IMPACT RESPONSE ANALYSIS OF MASONARY ARCH USING NUMERICAL MODELLING IN ANSYS

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We hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. We also declare that, as required by these rules and conduct, we have fully cited and referenced all material and results that are not original to this work.

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ABSTRACT

Apart from natural hazards like earthquakes and rock falls resulting from the land slides, infrastructures have suffered a lot from the extreme loadings caused by man-made hazards. These loading cases include impacts from high velocity projectiles, blasts and shocks resulting from explosions, and impacts from rock falls caused due to landslides, caused by deforestation in mountainous regions.

Studying the impact performance of the masonry arches that are used in bridge construction is the main purpose of this study. This area of structural analysis is not much explored into and needs further research. This study aims to asses and evaluate the performance of masonry arches using the finite element analysis technique. The modeling and analysis software employed to study the masonry arch performance is Ansys-LS Dyna and explicit dynamic analysis is carried out.

The modeling of the masonry arch requires understanding of response of the arch to loading conditions and the damage mode. To analyze the arch under extreme-loading scenario (Impact in this case) requires understanding of the impact phenomenon and the dynamic behavior of the structural components in high strain rates.

The finite element modeling of the masonry arch requires calibration of the material models with optimum properties and then validation of model arch by comparing its behavior with that of an experimental arch. Techniques of mesh independency, and comparison of arch behavior under self-load was carried out in this case to validate the FE model.

After validation with experimental results, the impact force generated by different impact conditions was studied by changing the magnitude and the loci of the impact.

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Chapter 1

INTRODUCTION

1.1 Background

Since recently it has becoming a common practice of terrorist organizations around the world to use explosives to destruct important infrastructures including historical structures. Of these, damages to historical masonry bridges that are arch structures, which have been used for railways and highways, will be a two-fold disruption of the traffic flow and lose of the historic fabric. Bridges can be catastrophically damaged locally or globally by an explosion on or nearby a bridge. In addition to the economical and heritage loss, the threat can cause additional causalities if the explosion results from an explosive device carried by a train or vehicle crossing the bridge.

The other major issue is the land sliding in the mountainous areas. It occurs due to earthquakes, tsunamis and sometimes due to tectonic plate motions depending upon the plate motions relative to the mountains. In Pakistan, this issue became very serious after earthquakes and with the increase in number of earthquakes and their reoccurrence rate. Due to these reasons land-sliding occurs and sometimes our contact with some areas breaks. So, it’s necessary to provide structures which can sustain impact loadings by land-sliding and remain in service. As being civil engineers we provide masonry arch structures which can withstand impact loads.

Bridges do sometimes fail. They may fail partially or totally collapse. Partial failure of the bridges results in collapse of different structural entities of the bridges, but the structure as a whole keeps erected, while in total collapse, the bridge comes down to the ground.

Experience can be gained by the past failure of structures, because when a bridge collapses it has certainly been pushed to the limit in some way. Therefore, structural collapses in general, and particularly bridge collapses, which are often most spectacular, have a key effect in the development of the knowledge of structural action and material behavior and have impelled research into particular fields.

In practice, failures occur in different forms in a material and are likely to be different for different materials (e.g. steel, concrete, and timber bridges). Common types of failure that occur in steel bridges are yielding (crushing, tearing or formation of ductile or brittle plastic hinges), buckling, fracture, shearing and fatigue (reduced material resistance, reversal of stress in welds and connections, vibrations) and corrosion. Common failure types in concrete bridges are fatigue (reduced material resistance, minor or major cracks), crushing, fracture and rupture. Failure in both steel and concrete bridges happened by the large deformations due to sway, violent shaking during seismic events, impact and erosion of soil in floods or settlement due to expansive soils may induce.

Failures may happen in service, but probably more often during construction. Physical causes are various such as erosion, reversal of stress, impact, vibrations, wind, and extreme events.

To cater the threats from such extreme loading conditions (e.g. impact loadings), efforts have been made during the past decades to develop methods of structural analysis and design to resist impact loads particularly from military defense engineering. However the issue has become serious for civil infrastructures too after the campaigns of terrorist organizations. Hence it is mandatory to make structural assessment for historic structures and take appropriate safety measures against the probable catastrophic damage.

Figure 1.1 Original Coastline and Bridge Structures

Figure 1.2 Damaged Coastlines and Bridge

Bridge structures along the coast lines are often subjected to hydrodynamic loads of various forms and intensities. The most dramatic loads are those due to tsunamis and storm surges as vividly as demonstrated by images of the Dec. 2004 Indian Ocean Tsunami and the Sept. 2005 Katrina Hurricane in the Gulf of Mexico. Other loads include wave impact, current induced scour, and floating debris impact.

1.2 Problem Statement

Impact loading analysis of arch masonry structures (bridges) involves the evaluation of structural performance of stone masonry arches, which are the basic structural components of such bridges. Hence the issue involves three major topics:

1. Accurate specification of actions on stone masonry bridges arising from impact loads. When an impact load occurs on masonry arch, the bridge will be subjected to impact pressure, impacts from fragments and base excitation from ground shock. Hence for accurate specification of such actions it is important to understand the interactions that occur between the structure and its surrounding during accident.
2. Accurate estimation of the resistance curve of structural system, the arches using either a simplified guidelines or using robust numerical simulations.
3. Accurate performance assessment of the bridge against such actions using either simplified procedures or refined numerical simulations.

1.3 Objectives

The structural assessment of masonry arch bridges against impact loads is a paramount issue as the landslides and rock falls affect our connection between different places and breaks inter-links. Numerical or analytical studies about the effect of impact on civil infrastructures have been limited; most of the works were done for military protective structures. On the aim of developing a simplified guideline for impact load performance of masonry arch bridges, the following aims and objectives are identified to be studied in this thesis as a basis towards the development of the knowledge of structural performance assessment of masonry arch bridges against actions resulting from impact loadings.

1. To review the impact mechanism and to define associated actions on masonry arch bridges.
2. To use most feasible modeling technique to model the masonry arch, which can then be applied to other similar structures (other masonry arches) to evaluate their performance under all types of loading conditions.

3. To identify the significant parameters that affect numerical simulation of stone masonry arch bridges in the Finite element Analysis and devise a formulation for their threshold values.
4. To study the behavior of stone masonry arches under high rate impact loading. The performance of stone masonry arches under impact has been simulated numerically and verified by comparing with experimental results.
5. Verification of the Finite element Model by techniques of Mesh independency and validation of results by comparison with experimental results.
6. To estimate the impact force generated by different impactors with different energy levels, impacting the arch at different points.

1.4 Methodology and Scope

This study addresses the problem first by studying a comprehensive overview about impact loads, the mechanism of impact loadings and the empirical formulation of impact loads. It also presents the significant cases illustrating the past-performances of masonry bridges impacts.

Secondly a prototype single span stone masonry arch was designed. Both dry joint and mortar joint arches were simulated both experimentally and numerically to investigate the resistance curve of the arch. A simplified limit analysis using RING software and advanced non-linear FEA using DIANA were carried to predict the failure modes and capacity curves of the arch. A displacement controlled quasi-static experimental test was conducted on the arch specimens to calibrate numerical models and to identify and define the bounds of significant parameters that affect the resistance curve.

To characterize the performance of stone masonry arches at high rate loading, a single span stone masonry arch was assessed both experimentally and numerically for impact loading. The prototype arches were tested experimentally using a drop weight apparatus. The numerical simulation was carried out using an explicit dynamic analysis software LS-DYNA.

1.5 Thesis Organization

The primary aim of this Thesis is studying the performance of stone masonry arch bridges under actions from impacts. Hence the Chapters of this report are organized in such a way that the three fundamental issues of the problem are clearly addressed. As discussed in the previous Sections, the issues are characterization of the impact mechanism and corresponding structural loads, identification of the resistance parameters to develop the capacity curve of masonry arches and evaluation of the high rate loading performance of such structural systems. A total of 7 Chapters are presented. Following this **introductory Chapter**, the **Chapter 2** briefly explains the mechanism of impact loading, impact load performance of infrastructures such as bridges and it provides background information about specifying impact actions on masonry arch bridges. **Chapter 3** discusses the different type of arches and their behavior and usages in different constructions.

Chapter 4 is devoted to numerical and experimental investigation of the behavior of the stone masonry arches. An explicit dynamic analysis, on the FE model of mortared stone masonry arch is carried out to predict the failure mode and to validate the results with the available experimental results. **Chapter 5** comprises of the information about the numerical modeling techniques used to build the FE model and brief information about the software used for carrying out the analysis.

Chapter 6 discusses the results obtained from the explicit dynamic analysis of the model stone masonry arch. The impact force assessment of the model under different impacts is followed by validation of the model with experimental results. **Chapter 7** extracts conclusion from the work presented here and recommendations for future researches are also suggested in it.

CHAPTER 2

IMPACT LOADING

2.1 Impact load

It is the dynamic effect on a structure either moving or at rest of a forcible momentary contact of another moving body. It is application of stressful pressure to an object [3].

Engineers usually have to deal with impact loading .problems related with impact load are quite difficult and different from static loading. A common case of impact loading is vehicle collision with a traffic barrier; involves coulomb friction, large displacements, elastic and plastic instability, material non-linearity, post-buckling strength and material behavior under high strain rates. Finite element methods can provide an ‘exact’ approach (in the sense that the modeling assumptions can be pulled to produce a recognizable match to test results), but considerations of a few first principles with some simplifying assumptions are required for reasonable and useful engineering estimates.

The main focus of this chapter is on the dynamic impact loads. Such loads are typically of a short duration and the response of the masonry bridges to such impact loading is an important safety consideration.

2.2 Sources of Impact loads

Generally the problems of impact loadings are extremely complex. Some common cases of impact loadings are:

a. Rock fall due to avalanches, volcanic eruption or earthquakes

b. Fragments striking after bomb blasts

c. Huge stones and trees coming with water during the floods hazard

Efforts have been made during the past decades to develop methods of structural analysis and design to resist impact loads due to the threats from such impact loading conditions, particularly from military defiance engineering departments. However the issue has become serious for civil infrastructures too after the campaigns of terrorist organizations. Hence it is mandatory to make structural assessment for historic structures and take appropriate safety measures against the probable catastrophic damage.

2.3 Examples of impact loading on Structures

The real time examples of impact loads are:

2.3.1 Impact loads of falling rocks

Infrastructures and buildings in mountainous regions are exposed to gravitational natural hazards. These risks are addressed through a variety of protection measures [17]. Significant costs associated with such measures – decisions should be made on a consistent scientific basis. A proper modeling of the processes, the performance of protection structure and the associated uncertainties is crucial. Rock-fall is an uncertain process – impossible to predict the time and extend of the next event [1]. Dynamic load extremes of rock falls on structures are dependent on the distance over which the impacting mass is stopped. Lower values are observed when braking occurs over a greater distance such as the case of a flexible wire-rope rock fall barrier. Conversely, highest peaks are observed when impacting a concrete gallery or wall with no or just a thin cushion layer. Therefore, an in-depth understanding of such impact loads is essential to properly design protection measures.

The risk of rock fall events increases due to global warming and population growth in alpine regions. Meanwhile the risk acceptance in our society decreases according to the state of our proper economic situation. Considering the high mobility requirements also in alpine regions, professionals need to improve the protection against rock fall hazards. Rock fall galleries are an efficient measure to protect roads and railways, mainly if the danger is locally concentrated. Rock falls poses a hazard in Utah because people live, work, and recreate in close proximity to mountains and mesas. Large rock fragments and boulders accelerate rapidly when dislodged from cliffs and hillsides and can cause significant damage to homes, property, roadways, and vehicles, as well as loss of life. Rock falls are initiated when rocks are dislodged by freeze/thaw action, rainfall, weathering and erosion of the rock and/or surrounding material, or root growth, and they can be triggered by ground shaking from earthquakes. Rock falls generally occur without warning, and have caused significant damage and fatalities in Utah. Rock falls are a natural process in which dislodged rock moves down slope by gravity. Dislodged rocks can travel at high velocities and cover significant distances; rocks travel down slope by bouncing, rolling, sliding, or free-fall. Rock-fall hazards are dependent upon a number of factors including geology, topography, and climate. Most rock falls originate on slopes steeper than 35 degrees, although rock-fall hazards are found on lesser slopes. Rock-fall sources include bedrock outcrops or boulders on steep slopes such as mountainsides, cliffs, bluffs, and terraces. Rock fall hazards may also exist along road cuts and other excavations. Rock falls in Utah typically occur more frequently during spring and summer months. This is likely due to spring temperature variations causing snow and ice to melt and re-freeze in rock fractures, snowmelt, and summer cloudburst storms. Rock falls occur where a rock source exists above slopes steep enough to allow rapid down slope movement of dislodged rocks.

