3D Elasto-plastic Seismic Numerical Modeling and Analysis of Masonry Arch Bridges

by

Muhammad Faisal Javed

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A thesis submitted in partial

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DEDICATED To

MY

PARENTS

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ABSTRACT

Pakistan is located on an earthquake prone area and old bridges were designed without any seismic provision. In the years (2005), tremors were felt in Pakistan due to the strong earthquakes at Kashmir, which highlight the earthquake threat to Pakistan.

This study focuses on seismic vulnerability of arch type masonry bridge structures in Pakistan, designed primarily for gravity loads, when they are subjected to earthquakes. A case study has been carried out for the vulnerability study for an 11 m span masonry arch bridge representing typical bridge structure in Pakistan. In the case study, nonlinear dynamic Analysis for the full scale structures are carried out. Three states limits PGA of Collapse (CO), Severe Damage (DS), Limited Damage (DL) along with their relationship with the acceleration, defining its three levels is the main objects to be achieved. The evaluation of the seismic vulnerability is carried out by verification of arch at mid span, piles, shoulders, and near support of the bridge. The capacity vs demand curves are obtained for elements of bridge based on the accelerograms due to the worst earthquake scenario in Pakistan. From these studies, it is concluded that the arch type masonry bridges in Pakistan may suffer some damage due to the worst possible earthquake.

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Chapter 1

Introduction

1.1 Introduction

Masonry construction signifies the modernization of cave dwellers symbolizing/leading towards the civilized life. In the beginning epoch, man exposed a mode of spending relatively less besieged life of cave by assembling the stones in confined manner. Stones proved its strength and effectiveness for habitation. It eventually boosts up the emergence to the construction of houses and buildings. Basically stones solely were utilized for houses regardless of any jointing material. Aran Islands, Ireland have rich ruins of stone huts with dug grounds. Elaborated stone techniques were then put forward by Egyptians in the form ancient structures, the pyramids.

Tigris and Euphrates lacked stone outcroppings; clay brick filled the position of masonry structures as western Asia was rich in clay deposits. More specifically sun-dried bricks faced with kiln-burned units were consumed in the construction of Assyrian and Persian empires. Pre mature failure construction was overcome by implementing different construction typologies. Stone slabs were used in Egyptian temples with the supporting close columns. Greeks used thin stones to cover wooden roof beams and Romans built huge arched bridges in large number. Placing of stones figuring out the arches, as a solution to its weight turn out to be a breakthrough in the account of masonry (Ajoy kumar, 2008). Structures are sturdily enhanced by the utilization of natural stones like granite, limestone or sandstone (Elisabeth Scheibmeir , 2012). Figures 1.1 and 1.2 shows some famous and typical mixed masonry arch bridges.

Figure 1.1: Shahrestan Bridge (twelfth century) on the Zayande River: Mixed masonry (stone blocks and bricks), 100m span and 11 arches

Figure 1.2: Alcantara bridge (2nd century) on Tajo river, mixed masonry, 194m long, 6 spans

Modern building industry has replaced the craftsmanship skills required for Cutting, centering, bedding by the time saving resources. The use of masonry arch is transformed to decorative elements (Boothby & Anderson Jr., 1995) but the masonry arch was been widely used in still existing bridges. Europe has approximately 200,000 individual structures comprised in the railway network and bridges. (Brencich & Morbiducci, 2007).Almost all of these structures have spans less than 200ft and consists of natural stones with lime or cement based mortar joints._Introduction of truss, scientific structural analysis and development of high-tensile resistant materials in the era of 16th, 17th and 19th century respectively declined the status of masonry as a material. They were overcome by the concrete in 20th century due having more flexibility which was indirectly affecting the study and standpoint of masonry.

Major apprehension includes Safety and collapse evading of the essential elements of modern infrastructure system. The significance of assessing the seismic exposure of existing structures is proved by the recent events in the Pakistan. Italian building code from 2008, NTC08 (CS.LL.PP., 2008),is the latest one defined for the first time specific criteria for existing masonry structures. It's being concerned with the degree of ambiguity ensuing from a global assessment of the structure (geometry, constructive details, material properties) and has been a considerable enhancement with reference to the criteria of safety/performance evaluation for masonry structures, taking into account local instrument and the non-feasibility of together with masonry to the abstract framework "ductile mechanism/brittle mechanism" functional for materials like reinforced concrete or steel (Marcari, 2012).

