 6; Brake load railway, horizontal load of 20 kN/m, max total force of 4500 kN. For the Arch Bridge the horizontal point load of 20 kN/m x 100 m = 2000 kN in any position.

A.4.2 Traffic Lanes

The load types and the lane numbers have been combined in the analyses and related to their corresponding eccentricity. The traffic lanes numbering and relative eccentricities are according to the Figure A.18.

Figure A.18: Traffic lanes on arch bridge

A.4.3 Load Cases

From several numbers of load cases (LC) which, have been used in the analyses of the Arch Bridge following ones are relevant to our study.
• **LC1- dead load of steel Arch Bridge**: the self weight of the steel structure was calculated for all steel work. Unit weight of steelwork was set to 77.0 kN/m$^3$ x 1.2 = 92.4 kN/m$^3$. The factor 1.2 was used to take care of extra weight of stiffeners and also includes a contingency.

• **LC2- dead load of concrete Arch Bridge**: the self weight of concrete slab was set manually as point loads on top of bearings. Unit weight of concrete was assumed to be 25.5 kN/m$^3$. The weight of the haunches was set manually at the points for the supports as nodal loads. Brake load support for the concrete slab has been designed as a concrete box at axis 11, in the centre of bridge with volume 1.6m x 2.0 m x 3.8 m = 12.2 m$^3$.

• **LC3- traffic load on entire Arch Bridge, maximum symmetric loading**: load cases 3 to 6 have been used in the performed P-Δ analysis of the Arch Bridge. In that analysis self weight of the structure was combined with 4 typical traffic loading situations in order to determine the second order effects of the axial force in the arch. Load case 3 defines a traffic loading situation which gives the largest total axial force in the upper chord.

• **LC4- traffic asymmetric, traffic loading on arch bridge, maximum asymmetric loading**: Load case 4 defines a traffic loading situation which gives the largest asymmetric force in the upper chord.

• **LC5- traffic loading on arch bridge, traffic on south east side**: Load case 5 is a traffic loading situation defining the most eccentric traffic loading to the south east side of the arch bridge.

• **LC6- traffic loading on arch bridge, traffic on south east side**: Load case 6 is a traffic loading situation defining the most eccentric traffic loading to the north west side of the arch bridge.

**LC7- superimposed dead load**: superimposed dead loads on the Arch Bridge are set as follows:

- **Roadways**:
  - Asphalt: 21.9 kN/m
  - Utility: 1.5 kN/m
  - Steel railings: 1.1 kN/m
  - Edge barriers: 5.1 kN/m

- **Railway**:
  - Track load: 2.7 kN/m
  - Guide rail track with fasteners: 2.5 kN/m
  - Utility: 3.5 kN/m
  - Steel railings: 1.1 kN/m
  - Edge barriers: 17.2 kN/m

Creep and shrinkages effects have also been considered in the design process in different time dependent & independent analyses where were appropriate.

### A.4.4 Load Combinations

The load combinations for the analyses of Urmia Lake Main Bridge are set up in accordance with the AASHTO code. Reduction factors for multiple highway lane loading:

- For one or two lane loading: 1.0
• For three lane loading: 0.9
• For 4 or more lane loading: 0.75

For simultaneous roadway and railway loading, the railway loading has been added without any reduction factor.

Dynamic impact factors for live load have been included in the load combinations. The factors depend on the members’ dynamic load length. In general, properties of the different parts of the structure have different dynamic impact factors for the traffic loads. Descriptions of the traffic load impact factors have been taken from Design Basis of the project and are as follows;
• Upper and lower chords: $L(\varphi) = 100$ m (span length):
\[
\delta(100) = 1.3 - 0.005 \times L(\varphi) = 1.3 - 0.005 \times 100 = 0.8 < 1.0, \text{ i.e. } 1.0 \text{ for road traffic}
\]
\[
\delta(100) = \frac{2.16}{\sqrt{L(\varphi) - 0.2}} + 0.73 = \frac{2.16}{\sqrt{100 - 0.2}} + 0.73 = 0.95 < 1.0, \text{ i.e. } 1.0 \text{ for rail traffic}
\]

• Transversal beams outside lower chords (road part): $L(\varphi) = 10$ m (beam length):
\[
\delta(10) = 1.3 - 0.005 \times L(\varphi) = 1.3 - 0.005 \times 10 = 1.25, \text{ for road traffic}
\]

• Transversal beams between lower chords (rail part): $L(\varphi) = 8$ m (beam length):
\[
\delta(10) = \frac{2.16}{\sqrt{L(\varphi) - 0.2}} + 0.73 = \frac{2.16}{\sqrt{8 - 0.2}} + 0.73 = 1.55, \text{ for rail traffic}
\]

• Diagonals $L(\varphi) = 9$ m (span between diagonals for lower chord):
\[
\delta(9) = 1.3 - 0.005 \times L(\varphi) = 1.3 - 0.005 \times 9 = 1.26, \text{ for road traffic}
\]
\[
\delta(9) = \frac{2.16}{\sqrt{L(\varphi) - 0.2}} + 0.73 = \frac{2.16}{\sqrt{9 - 0.2}} + 0.73 = 1.50, \text{ i.e. } 1.0 \text{ for rail traffic}
\]