2.3.2 Impact Load due to Debris Flow

During a tsunami or storm rush, water-borne objects (e.g., boats, oil rigs, vehicles, drift wood, etc.). In Figure2.2, an oil drilling Rig may hit a coastal bridge with tremendous impact force. This scenario involves highly nonlinear coupled fluid (tsunami or storm surge flow); structure (debris); structure (bridge component) interaction and the physics is often very complex. No simple formula exists for prediction such loads on a structure except for the simple case of a drift wood hitting a rigid wall.

Figure 2.1 Rock slide closed SR-14 in Cedar Canyon, Iron County Utah January 2009 [17]

The resulting empirical formula was based on two sets of experiments, one in a small water tank and the other for full-scale impact in air [36]. In a realistic scenario, such as the case of an oil drilling rig: the oil drilling rig was broken, loose and then hit the bridge across Cochrane-Africa town in Mobile, AL (Figure2.2), the dynamic loading on the bridge can either be estimated by hand calculation of momentum balance or by parametric study using complex finite-element model and simulation.

2.4 Impact Load Mechanism

The mechanism of impact loads is based on conservation of energy and momentum. The decelerating force on the striking body demonstrates the conservation of momentum. The kinetic energy of the impacting body will be partially converted to strain energy in the target and partly dissipated through friction and local plastic deformation and strain energy is 'radiated' away as stress waves. The details are very difficult to predict, but some simple estimates based on first principles can usually result is reasonable estimates for response [51].

The chief problem usually involves estimation of deformability. The assumption of a rigid impact is generally useless, since rigidity implies an instantaneous velocity change, therefore infinite acceleration and an infinite force. In real structures the deceleration is limited by elastic and plastic deformation, which in effect cushions the blow, and a major ‘trick’ is making a reasonable estimate the local compliance or stiffness at the point of impact.

Issues arise in estimation of deformability. In real structures the deceleration is limited by elastic and plastic deformation, which helps in estimation of the local compliance or stiffness at the point of impact. Where impact is a routine service condition, the structure should remain elastic or nearly so and a true dynamic analysis may be required. In many structural or mechanical design problems the requirement is to provide proof that the structure remains substantially intact, even though damaged. Local plastic deformation may be tolerated, provided the overall response is nearly elastic.

Figure 2.2 Oil drilling rig broken and loose

CHAPTER 3

ARCHES

3.1 Explanation of Arch

An Arch is a structure that spans a space and supports structure and weight above it. An arch is a *pure compression* form. It can span a large area by resolving forces into compressive stresses and, in turn eliminating tensile stresses. This is sometimes referred to as arch action [21]. As the forces in the arch are carried to the ground, the arch will push outward at the base, called *thrust*. As the *rise*, or height of the arch decreases, the outward thrust increases. In order to maintain arch action and prevent the arch from collapsing, the thrust needs to be restrained, either with internal ties, or external bracing, such as abutments.

3.2 Arch History

True arches, as opposed to corbel arches, were known by a number of civilizations in the Ancient Near East, the Levant, and Mexico, but their use was infrequent and mostly confined to underground structures such as drains where the problem of lateral thrust is greatly diminished. A rare exception is the bronze age arched city gate of Ashkelon (modern day Israel), dating to ca. 1850 B.C. An early example of a voussoir arch appears in the Greek Rhodes Footbridge [21]. In 2010, a robot discovered a long arch-roofed passageway underneath the Pyramid of Quetzalcoatl which stands in the ancient city of Teotihuacan north of Mexico City, dated to around 200 AD.

The ancient Romans learned the arch from the Etruscans, refined it and were the first builders to tap its full potential for above ground buildings. The Romans were the first builders in Europe, perhaps the first in the world, fully to appreciate the advantages of the arch, the vault and the dome.

Throughout the Roman Empire, their engineers erected arch structures such as bridges, aqueducts, and gates. They also introduced the triumphal arch as a military monument. Vaults began to be used for roofing large interior spaces such as halls and temples, a function which was also assumed by domed structures from the 1st century BC onwards. The segmental arch was first built by the Romans who realized that an arch in a bridge did not have to be a semicircle, such as in Alconétar Bridge or Ponte San Lorenzo. They were also routinely used in house construction as in Ostia Antica (see picture).

The semicircular arch was followed in Europe by the pointed Gothic arch or ogive whose centerline more closely followed the forces of compression and which was therefore stronger. The semicircular arch can be flattened to make an elliptical arch as in the Ponte Santa Trinita. Both the parabolic and the catenary arches are now known to be the theoretically strongest forms. Parabolic arches were introduced in construction by the Spanish architect Antoni Gaudí, who admired the structural system of Gothic style, but for the buttresses, which he termed "architectural crutches". The catenary and parabolic arches carry all horizontal thrust to the foundation and so do not need additional elements.

The horseshoe arch is based on the semicircular arch, but its lower ends are extended further round the circle until they start to converge. The first known built horseshoe arches are known from Aksum (modern day Ethiopia and Eritrea) from around the 3rd–4th century, around the same time as the earliest contemporary examples in Roman Syria, suggesting either an Aksumite or Syrian origin for the type of arch.

3.3 Arch configurations

The most common true arch configurations are the *fixed arch*, the *two-hinged arch*, and the three-hinged arch.

3.3.1 THE FIXED ARCH is most often used in reinforced concrete bridge and tunnel construction, where the spans are short. Because it is subject to additional internal stress caused by thermal expansion and contraction, this type of arch is considered to be statically indeterminate.

3.3.2 THE TWO-HINGED ARCH is most often used to bridge long spans. This type of arch has pinned connections at the base. Unlike the fixed arch, the pinned base is able to rotate, allowing the structure to move freely and compensate for the thermal expansion and contraction caused by changes in outdoor temperature. Because the structure is pinned between the two base connections, which can result in additional stresses, the two-hinged arch is also statically indeterminate, although not to the degree of the fixed arch.

Figure 3.1 Reichsbrücke in Vienna, Austria: The rollers at the pinned bases are visible in this two-hinged arch.

3.3.3 THE THREE-HINGED arch is not only hinged at its base, like the two-hinged arch, but at the mid-span as well. The additional connection at the mid-span allows the three-hinged arch to move in two opposite directions, and compensate for any expansion and contraction. This type of arch is thus not subject to additional stress caused by thermal change. The three-hinged arch is therefore said to be statically determinate. It is most often used for medium-span structures, such as large building roofs.

Figure 3.2 Rosssgraben bridge near Berne, Switzerland, showing the hinge at mid-span of this three-hinged arch

3.4 Types of arches

Arches are constructed in four basic shapes that frame and support doors, windows, porches, and other wall openings:

3.4.1 FLAT ARCH

An arch having a horizontal intrados with voissoirs radiating from center below, often built with a slight camber to allow settling is called a flat or jack arch.

3.4.2 FRENCH ARCH

A flat arch with voissoirs inclined to the same angle at each side of the center. The mortar joints do not, therefore, radiate to a common center. Not, technically, a proper arch, and of weak form.

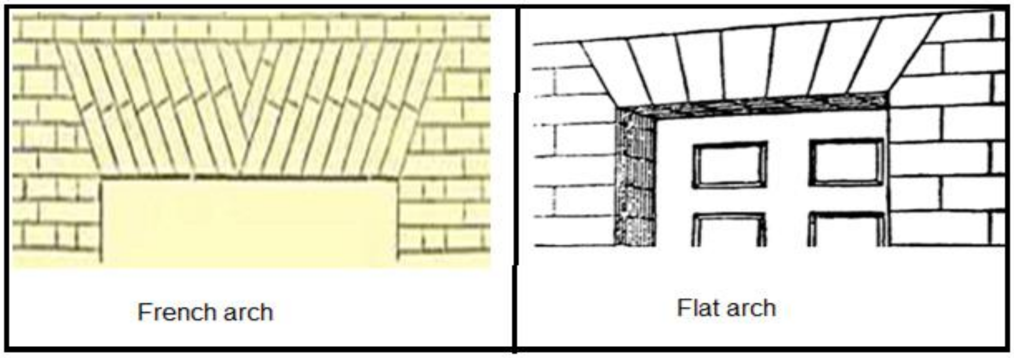


Figure 3.3 Types of arches

3.4.3 TRIANGULAR ARCH

It is a primitive form of arch consisting of two stones laid diagonally to support each other over an opening. Hence, the span is limited by the size of the available material.

3.4.4 CORBEL ARCH

A false arch formed by corbelling courses from each side of an opening until they meet at a midpoint, where a capstone is laid to complete the work. The stepped reveals may be removed but no arch action is affected.

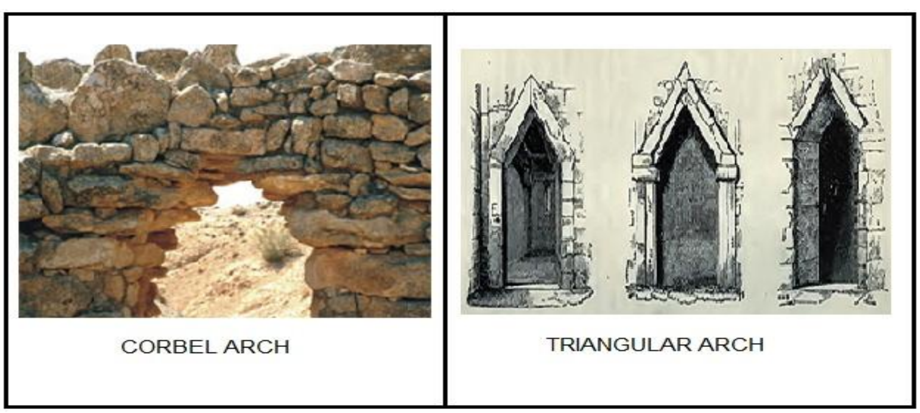


Figure 3.4 Types of Arches

3.4.5 ROUND ARCH

Round arches are generally single centered or four centered arches. Following are the various types of round arches:

3.4.6 ROMAN ARCH

A Roman arch is a strong rounded arch with a semicircular intrados. There are several examples of Roman arches in ancient Roman structures.

Figure 3.5 Types of Arches

3.4.7 SEGMENTAL ARCH

An arch struck from one or more centers below the springing point, which forms a partial curve or eyebrow. This arch is so named because it formed from a segment of a circle. It is an extremely common form of arch both in stone and in brick.

3.4.8 STILTED ARCH

An arch resting on imposts treated as downward continuations of the archivolt.

3.4.9 BELL ARCH

An arch resting on two large corbels with curved faces.

3.4.10 HORSE SHOE ARCH

An arch having an intrados that extends above the springing before narrowing to a rounded crown also known as Horse shoe or Moorish arch.

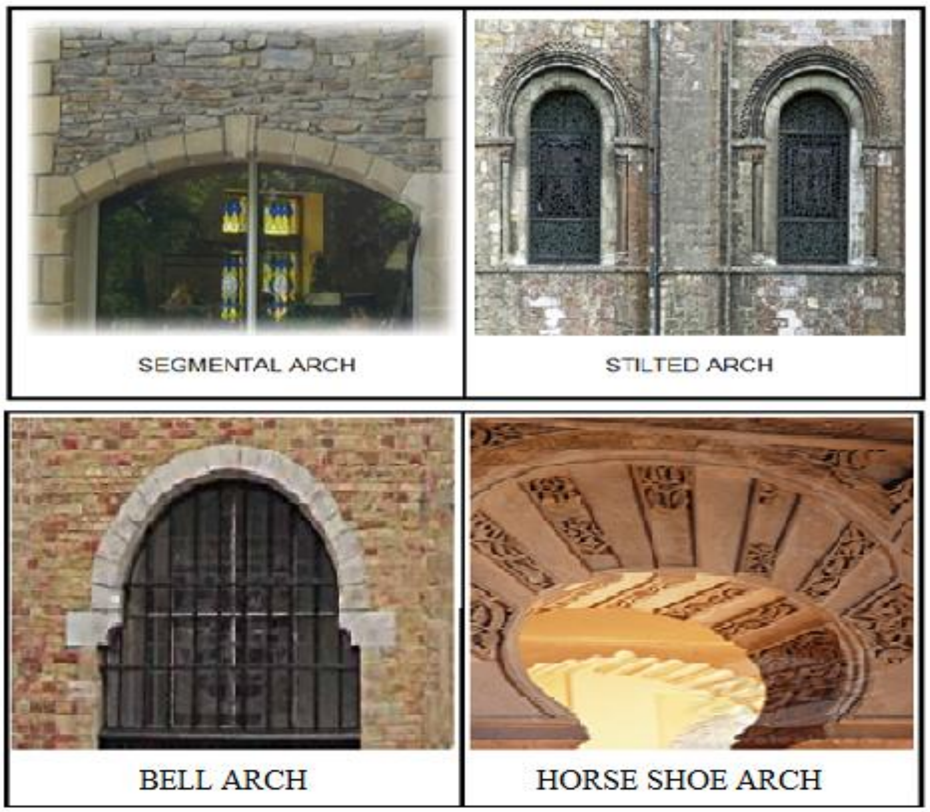


Figure 3.6 Types of Arches

3.4.11 BASKET HANDLE

A three centered arch having a crown with a radius much greater than that of the outer pair of curves also called anse de panier.