In the history of Indo-Pak, Earthquake of 8th October, 2005 is considered as the dead list earthquake. 3.3 to 3.5 million people were affected and 2.5 million were displaced from their origins.80, 000 left injured while deaths were exceeded from 70,000.Most of the buildings constructed with unreinforced stone and brick masonry were damaged (Muhammad Javed, 2009).

1.2 Importance of structural analysis of masonry

To come across the epoch of initiation of masonry construction in our ancestors is the exigent charge to cover up as it illustrates the mankind to step into civilization. Throughout the centuries it’s being categorized as the art of construction. Subsequent to overcoming its downsides (e.g. utilizing the masonry compressive stress) till nineteenth century it gets to the peak of its propensity. Though concrete and steel has dominated the respective position but still the masonry constructions possesses architectural beauties. Tourism being augmented in the East Asia, northern Africa, associated to these historical edifices became a mounting industry in Europe from the second half of the prior century. In certain countries the gross national income is based on tourism. Irrespective to ancient architectural heritage (for example, see Figure 1.3) or modern heritage (for example, see Figure 1.4).

Figure 1.3: Colosseum (80 A.D) Rome, Italy

Figure 1.4: Agra Taj Mahal (1650 A.D), Agra, India

Analytical study of construction material has recently come to attention of researchers after the significant damages to the heritage construction in the last decades as no specified rules were tag along for construction in ancient times. Till now the progress in the subjected field is not satisfactory indicative of the motives of less awareness and insufficient funds reserved for the heritage structure in common with less expertise in the specified field. The study of structural engineering is more oriented to modern materials; from macro-level to micro-level of study of masonry behavior requires a devoted insight focus in the behavior. (Ajoy Kumar, 2009).

An Unreinforced masonry building (or UMB, URM building) is a type of building where load bearing walls, non-load bearing walls or other structures, such as chimneys are made of brick, cinderblock, tiles, adobe or other masonry material, that is not braced by reinforcing beams. The term is used in Earthquake engineering as a classification of certain structures for earthquake safety purposes, and is subject to minor variation from place to place.

URM structures are vulnerable to collapse in an earthquake. One problem is that most mortar used to hold bricks together are not strong enough. Additionally, masonry elements may "peel" from the building, and fall onto occupants or passersby outside. In California, constructions of new unreinforced masonry buildings were prohibited in 1933, and state law (enacted in 1986) required seismic retro-fitting of existing structures. Retrofits are relatively expensive, and may include the building being tied to its foundation, tying building elements (such as roof and walls) to each other, so that the building moves as a single unit, rather than creating internal shears during an earthquake, attaching walls more securely to underlying supports, so that they do not buckle and collapse, bracing or removing parapets and other unsecured decorative elements. Retrofits are generally intended to prevent injury and death to people, but not to protect the building itself (Broaderick Perkins,2004). According to the 2006-04 CA seismic safety commission report, there are still 7800 URM buildings with no retrofitting in CA. 1100 in the city of Los Angeles.

The California law left implementation and standards, up to local jurisdictions. Compliance took many years. As of 2008, most (but not all) of the unreinforced masonry buildings have undergone retrofitting (Selena Robert, 2008). There is particular cause for concern in regions which can generate strong earthquakes, but only rarely. Such regions may not have regulations limiting the construction of UMBs, or have only implemented them recently. Public awareness of earthquake safety may be low. For example, the Wasatch Fault in the U.S. state of Utah closely parallels the state's most populous metropolitan area, the Wasatch Front (which includes the state capital Salt Lake City). The Wasatch Front has a population of 2 million, and contains 200,000 UMBs compared with the entire state of California's 25,000 (Desert News Article, 2013).Utah has recently retrofitted many public UMBs to better withstand earthquakes, but most UMBs in the state are private homes.

The lack of earthquake codes preventing the construction of UMBs was a major factor in the high death toll in the 2010 Haiti earthquake.

1.3 Objective

As haven't got significant damage evidences of recent earthquakes, not perceiving the potentially high seismic vulnerability of masonry bridges. Particularly the assessment of interaction problem between infill material and the side walls of the bridges under seismic excitation is not been achieved yet but seems to be a possibly relevant damage mode. Irrespective of the mentioned issues the interaction between soil, sub and superstructure, in relation with possible out of plane collapse. Before any structure damage from any other mechanism the failure mode is probable to transpire at the very commencement of the seismic excitation. The out-of-plane collapse occurrence is being accepted even for low level acceleration. With the parametric study on a bridge and soil typologies failure mechanism is being occurred in the current work. Acceleration demand and capacity are compared in each case for certain conclusions.