A.5 Analyses

A.5.1 Static Analyses

In axis 10/11 and 12/13 both Viaduct Bridges and Arch Bridge are supported on the same pile cap. This pile cap is 4 m high and gathers a group of 26 piles. Top of the columns are connected by a cross beam and provides sufficient room for placing 5 different types of bearings for the Arch Bridge and 9 more bearings for Viaduct Bridges. The load cases have been covered by the static analyses include all situations relevant for the bridge in operation. The effect of foundation stiffness was incorporated through a “Soil-Structure Interaction Analysis”. This means that effects of foundations have been included in the structural model by foundation stiffness matrices through spring elements.
A.5.2 Dynamic Analyses

The dynamic analyses utilize the same models as for the static analyses. The dynamic investigations performed for the Urmia Main Bridge is concentrated on the response of the bridge to earthquake loading. Load reactions due to this loading have been determined based on the standard “Multimodal Spectral Method” as described e.g. by AASHTO.

Bedrock / stiff soil outcrop acceleration for the ULS level AASHTO earthquake corresponding to the 475-year event (10% probability of exceeding in 50 years) has been used.

The program SHAKE was used for the site response analysis. For this purpose, two earthquake records which had been provided by client (Ghaen: L-component, 1979, and Zarrat St.: L-component, 1994) were used as outcropping bedrock input. The results of the SHAKE analysis have been used to estimate the degraded shear modulus values compatible with the strain levels during the earthquake. The program PILES was then used to obtain the pile-soil system and the acceleration time history of the pile cap (foundation caisson) was calculated.

The dynamic earthquake analyses have been based on the results of the basic earthquake investigations for the region and the results from the soil investigations which influence the amplification from bedrock to pile caps.

The fundamental results for a multimode spectral analysis are the natural frequencies and eigenmodes resulting from the eigenvalue calculations. A list of the three natural frequencies in the extreme event limit state (500 year) is given in Table A.4. A total of 300 modes have been used for the response calculations of the Urmia Lake Bridge.

Table A.4: Natural frequencies of Arch Bridge

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency, Hz</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.409</td>
<td>Arch Bridge movement in longitudinal direction</td>
</tr>
<tr>
<td>4</td>
<td>0.521</td>
<td>Arch Bridge out of plane movement</td>
</tr>
<tr>
<td>10</td>
<td>0.828</td>
<td>Arch Bridge out of plane movement, Twisting of road plane</td>
</tr>
</tbody>
</table>

A general overview of displacement results are given in the following Tables A.5 and A.6 based on the performed earthquake analyses for the extreme event limit state conditions.

Table A.5: Maximum displacements of the substructure, 500 yr earthquake loading

<table>
<thead>
<tr>
<th>Location</th>
<th>Displacement- Direction, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>At Bottom of Pile Cap</td>
<td>Longitudinal</td>
</tr>
<tr>
<td>Pier 10/11</td>
<td>±167</td>
</tr>
</tbody>
</table>
Table A.6: Maximum displacements of the superstructure, 500 yr earthquake loading

<table>
<thead>
<tr>
<th>Location</th>
<th>Displacement, mm</th>
<th>Location</th>
<th>Displacement, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>At Bridge Deck Surface Level</td>
<td></td>
<td>Longitudinal X-Dir</td>
<td>Transversal Z-Dir</td>
</tr>
<tr>
<td>Arch Bridge, Pier 12/13</td>
<td>±352</td>
<td>Arch Bridge, Pier Middle Span</td>
<td>±326</td>
</tr>
<tr>
<td>Top of Arch</td>
<td></td>
<td></td>
<td>±336</td>
</tr>
</tbody>
</table>

Fatigue Analyses

The fatigue analyses utilize the same models as for the static analyses. Thus no further description of models is included in this part. It has been carried out based on the static fatigue load models referenced in AASHTO for the roadway and in Eurocode 1-Part 3 for the railway. The fatigue analyses has been performed by a static analyses of the FLS load model given in AASHTO section 3.6.1.4 and ENV1991-3, page 92 and annex F.

Accidental Analyses

Likewise, the accidental analyses utilize the same models as for the static analyses. Thus no further description of models has been included in this part. The main task was the study of the bridge capacity after loss of one brace (diagonal). The purpose was thus to check the structure in an idealized post-damage condition to ensure that progressive collapse wouldn’t develop subsequently. This has been achieved by selecting a representative number of braces for removal and then to perform checks on the remaining structure to ensure that the remaining members are within the elastic range for a probable level of bridge loading. In terms of safeguarding against progressive collapse, this is a conservative approach because no amount of yield/plastic behaviour in the intact members is permitted.

The accidental analyses have been performed by a static analysis of the load in the bridge in-service condition. Several analyses have been performed for simulation of different loss of stay scenarios. The selection of stay for removal has been based on results in Arch Bridge design reports. The most loaded elements have been removed one by one. In the post-damage accidental case scenario, primarily dead load and live load were included.