3.4.12 FLORENTINE ARCH

An arch having its extrados struck from a centre further up the central vertical axis than that of the intrados.

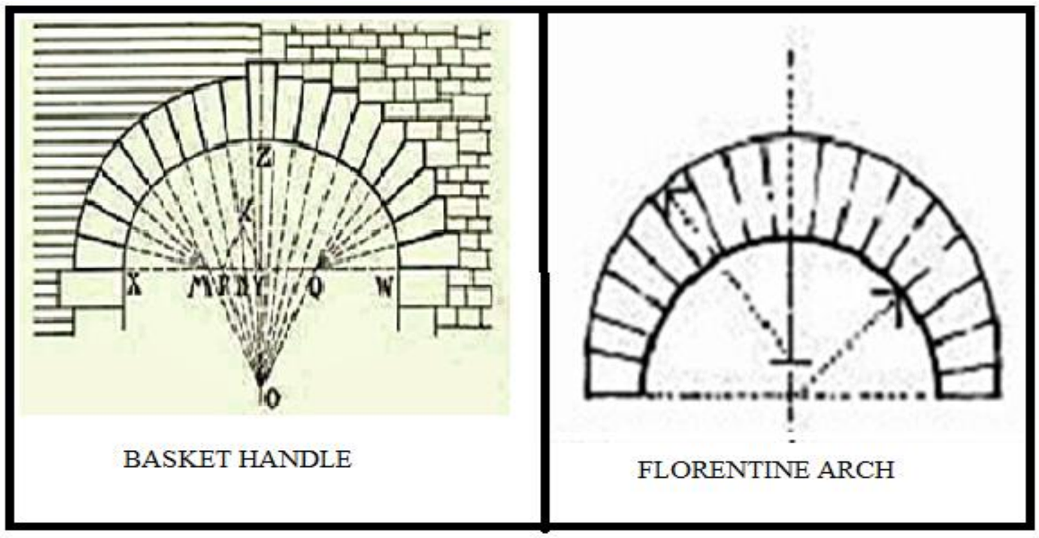


Figure 3.7 Types of Arches

3.4.13 POINTED ARCH:

3.4.13.1 GOTHIC ARCH

It is a pointed arch especially having two centers and equal radii.

3.4.13.2 LANCENT ARCH

A pointed arch having two centers and radii greater than the span.

3.4.13.3 DROP/DEPRESSED ARCH

A pointed arch having two centers and radii less than the span.

3.4.13.2 OGEE ARCH

A pointed arch, each haunch of which is double curve with the concave side uppermost.

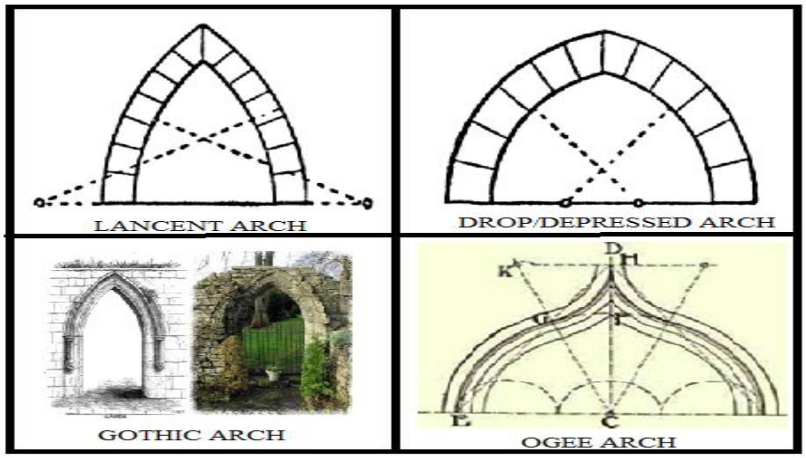


Figure 3.8 Types of Arches

3.5 Structural Principle

There are three types of behaviors which can be expected from a simple structure to counter gravity and vertical loads

3.5.1 Beam/lintel behavior

In beam/lintel behavior a bridge simply acts as a beam and that is why the load withstands due to resisting bending and the maximum stresses are in the middle of the span. That is why it is inappropriate to use it for low tensile strength materials

Figure 3.9 Lintel Behavior

3.5.2 Corbel behavior

It is the solution between beam and arch. It generates horizontal thrust at springers. They are also known as false arches because they are not entirely self-supporting structures. Because still tensile stresses left behind in corbel structure which isn't transformed in to compressive stresses and if thickened walls and abutments are not provided on the sides they will tend to collapse each side if the archway inwards.

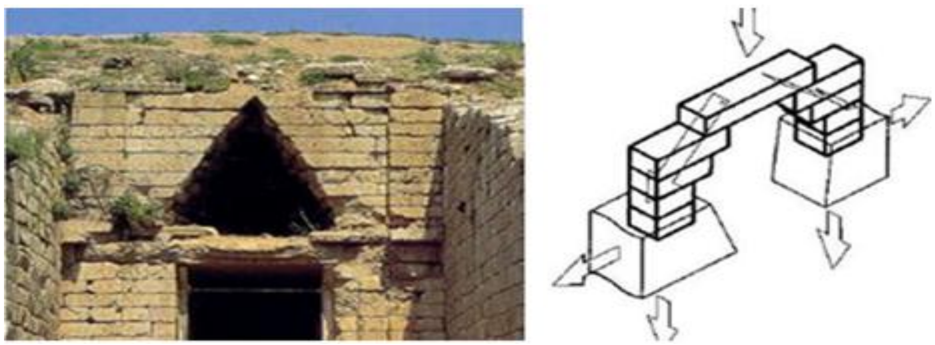


Figure 3.10 Corbel behavior

3.5.3 Arch behavior

There is pure compression stresses in arches while tensile stresses are completely eliminated. As the forces in the arch are carried to the ground, the arch will push outward at the springer, called *thrust*. As the *rise*, or height of the arch decreases, the outward thrust increases. That is why arches are the ideal shape for low tensile strength material (e.g.) masonry.

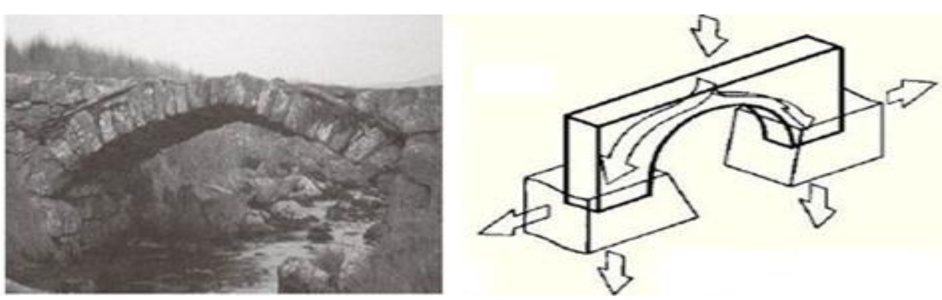


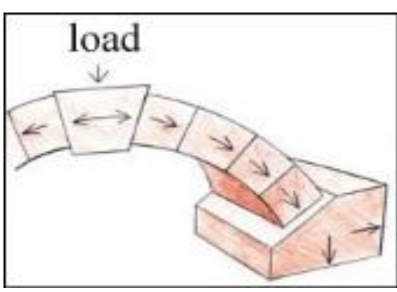
Figure 3.11 Arch behavior

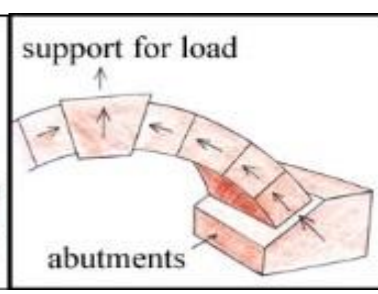
3.6 Load mechanism in arches

Arch takes load by combined action of axial thrust and flexure as a whole. And among different sections of the arch (stone and joints) these stresses occurs with different intensity Compressive, Tensile and Shear stresses. The feature which makes arches masonry arch a unique kind of structure is that It can span a large area by resolving forces into compressive stresses and, in turn eliminating tensile stresses which is often known as arch action which gives them a great natural strength that's why there are still structures n bridges around which are thousands of years.

Instead of pushing straight down like other structures, the weight of an arch bridge is carried outward along the curve of the arch to the supports at each end. These supports, known as abutments [21], carry the load and keep the ends of the bridge from spreading out. This is why when supporting its own weight and the weight of crossing traffic, every part of the arch is under compression.

The load at the top of the key stone makes each stone on the arch of the bridge press on the one next to it. This happens until the push is applied to the abutments, which are embedded in the ground. While the ground around the abutments is squeezed and pushes back on the abutments .For every action there is an equal and opposite reaction. The ground which pushes back on the abutments creates a resistance which is passed from stone to stone, until it is eventually pushing on the key stone which is supporting the load.





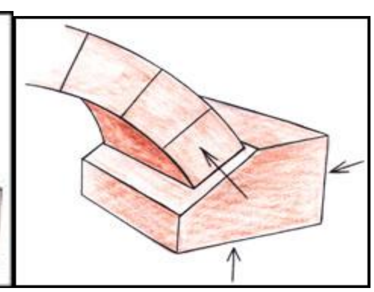


Figure 3.12 Loads Figure 3.13 Reaction Figure 3.14 Abutments

3.6.1 Keystone

One of the important features in stone arch is keystone. It is wedge shaped peace placed at the apex of arch and in construction it is placed in the end. Its one major function is to lock all the stones pieces at their position to hold the arch together otherwise the arch would collapse.

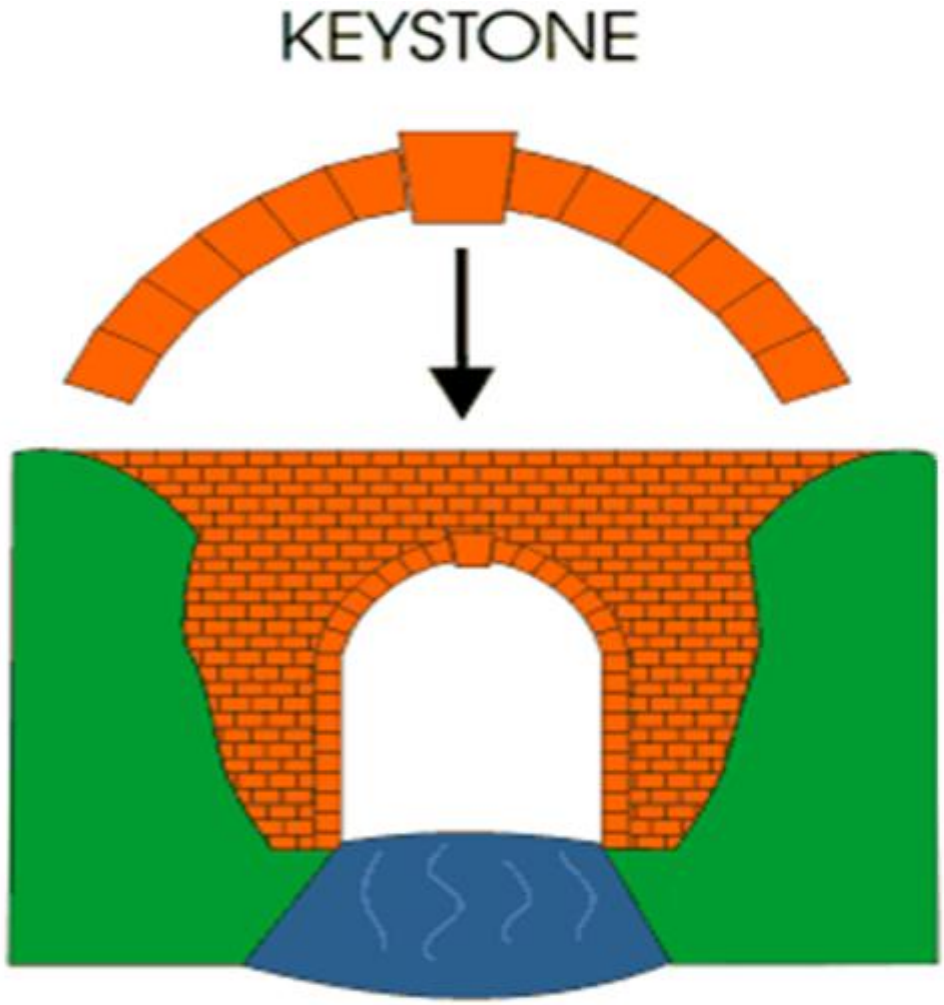


Figure 3.15 Keystone

CHAPTER 4

MODEL ANALYSIS (SINGLE SPAN ARCH)

4.1 General

The key determination of this Chapter is to Study the impact load capacity of stone masonry arches models. In order to investigate whether or not a dynamic enhancement existed at global label, it is necessary to predict the impact load force deformation capacity of a structure. Moreover, this will gives a bound to the energy absorbing capacity of the arch, the peak static strength, and both pre-peak and post- peak effective stiffness of the structure.