The aims to be achieved with the respective research are the seismic evaluation of historical bridges structure. Multi-span bridge structures with arch type geometry are analyzed.

Chapter 2 is Devoted to masonry bridges has detailed description concerning the construction and various structural parts of the masonry. Also it will present a very basic conception on numerical modeling of masonry structures applying finite elements approach (both for macro- & micro-modeling approach). This will contain the geometrical and structural idealization of masonry constructions. A brief summary of different kind of nonlinear material modeling will discuss the specificity of the models for a particular construction material.

Chapter 3 is related with Seismic history of Pakistan and Peshawar region and seismic codes development is discussed.

Chapter 4 is related with the finite element modeling, analysis parameters and response spectra of the soil of the bridge are discussed in detail.

Chapter 5 is devoted to the working out of the level of demand on the bridge kept under consideration. Acceleration demand along with the elastic response spectrum is utilized for calculating this demand. AASHTO LRFD 2012 is based for the contraction of this spectrum and the acceleration consequent to the dominant frequency of oscillation of the bridge in the transverse direction is comprehended from the spectrum. Assumption of bridge to be in relation with the seismic zone 2 as indicated in Code and located on the soft Soil category is followed in determining the acceleration at the base of the bridge cases. Next to the acceleration response factor, acceleration demand at the center of the pier is resolute. Seismic demand of the structure is also concerned with the verification of arch of the bridge at the central point of span.

Chapter 6 covers the results obtained from the comparison of the acceleration capacity and demands for the different cases are summarized. To test out bridge failure or endurance and quantifying its displacement from the limit condition, consequent to demand equal to capacity, the comparison between demand and capacity is examined. In conclusion, the most pertinent values derived from this exertion and delineated implications for future research, with particular emphasis issues where further investigation is required are demonstrated.

Chapter 2

Masonry Arch Bridges and Seismic Assessment

2.1 General

Arches are considered as the key structural elements of the Masonry Bridge, usually comprised of two or more arches depending on the width of the river to be crossed. Cornices are used to cover the vaults from outside while abutments for the rest. Granular materials from mines or river underneath are used as the filling material in the bridge for providing horizontal decking. Hollow concrete spheres, being considered as sophisticated material are used in domes and vaults for increasing stiffness with fewer loads. Parapet walls are topped at the spandrel walls within which filling material is embedded. Figure 2.1 and 2.2 shows typical sections of masonry arch bridges.

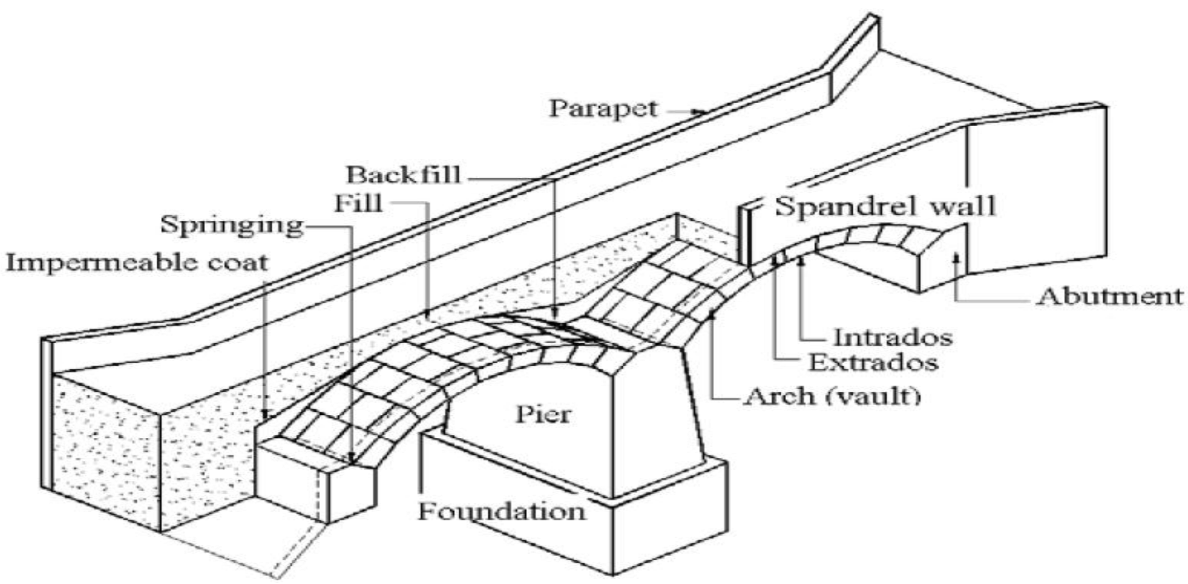

Figure 2.1: Axonometric section plane of a typical masonry bridge (Galasco, et al.2004)

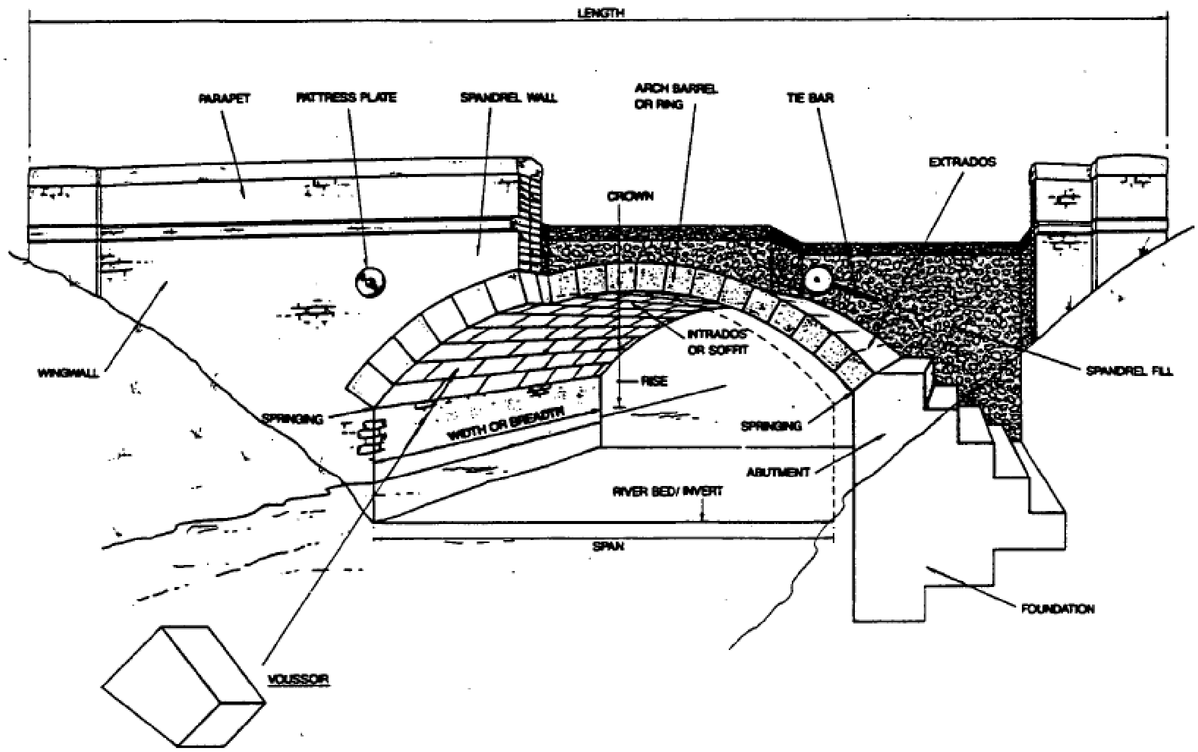

Figure 2.2: Identification of the different parts constituting a masonry arch bridge [Galasco et al. 2004]

2.2 Structural arrangement

For existing masonry bridges, one of the major difficulties consists in the determination of the characteristics of the material used in the construction. This is mainly due to intrinsic characteristics of the masonry, which is very anisotropic material, whose mechanical properties are strongly dependent on the properties of its constituents. In particular, for existing structures it is far from trivial to determine the consistency of the grout, when present or the quality of the structural elements (brick or stones) used. Often, different types of materials may be used for different parts of the bridge, due to structural reasons (need for higher resistance of the more heavily stressed parts and for lower weight of the non-structural parts), as well as to economic reasons.

The geometry of the bridge is strongly influenced by topography of the valley to be crossed. Wide and deep valleys are often crossed by bridges called viaducts, with more spans on high piers, whilst wide but shallow valleys are crossed by bridges with more spans on short piers. Narrow valleys and minor streams are usually crossed by single-span-bridges.