Impact loads, just like blast loads arise from interactions of systems that occur at very high rate. The interaction involves contact between the systems and results in the transmission of kinetic energy from one body to the other. These interactions induce a very high strain rates (loading rates), which are highly transient and impulsive, in the target structures (one of the interacting systems). However the second load type, i.e blast loads, arises when a very fast moving blast air engulfs the structure here as the first loading type occurs when a fast moving projectile hits the structure

To characterize the performance of stone masonry arches at high rate loading, a single span stone masonry arch was assessed numerically for impact loading. The numerical simulation was carried out using explicit dynamic analysis software ANSYS. Finally a calibration of the numerical model was calibrated using the experimental results obtained from research papers.

4.2 Modeling of Arch

There are three modeling techniques by which a compuer based FE model of a stone masonry arch can be created.

1) Single uniform structural model of arch
2) Arch with stone blocks and Dry joints
3) Arch with stone blocks and Wet joints

4.3 A Prototype Arch model

A prototype stone masonry arch was designed for impact response simulation. For this reason, the global dimensions of the arch were fixed based.

The specification of the span, L, the rise, H, and the out of plane thickness, W are primarily based on the geometric constraints. The radial thickness of the arch ring is also given. Moreover the radial thickness-span ratio, T/L, and the span-rise ratio, H/L, was adjusted to conform to the ranges set from past experience

4.4 Dynamic Material Properties

The approximate ranges of the expected strain rates for various loading conditions are shown in the Figure 4.1. It can be observed that normal static strain rate is located in the range of (10e-6-10e-4) s^-1, whereas dynamic loads yield strain rates in the range: (10e-4-10e4) s^-1.

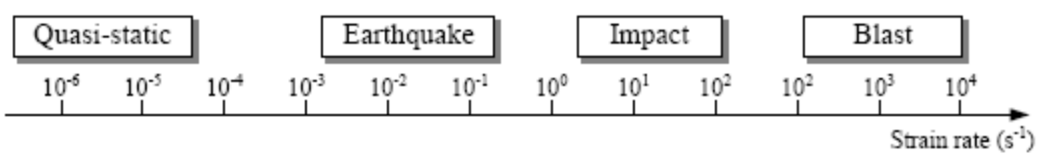


Figure 4.1 The approximate ranges of the expected strain rates

This typically high straining (loading) rate would change the dynamic mechanical properties of encountering structures and, accordingly, the expected structural performance of numerous structural elements. The strength enhancement of structures at high rate loading with that of the static loading conditions is referred as Dynamic enhancement factor (DIF). For masonry structures subjected to this high rate loading, the strength of the units and the joints can increase significantly due to strain rate effects[8].

4.4.1 Properties of Stone Blocks

The mechanical properties of stone blocks can be fairly dissimilar from that under static loading and under dynamic loading conditions. Stiffness or strength of granite stones has not been found. However it can be well predicted from that of concrete and brick.

Based on dynamic uniaxial compressive tests conducted (by Wei and Hao) to study the strain rate effect of brick, the dynamic increase factor (DIF) for strength of brick material was formulated as [47]

$$DIF = \begin{cases} 0.0268 ln\dot{\varepsilon} + 1.3504 \; for \; \dot{\varepsilon} \leq 3.2 s^{-1} \\ 0.2405 ln\dot{\varepsilon} + 1.1041 \; for \; \dot{\varepsilon} > 3.2 s^{-1} \end{cases} \tag{4.1}$$

4.4.2 Properties of stone masonry Joints

In a stone masonry arches, the joints may be dry or mortared. In case of dry-jointed masonry, the blocks are stabilized by traction forces which are friction and normal thrust forces.

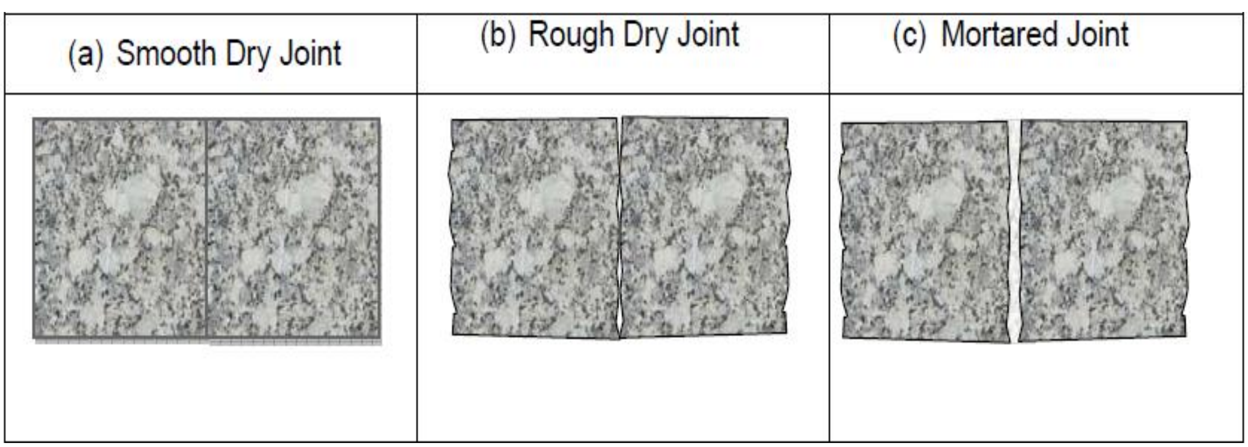


Figure 4.2 Dry and mortar joints

The structural behavior of stone masonry joint is described in the form of a relation between the normal and shear tractions and the normal and shear relative displacements along the interface.

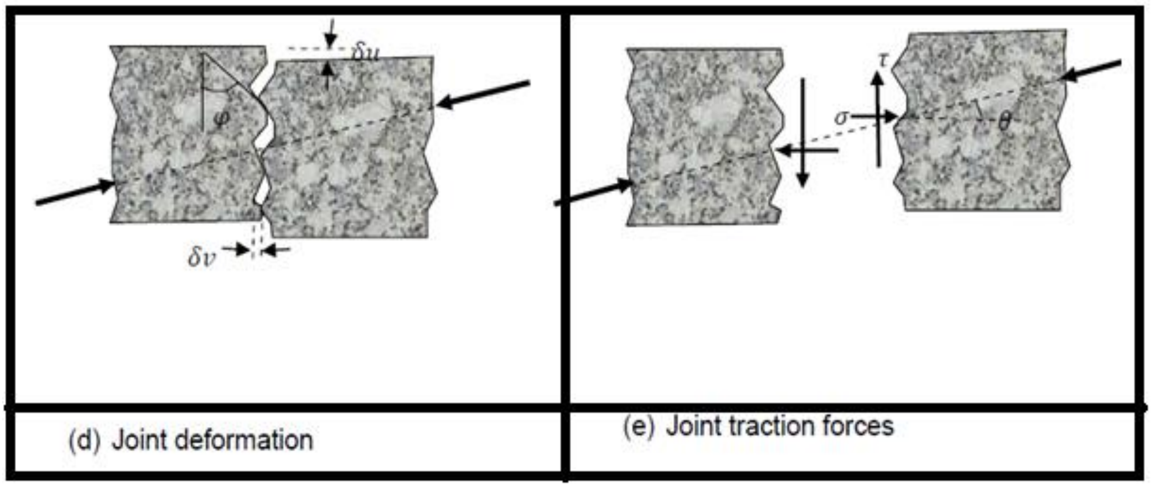


Figure 4.3 joint deformation and traction forces

In Coulomb friction model, two surfaces in contact can carry shear stresses up to a certain level of magnitude through their interface before the sliding starts relative to each other. This state is known as pre-peak behavior or sticking.

For dry joints, this is insured by interfacial interlock-characterized by coefficient of friction and the normal contacting stress. For the mortared joints, the resistance is assisted by the interfacial adhesion (cohesion).

Material	**Density (kg/m^3)**	**Modulus of elasticity (GPa)**	**Poison ratio**
Stone (granite stone)	**2500**	**35-50**	**0.2**
Mortar (lime)	**1700-2100**	**0.15-0.25**	**0.1**

Table 4.1 Material properties of stone and mortar

4.4.3 Properties of Dry-Joint Stone interfaces

In the simulation of masonry joints in which we uses explicit finite element code LS-DYNA3D, the interfaces are represented by means of specialized contact surface formulations. Dry joints are simulated using surface to surface contact formulations and mortared joints are simulated using tied surface to surface formulation. Both formulations are based on Mohr Coulomb failure criteria as discussed in Chapter 3 with two exceptions. Primarily the softening of friction coefficient depends on the relative velocity of the surfaces in contact, in which the peak coefficient of friction is referred as coefficient of friction for static and the residual friction coefficient as dynamic. Typically, coefficient of friction for static is higher than the coefficient of friction for dynamic. ANSYS provides the following exponential decay friction model (see Figure 4.2).

$$\mu = \mu_d * \left(1 + \left(\frac{\mu_s}{\mu_d} - 1\right) e^{-D\dot{v}}\right) \qquad 4.2$$

Where:

μ_s =coefficient of friction for static

μ_d =coefficient of friction for dynamic

v =slip rate (interface tangential velocity) in units of length/time.

D = decay coefficient, has units of time/length. When D is zero, the equation is revised to be μ = μ (d) in a case of sliding and sticking. Secondly cracking and gaping failure mode in the tensile regime is simulated using tied surface to surface contact formulation. In this methodology, nodes on the surface of masonry unit remain tied to the surface of an adjacent unit at the same time as failure criterion (Eq. 4.1) is satisfied, after which the surfaces are free to separate or slide with friction.

$$\left(\frac{\tau}{c}\right)^2 + \left(\frac{\sigma}{f_t'}\right)^2 \leq 1 \qquad 4.3$$

Where

Ʈ=shear interface stress

C= cohesion

σ =normal interface stress (tensile) and

f'_t= tensile strength of the joint

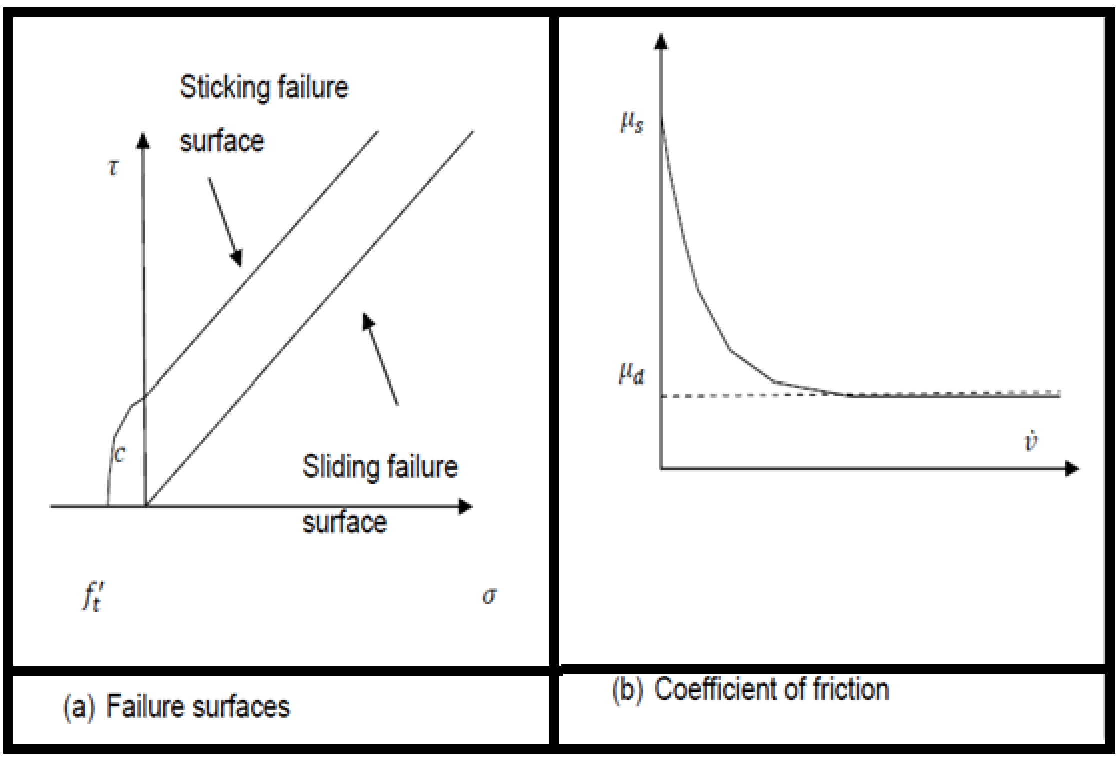


Figure 4.4 Failure surfaces and coefficient of friction

Masonry that is subjected to high stress rates shows, enhanced bond strength due to the finite crack propagation velocity [24]. It has been reported that the strain rate sensitivity of mortar appeared to be a function of water content. Shrinkage problem can lead to the formation of micro cracks and voids within the mortar matrix. However, the formation of micro cracks and voids may not be uniform over a cross-section because moisture evaporates more easily from the perimeter. And hence it might be expected that the tensile failure stress at the perimeter will be less than at the center. If a masonry specimen is subject to static load, stresses are free to progressively redistribute in the cross-section prior to complete failure occurring. However, when dynamically loaded, stresses in a specimen will not have time to redistribute and consequently failure may effectively involve simultaneous mobilization of all bonds at the unit–mortar interface.

It has been reported that brick-mortar joints subjected to tensile load at strain-rates of approximately 1 have an apparent dynamic enhancement of the bond strength with DIF equal to 3.1(24). Taking the strain rate of as the reference static strain rate, DIF for ultimate compressive strength of mortar was formulated as (20)

$$DIF = \begin{cases} 0.0372 ln\dot{\varepsilon} + 1.4025 \; for \; \dot{\varepsilon} \leq 13s^{-1} \\ 0.3447 ln\dot{\varepsilon} + 0.5987 \; for \; \dot{\varepsilon} > 13s^{-1} \end{cases} \qquad 4.4$$

Hence based on the aforementioned works, DIF for bond strength of mortared joints can be assumed to vary from 1.44 to 3.1 for impact loading conditions.

CHAPTER 5

RESEARCH AND ANALYSIS METHODOLOGY

5.1 load capacity assessment

Three pronounce method for calculating the load capacity of an arch are.

1) Hand based methods
2) Computational limit analysis
3) Advanced nonlinear tools based on FEM

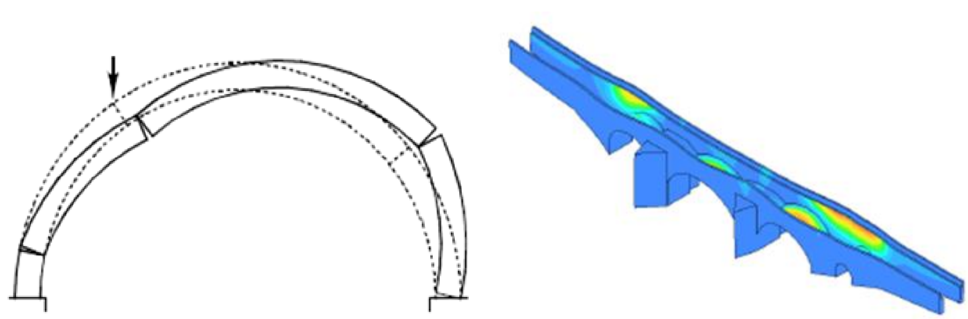


Figure 5.1 Analysis methods

5.2 Computational Limit Analysis (kinematic approach)

Limit analysis potentially provides a highly effective means of verifying the safety of structures and has successfully been applied to masonry arch bridges for many years. Hand based limit analysis techniques have been largely superseded by computer based methods which are the primary focus of this paper. Recent developments to ‘thrust line’, discrete ‘rigid block’ and various combined soil-arch interaction limit analysis models for masonry arch bridges are discussed and areas where further work is required are identified

Basically in limit analysis structure is transformed in an admissible mechanism, through the introduction of plastic hinges. Assumptions are made that sliding and crushing does not occur. Virtual work method is used to calculate a load factor and from that smallest load factor is obtained which provides the collapse load.

The theorems of plastic limit analysis require satisfaction of certain conditions and before proceeding further it is worthwhile to simply state these:

a) Equilibrium condition: The computed internal actions must represent a state of equilibrium between the internal and external loads (the corollary of the equilibrium conditions are compatibility conditions, which should instead be satisfied if an energy method is being used).
b) Mechanism condition: Sufficient releases must be made to transform the structure into a mechanism.
c) Yield condition: The stresses in the material must be less than or equal to the material strength (e.g. shear, crushing and tensile strength limits must all be respected).

To perform limit analysis, masonry is molded as assemblages of right blocks connected through joints. And the limit load occurs at small overall displacements. In this theory, the hypothesis is that masonry has zero tensile strength and shear failure at the joints is perfectly plastic. Hinging failure mode at a joint is perfectly plastic. Hinging failure mode at a joint occurs for a compressive load independent from the rotation. In kinematic approach geometry and load distribution are known then limit analysis is applied with a reduced number of parameters are applied due to which failure mechanism (failure pattern) and load magnitude (limit load) are computed

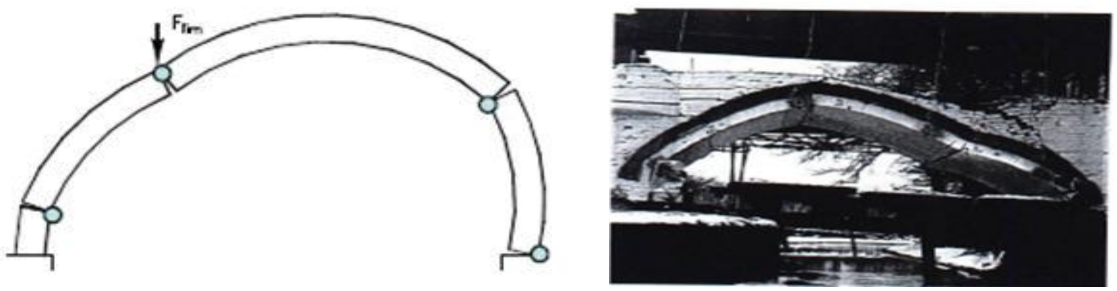


Figure 5.2 limit analysis

5.3 Numerical Structural Capacity Assessment (Non-Linear FEA)

The non-linear behavior of the prototype arch under impact loading was simulated using a finite element software package, ANSYS. A simplified discrete-crack finite element modeling approach has been adopted to model the non-linear impact load behavior the arch [8, 21]. The model was developed using linear elastic plane stress elements for masonry. A displacement based non-linear static analysis was carried out to predict the failure mode of the arch in advance of the experimental work.

5.4 Finite Element Modeling (Analysis)

Finite Element modeling is one of numerous numerical methods that is used to solve complex problems and is the prevailing method used today. As the name infers, it takes a complex problem and breakdowns it into a finite number of simple problems.

Finite element analysis (FEA) is a computerized technique for showing how a product reacts to real-world forces, heat, fluid flow, vibration, and other physical effects. Finite element analysis elaborates whether a product will break, wear out, or works same as it was designed. FEM is basically a product development process; it is tool to predict what will happen to product when the product is used.

FEA working is breaking down a real object into a large number (thousands to hundreds of thousands) of parts specifically called as finite elements, such as little cubes. Mathematical equations are used to predict the behavior of each element. A computer then combines all the individual behaviors to predict the behavior of the actual object.

In many physical effects as following, FEM is a tool that helps to predict the behavior of products.

a. Mechanical stress
b. Heat transfer
c. Fluid flow
d. Mechanical vibration
e. Motion
f. Electrostatics
g. Plastic injection molding

5.5 Software used for Structural Simulation & Finite Element Analysis

There are several software's used for finite element analysis. these are some major software's ; ANSYS is used in case when there is Extreme Loading for Structures Software made by Applied Science International for non-linear dynamic structural analysis, blast, seismic, progressive collapse, impact and other loading.

a. ADINA which is finite element modeling software for structural, heat transfer, fluid, and multi-physics electromagnetic problems, including thermo-mechanical and coupling fluid-structure interaction.
b. LS-DYNA (By: LSTC - Livermore Software Technology Corporation)
c. LUSAS MADYMO (By: TASS - TNO Automotive Safety Solutions)
d. Range Software a Multi-physics simulation software
e. SAMCEF (CAE suite developed by the Belgian company)
f. SAP2000 mainly for civil engineering structures

g. Visual FEA it is Korean software for structural and geotechnical analysis.

h. RING software which is also used for FEM.

i. We are working with ANSYS for structural simulation and finite element analysis of a stone masonry arch due to impact loads.

5.6 ANSYS

ANSYS is professional software that offers engineering model solution sets in engineering simulation that is required by a design process. Various industries companies use ANSYS software. This software put a virtual task through a rigorous testing procedure (for example crashing a car into a brick wall, or running for several years on a tarmac road) before it converts to a physical object. Ansys is used as general purpose software, to simulate dealings of all branches of physics, structural, vibration, fluid dynamics, heat transfer and electromagnetic for engineers.

So ANSYS, which aids to simulate tests or working situations, enables to test in virtual atmosphere before manufacturing samples of products. In addition, in order to determine and improve weak points, to compute life and foreknowing probable problems are possible by 3D simulations in virtual atmosphere This software with its segmental structure gives a chance for taking only needed features. It can work incorporated with other useful engineering software on desktop by adding CAD and FEA linking modules. It can also import CAD data and also enables to build geometry with its "preprocessing" abilities. Similarly in the same preprocessor, finite element model which is required for computation is generated. Results can be viewed in numerical and graphical interface, after defining loadings and running analyses. It can perform advanced engineering analyses safely, quickly and practically by its diversity of contact algorithms, time based loading features and nonlinear material models. Another tool in ANSYS is Workbench, which is basically a platform which assimilates simulation technologies and parametric CAD systems with unique mechanization and performance. The power that is used by ANSYS Workbench derives from ANSYS solver algorithms with years of experience. Furthermore, one of the objects of ANSYS Workbench is confirmation and improving of the product in virtual atmosphere. Workbench, which is engraved for high level compatibility with especially PC, is more than an interface and everyone who has an ANSYS authorization that is a license can work with ANSYS Workbench. As a result, Ansys abilities are controlled due to license.

Chapter 6

NUMERICAL RESULTS

6.1 Numerical Impact Response Simulation (ANSYS)

Numerical methods for solving structural responses resulting from interactions occurring at high rate, such as impact, are generally divided into those used for prediction of actions (loads) experienced by interacting bodies and those for calculation of the resulting structural responses (action effects). The methodologies are generally categorized into uncoupled and coupled analyses.

In uncoupled analysis the actions are calculated as if the interacting structures were rigid and then applying these loads to a responding model of the structure of interest. When the actions are estimated with a rigid model of the structure, the loads on the structure are often over-predicted, particularly if significant motion or failure of the structure occurs during the loading period. For example, the blast load on structures is simulated as a transient pressure, and the response of structures under impact is estimated using impact factors for the static responses. Examples of computer codes for such simulations are DIANA, and ANSYS.

For a coupled analysis, both the interacting bodies are modeled simultaneously to estimate the action effects within the components of each system. For example in estimating the effect of explosions on structures, the CFD (computational fluid mechanics) model for blast-load prediction is solved simultaneously with the CSM (computational solid mechanics) model for structural response. Whereas for impact response of structures, both the colliding systems are modeled explicitly being interlinked by superficial interface (contact elements in FE formulation). Computer codes, which are used to evaluate the responses by explicit model of interacting bodies, are AUTODYN, LS-DYNA.

For this study, the impact response of prototype arch was simulated using coupled analysis in which the arch and the drop weight were modeled explicitly and solved simultaneously.

6.1.1 DEVELOPMENT OF FEM MODAL

The arch is made up of 13 discrete stone block units joined together by wet mortared joints. The height of the model arch is 0.4m, and the clear span length is 1.2m. The radial thickness of the arch is 0.16 m, while the out of plane thickness of the arch is 1.2 m. The model impactor is cylindrical, made up of steel, with radii calculated with the help of mass and density of the steel. The arch is fixed a both supports. Contact surface is automatically generated with built in ANSYS functions. The initial velocity given to the nodes of impactor are calculated via height of drop. The properties given to the impactor and arch elements are discussed in the previous chapter.