2.3 Characteristics of the filling material

The space between the two walls of a masonry arch bridges is filled up with some material, in order to create a horizontal plane. This infill material must be light and able to drain the water; moreover it should contribute to the repartition to the arch of the concentrated loads applied on the horizontal plane on top of it.

To reduce the thrust on walls the filling material comprised of incoherent material (such as soil or mucking resulting from mines excavation) can be altered by dry stones, coarse aggregate, gravel, ballast or, more recently, low resistance concrete. In spite of filling material, utilization of some brick vaults is usual in case of viaducts (figure 2.3). The reason although doubtful but still comes with the aim to reduce the weight acting on the arch. The infill material will always be referred to as soil in this write up, for the sake of simplicity.

Table 2.1 summarizes some classic values of the precise weight of the infill material, suggested by Gambarotta et al. It’s being mentioned that in case of insufficient information regarding material values between 17 and 19 kN/m^3 may be practically presumed for the specific weight as 17 kN/m^3 has been selected.

The height of soil within horizontal plane and top of arch must not be less than 40 cm as defined by Albenga [1953], however the respective value can be let down in case of small bridges up to 30cm but not less than 15 cm. Thickness of the stratum is proportionate to arch thickness at the apex stone. Certain pressure is produced on the sides of walls by the filling material that can be segmented into static (always present) and dynamic (developed only when the structure is subjected to dynamic loads) parts.

Figure 2.3: Example of Viaduct

Table 2.1: Values of specific weight for some types of filling materials

Material	Specific Weight (KN/m^3)
Incoherent material	16-18
Dry stones	18-21
Aggregate or gravel	14-18
Low resistance concrete	21

The stimulus of outwardly non-structural elements like filling and spandrel wall on the load-bearing capacity has long been underrated. Outmoded analysis approaches of static or kinematic limit analysis are pertained vault, abutment and piers, leading to conformist results. In case of vertical loading, the filling material always upsurges the failure load, distributing concerted loads from the surface of the bridge and aggregating stability by acquaint with initial compression to the arch (Ford, et al., 2003). The influence of filling material and spandrel walls might accepts different affect from seismic actions. The arched structure is modeled in 2D for vertical loading; the hypothesis of 2D behavior is not appropriate anymore for seismic actions, leading to load-redistributions in a 3D way (Galasco, et al., 2004). Typical failure appliance for masonry arch bridges has been shown (Rota, 2004), that under seismic action consisting of the upending of the spandrel walls, which are pushed out-of-plane by the mass of the filling.

Experimental conclusions in comparison with the numerical results related to number of vaults (Brencich & Sabia, 2008) and height of the arches rise (Brencich, et al., s.d.), have crucial value for the load-bearing capacity. Mode shapes and the damping values (Brencich & Sabia, 2008) are highly affected by the number of vaults that is the reason for avoiding self repeating model for simplifying the multi-span bridges. On the subject of the height of the rise, height of the arch have indirect relationship with the number of vaults have on the load-bearing capacity. But it’s not proved positive for seismic action.

Basically the strength of the strength of arches and vaults depends on the geometrical structure, supporting the conditions indeed material properties have minor influence as documented in Roca and Orduña (Roca &Orduña, 2012). Before the scientific revolution, ancient empirical criteria were based on mere geometric approaches, which are still valid nowadays irrespective of non-scientific base. Far along the first rational approaches led to graphic statics devising the perception of thrust lines (Figure 2.4 and 2.5), that are still applied in modern limit analysis within the concept of lower bound theorem.