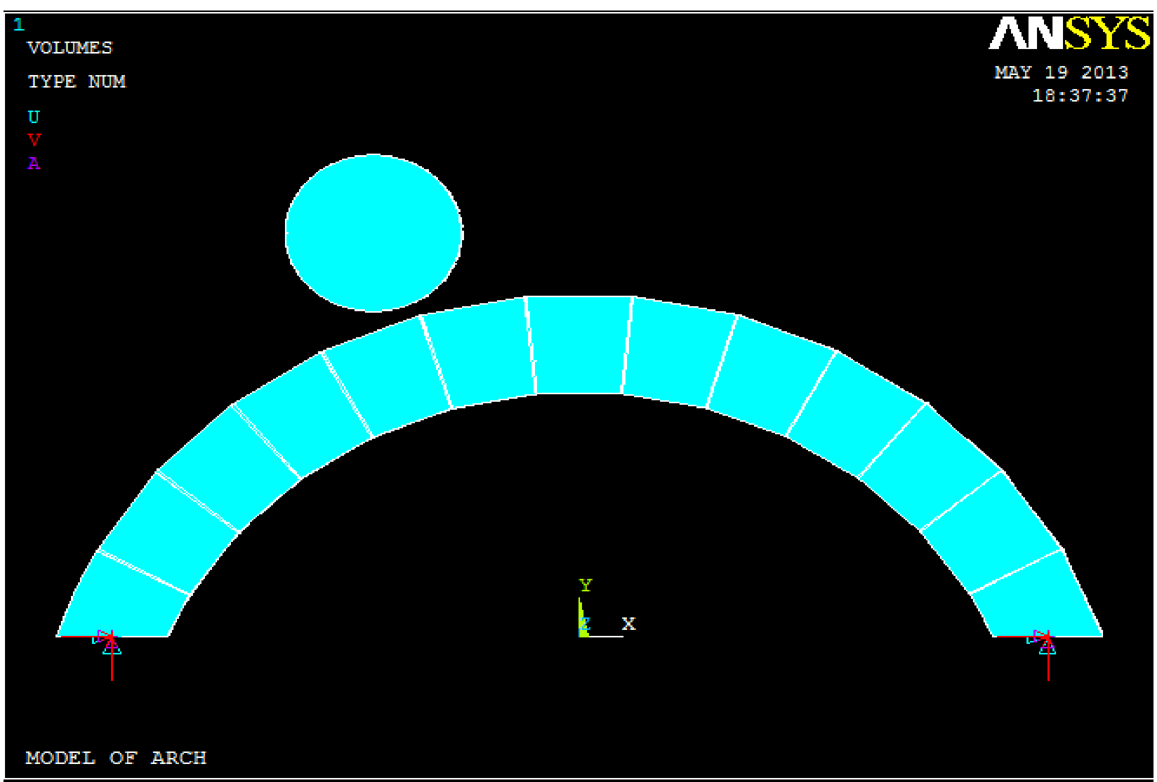


Fig 6.1 Geometry of model arch

Test setup	Height (cm)	Weight (kg)	Initial velocity (m/s)	Target kinetic Energy (j)
M60H70	70	60.9	3.71	419.11
M80H45	45	80	2.97	357.57

Table 6.1: Test setups for impact loading experimental and FEM (Quarter Span)

Test setup	Height (cm)	Weight (kg)	Initial velocity (m/s)	Target kinetic Energy (j)
M60H70	70	60.9	3.71	419.11
M80H45	45	80	2.97	357.57

Table 6.2: Test setups for impact loading experimental and FEM (Mid Span)

6.1.2 Structural Modeling and Analysis

An explicit dynamic analysis was carried out to predict the failure mode and capacity of the prototype arch under impact in advance of the experimental work. The non-linear behavior of the prototype arch under impact loading was simulated using the 3D non-linear explicit dynamic analysis program, LS-DYNA (integrated with ANSYS). A simplified discrete-crack finite element modeling approach has been adopted to model the non-linear performance the arch under impact load. The model was developed using linear elastic solid elements for masonry units and impactor block, and using contact elements for all interfaces in the system. The detailed description of the modeling and analysis procedure is given as comment in source code of the model (Appendix). The 3D model is shown in the Figure 6.2.

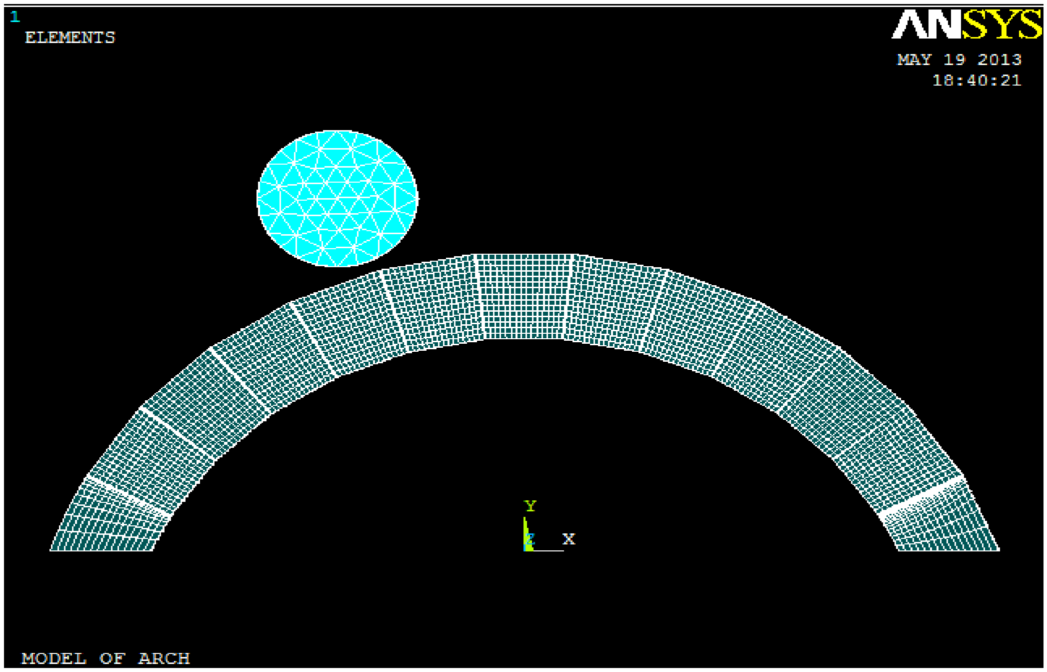


Fig 6.2 Meshed geometry of arch

Analysis for Impact- following the dynamic relaxation analysis for gravity loading, an explicit dynamic analysis was carried out applying base acceleration to both the arch and the impactor block.

6.2 Validation of Structural Models

6.2.1 Mesh independency

For the validation of the geometry of the model arch, technique of mesh independency is used. In this technique, solution is done repeatedly over the same boundary and loading conditions, while changing the mesh size. If the results such as deflections and stresses change with variation in mesh size, then the geometry is flawed, if not, then the geometry is correct.

In this arch model, solutions were carried out for self-weight over varying mesh sizes, and deflections were noted for each mesh size. Maximum deflection noted at the mid-span for each case is 0.000682m and the Von Mises stress at the mid span-intrados is 841 Pa.

There was no change in the deflections; hence the geometry has no flaws in it.

6.2.2 Stresses in arch under self-weight

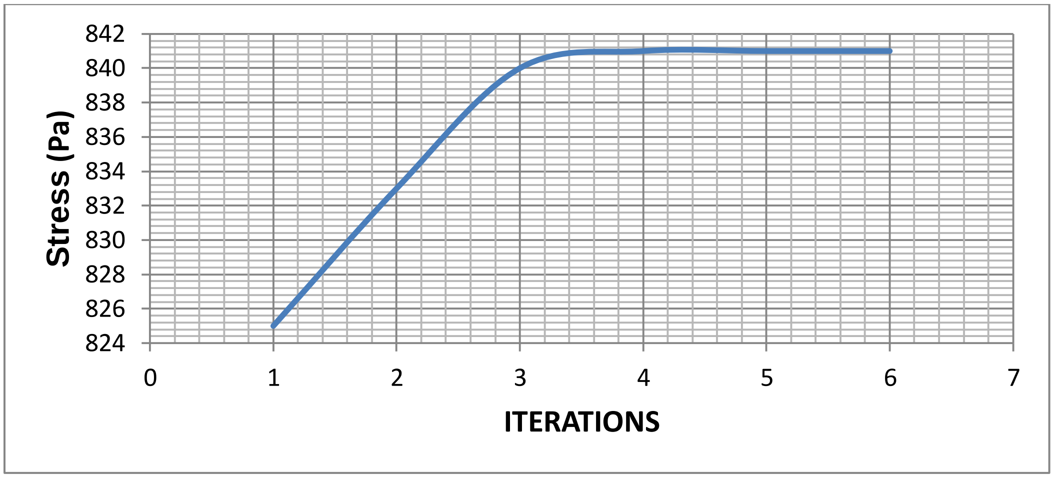


Figure 6.3 mesh independency curve

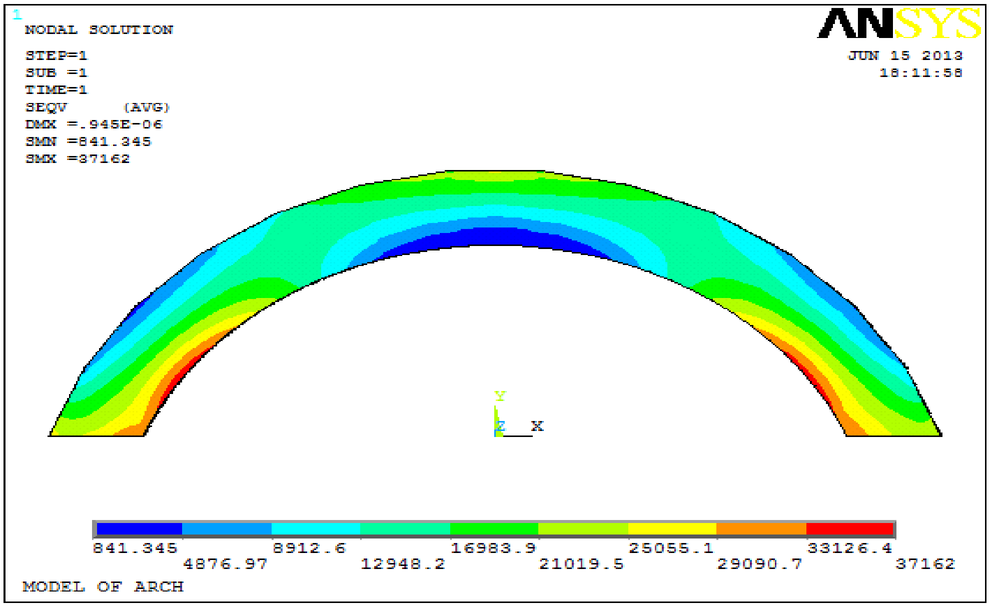


Figure 6.4 stress in arch under self-weight

6.2.3 Comparison with experimental results

To validate the numerical model parameters, a calibration of the experimental results from the study of "Paulo B. Lourenço and Tibebu Hunegn Birhane" and numerical results was made for the specimen M60H70 (mortared masonry arch subjected to mass of 60.9kg dropping from a height of 70cm). The failure mechanism of the masonry arches observed during the experimental test was identical to that of the FE nonlinear explicit dynamic analysis using ANSYS as shown in the Figure (6.5) and Figure (6.6). One rotational hinge and one sliding hinge were prominently observed at the same location as that of the numerical simulation. The rotational hinge was formed near the left support while the sliding action was suppressed due to the thrust because the thrust force goes to the near support predominantly. While near the right support, a sliding hinge was formed. There is a satisfactory agreement between the experimental and the computed damage mode.

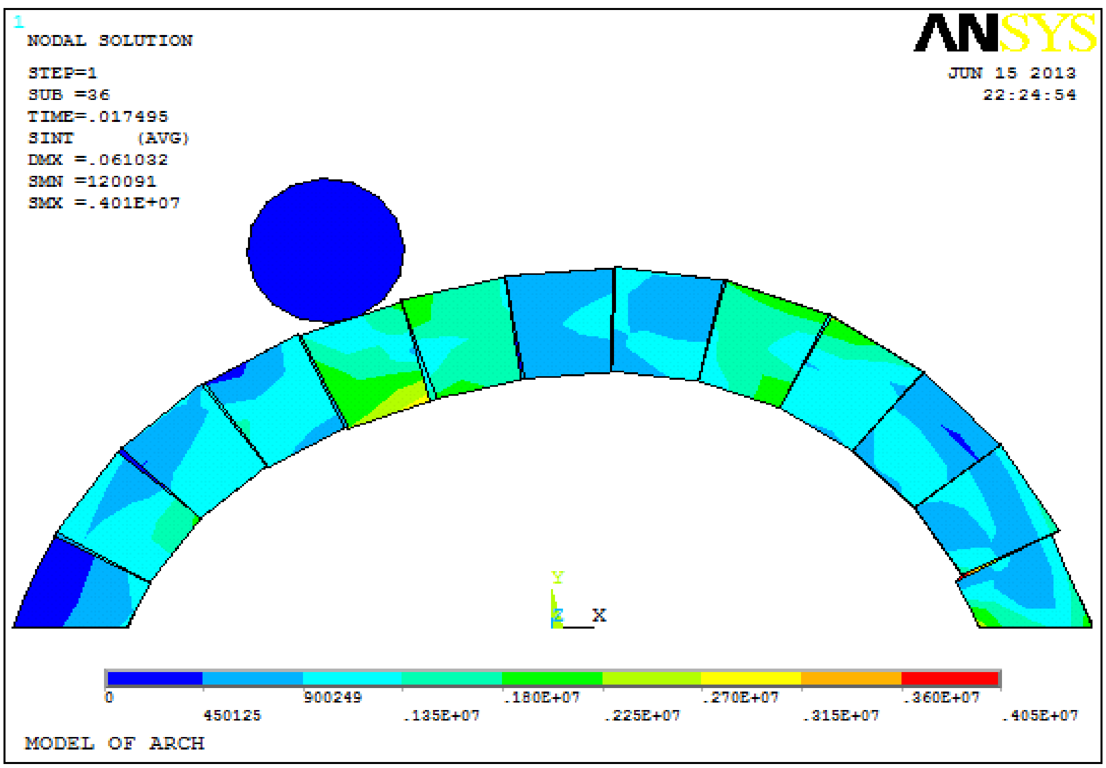


Figure 6.5 Stress in Impact model M60H70

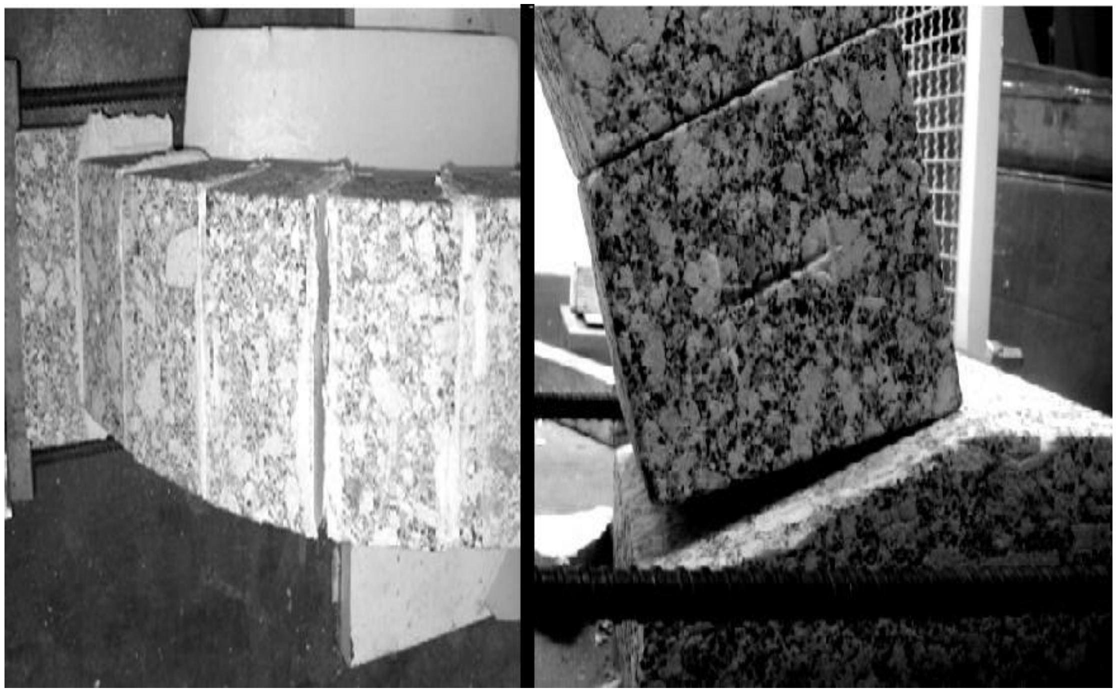


Figure 6.6 Hinges formed in experiment (1 rotational (left) + 1 sliding (right))

The difference in the maximum deflection in the experimental results and the model results is 15%, which is satisfactory because numerical modeling always considers ideal conditions and there is a difference between the impact load transfers to the arch in both cases. In the experiment, load is transferred to the arch indirectly by placing a wooden pad on it, while in the FE model; load is transferred to the arch directly.

From the two graphs below, the deflection in the experimental results of mortared masonry arch subjected to mass of 60.9kg dropping from a height of 70cm is around 32 mm, while in the corresponding finite element model subjected to the same impact load, the maximum deflection comes out to be around 27 mm, which is close enough to the experimental results.

EXPERIMENTAL

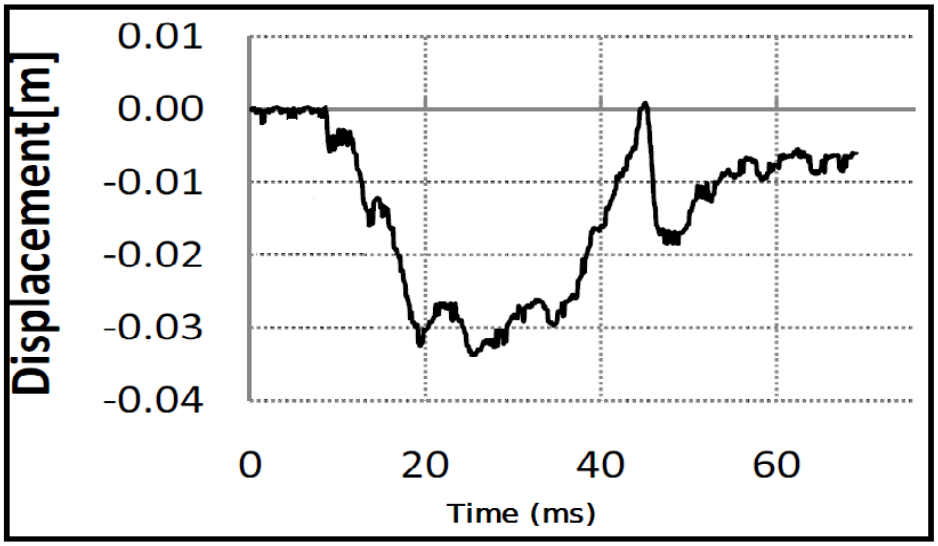


Figure 6.7 Experimental deflections of M60H70

FINITE ELEMENT MODEL

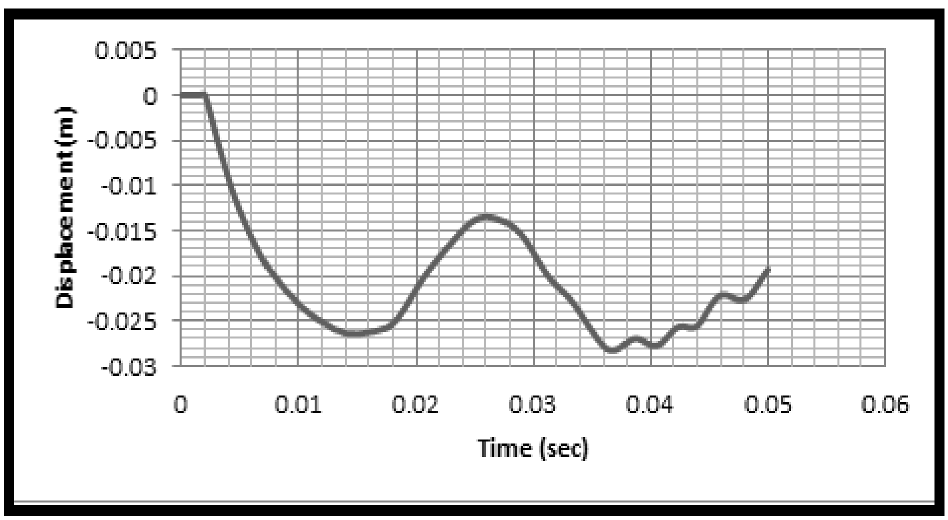


Figure 6.8 FEM deflection of M60H70

6.3 Results and Discussion

The results of numerical simulations of the pilot models, with material properties and loading conditions described in Appendix will be presented.

6.3.1 Impact force at quarter span (M60H70)

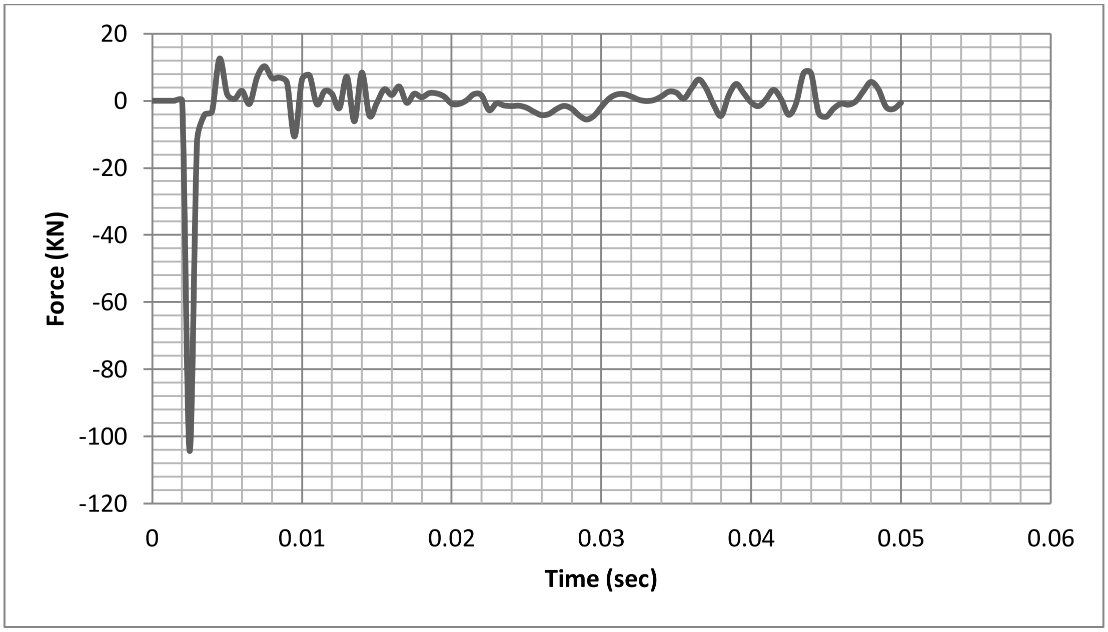


Figure 6.9 Impact force at quarter span (M80H75)

6.3.2 Impact force at quarter span (M80H75)

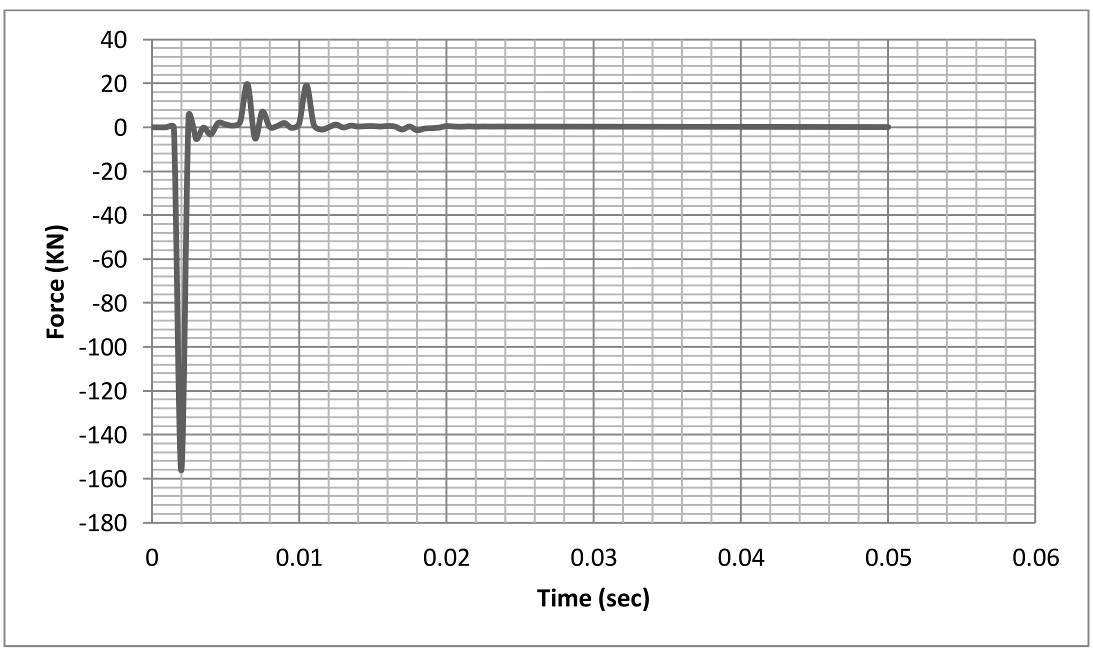


Figure 6.10 Impact force at quarter span (M80H75)

6.3.3 Deflection at mid span (M60H70)

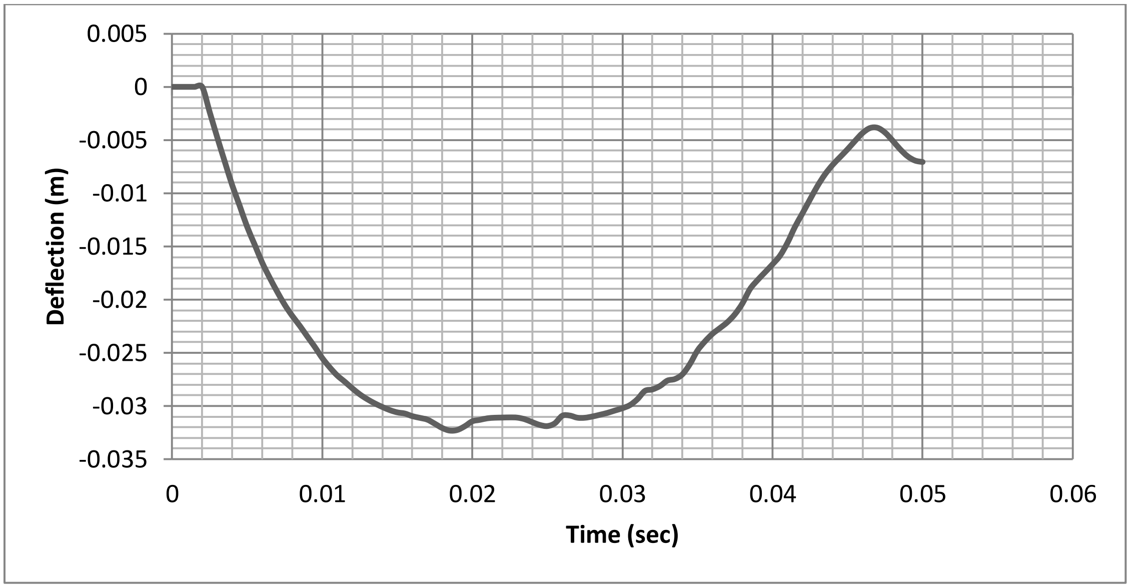


Figure 6.11 Deflection at mid span (M60H70)

6.3.4 Impact force at mid span (M60H70)

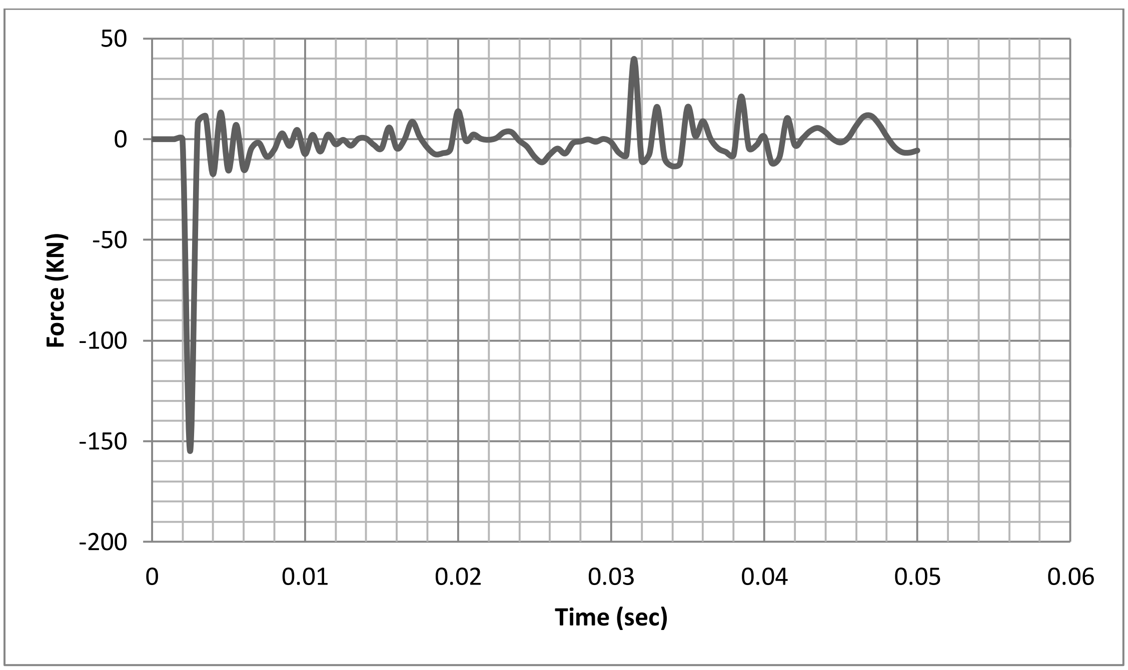


Figure 6.12 Impact force at mid span (M60H70)

6.3.5 Impact force at mid span (M80H45)

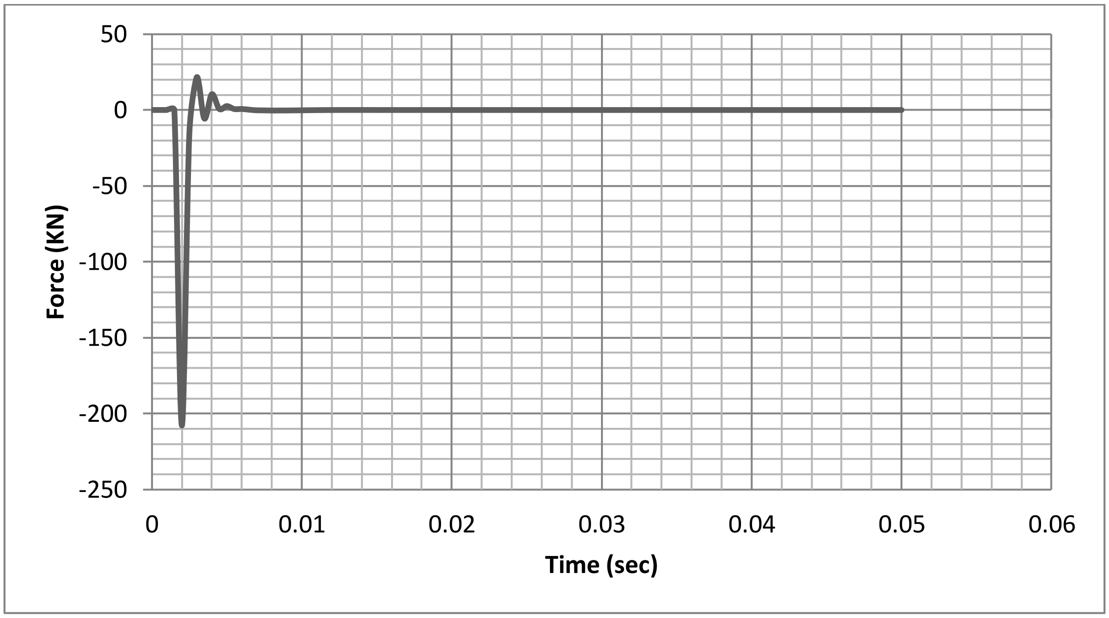


Figure 6.13 Impact force at mid span (M80H45)

From the above graphs, it is obvious that with increasing the mass and the drop height of the impactor, impact force is increased. It is also observed that with the same height and mass of the impactor, more impact force in y-direction is generated if the impact is done on mid span as compared to the case when impact is done on quarter span. The reason for this is that when the impact is done on mid span, the impactor exerts all of its kinetic energy downwards on the arch, which produces a large impact force vertically downwards, and hence more deflections, while when the impactor hits the arch at quarter span, some of its kinetic energy is directed towards moving the arch horizontally due to geometry of the arch, in which case, we do not get as much impact force as was the case with mid span impact.

Another reason is when the impactor hits the arch at quarter span, some sliding and rolling takes place due to the geometry and hence some impact energy is dissipated.

CHAPTER 7

Conclusions and Recommendations

7.1 Conclusions

In this Thesis work, a numerical study is conducted to investigate the performance of stone masonry arches at low velocity impact loading resulting from falling weights at prescribed height. The behavior of stone masonry bridges impact loading has been analyzed using anon-linear explicit dynamic analysis software (ANSYS). From the results of the numerical and experimental works which have been executed in this study, the following conclusions are made:

1. A non-linear FE simulation based on plastic kinematic material model has been proposed for evaluation of the resistance function of stone masonry arches. And the model has been calibrated based on experimental results.
2. It has been found that the non-linear FE simulation based on said material models is highly dependent on parameters of the masonry joints.
3. Results obtained from numerical results for deflection are less than the experimental results. The variation in deflections of the experimental results and FEA results is about 15.6%.
4. Once a modeling technique is validated, it can be applied to model and analyze similar structures. This will result in saving the experimental cost, time and setup. Further the proposed model can be applied for prototype arch structures.

7.2 Recommendations

Masonry arches are main and unavoidable components of a transport system; moreover, their heritage value is a paramount. Hence to ensure safety of such arches against impact induced actions, the understanding of their behavior under impact loading should be continued by conducting both analytical and experimental researches.

As an extension of this study, the following is recommended for further research work:

1. The parameters identified to simulate the behavior of the prototype Arch can be used to generate a refined numerical model of real bridges for impact assessment.
2. The results of experimental and the refined numerical simulations developed in this study can be used as a basis for formulating simplified analytical models which can be used for fast structural assessment.
3. More refined parametric studies can be studied to define the trends of the capacity curves of stone masonry arches (with either mortar or dry joints) associated with different failure mechanisms. Once the general behavior of capacity curves are identified a simplified resistance curve for SDOF model can be extracted.
4. The simplified capacity curves can be used to generate assessment curves using dynamic analysis of the corresponding SDOF models. These carves can be used for quick assessment of masonry bridges against impact loading.

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APENDIX

ANSYS LS-DYNA command file for 3D modeling of Arch

```
FINISH

/CLEAR

/TITTLE, MODEL OF ARCH

!____________________________________________________

!THIS PROGRAME IS ABOUT GENERATING A MODEL FOR ARCH
!____________________________________________________


/PREP7  !ENTERING INTO THE PREPROCESSOR

R1=0.6    !INNER PARAMETER OF ARCH OR SPAN LENGTH OF ARCH

R2=0.76

J=-R1

I=-R2     !OUTER PARAMETER OF ARCH

HI=0.4    !HIEGHT UP TO INNER SIDE OF THE ARCH

HO=0.56   !HIEGT UPTO OUTER SIDE OF THE ARCH

!----------------------------------------------------

!DEFINNING THE KEY POINTS FOR CURVE FITTING COMMAND

!----------------------------------------------------

    K,1,R1,0,0

    K,2,J,0,0

    K,3,0,HI,0
```

```
K,4,R2,0,0

K,5,I,0,0

K,6,0,HO,0

LARC,1,2,3

LARC,4,5,6


!----------------------------------------

    !GENERATING KEY POINT ON THE OUTER LINE

!-----------------------------------------


RA=0.0769

KL,2,RA,7

KL,2,2*RA,8

KL,2,3*RA,9

KL,2,4*RA,10

KL,2,5*RA,11

KL,2,6*RA,12

KL,2,7*RA,19

KL,2,8*RA,13

KL,2,9*RA,14

KL,2,10*RA,15
```

```
KL,2,11*RA,16

KL,2,12*RA,17

KL,2,13*RA,18


!-------------------------------------

    ! PARTITIONING THE STONES SIZE

!-------------------------------------


LANG,1,7,90

LANG,3,8,90

LANG,5,9,90

LANG,7,10,90

LANG,9,11,90

LANG,11,12,90

LANG,13,13,90

LANG,15,14,90

LANG,17,15,90

LANG,19,16,90

LANG,21,17,90

LANG,13,19,90

!-----------------------------------
```

```
KL,2,0.0102,33

KL,2,1-0.0102,34


!-----------------------------------------


LSTR,34,2

LSTR,5,2

LSTR,1,4

LSTR,1,33

 RA2=.02     !DEFINING SIZE OF MORTER IN JIONTS


!--------------------------------------------

KL,1,RA2,43

KL,3,RA2,44

KL,5,RA2,45

KL,7,RA2,46

KL,9,RA2,47

KL,11,RA2,48

KL,13,RA2,49

KL,15,RA2,50

KL,17,RA2,51
```

KL,19,RA2,52

KL,21,RA2,53

KL,19,RA2,54

KL,21,RA2,55

KL,23,RA2,56

KL,25,RA2,57

LANG,2,43,90

LANG,31,44,90

LANG,33,45,90

LANG,35,46,90

LANG,37,47,90

LANG,39,48,90

LANG,41,49,90

LANG,43,57,90

LANG,45,51,90

LANG,47,52,90

LANG,49,53,90

LANG,51,56,90

LANG,45,50,90

```
!-----------------------------------------

! GENNERATING THE AREAS OF THE STONES

!-----------------------------------------


A,20,7,4,1

A,21,8,35,44

A,22,9,36,45

A,23,10,37,46

A,24,11,38,47

A,25,12,39,48

A,31,19,40,49

A,26,13,41,57

A,14,61,50,27

A,28,15,42,51

A,29,16,58,52

A,30,17,59,53

A,2,5,60,56


A,20,44,35,7

A,21,45,36,8

A,46,37,9,22
```

A,47,38,10,23

A,48,39,11,24

A,49,40,12,25

A,57,41,19,31

A,50,61,13,26

A,51,42,14,27

A,52,58,16,29

A,53,59,16,29

A,56,60,17,30

A,52,58,15,28