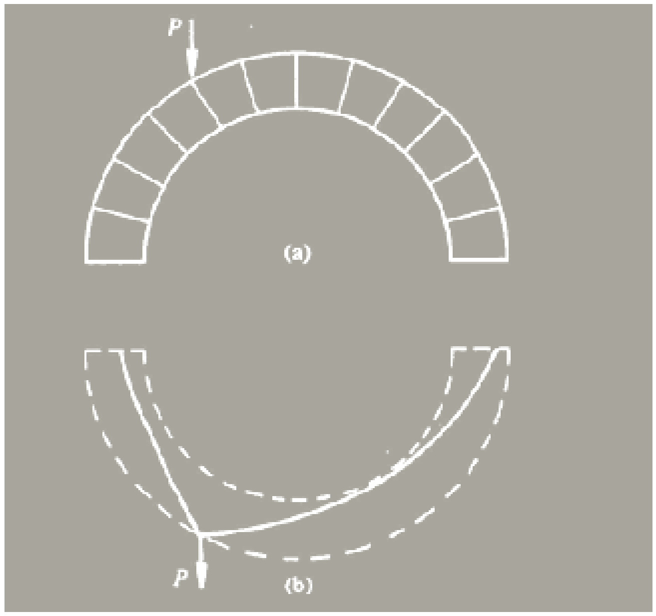

Figure 2.4: Concept of Thrust lines (Roca & Orduña, 2012) a) load Condition b) Inverted catenary model

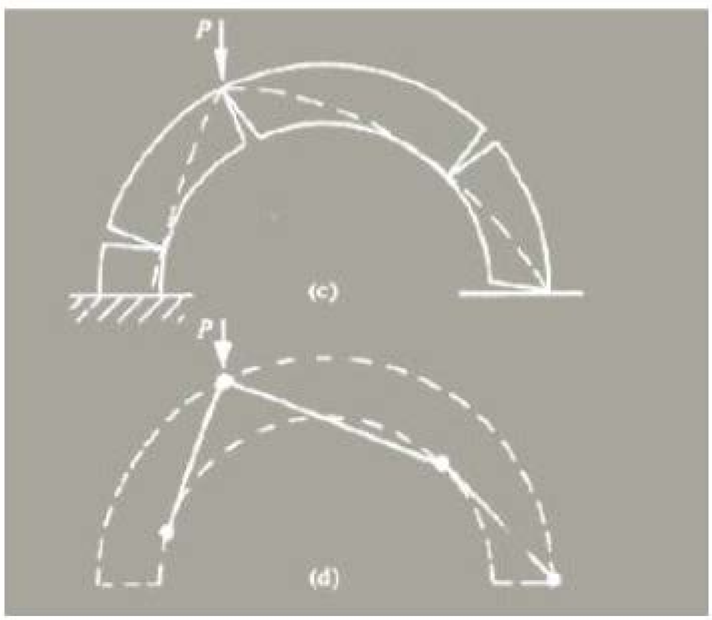

Figure 2.5: Concept of thrust lines (Roca & Orduna,2012) c) Thrust line and corresponding hinges d) Kinematics of resulting mechanism

As formulated by Hayman (Roca & Orduña, 2012), (Gilbert, 2007), Geometric analysis is still considered valid for the structural analysis of masonry arches. In light of Hymen hypothesis there is null tensile strength, infinite compression strength and no sliding between the stone blocks. Last decades can be initiated for the formulation of numerical methods and programs, responsible for accurate analysis of feasible structures by applying Discrete Element method (DEM), Discontinuous Micro-Modeling (FEM), Continuous Macro-Modeling (FEM) or Macro blocks. Material science has faced detailed material parameters in literature and deliberate evolution (Lourenço, 2001) specifically in historic material. Roca et al (Roca, et al., 2010) has explained progressive approach deep understanding in the complexity of historic masonry by researchers work.

Due to the composite nature, historic masonry posses complex mechanical phenomena, as being comprised of a unit (brick, block or stone) with a mortar, brittle behavior in tension, high compressive strength and preside over by friction in shear. In addition, historic masonry is well thought-out as an anisotropic material for demanding high number of parameters, specifically in post-yielding which are difficult to be obtained.

2.4 Seismic damage to masonry bridges

Period of 1830 – 1930 is considered as the constructive era for masonry construction in Asia. The former archives retain insufficient seismic damages to these structures as their seismic history is quiet short. It gives the impression likely that, a masonry bridge without having specified mechanical deficiencies can bear earthquakes of temperate intensity, without being heavily damaged. However, the retort of these bridges to major earthquakes requires detailed investigations.

Throughout the world awareness about seismic damage is limited and of no worth. Even in under developed areas of the world, obtaining information regarding the subject is not approachable as masonry structures are of limited extension irrespective of the industrial zones where the modern construction is not considered as the masonry. The seismic risks are concerned with the functionality of particular parts of the system of infrastructures rather than association with human lives loss. In addition the reason of the scare evidences available, specifically for the extents where an earthquake causes many victims and consequently this kind of infrastructural damage is not measured as significant even though infrastructure techniques vary from place to place in light of native knowledge and availability of resources. Detailed study of evidences of seismic damage to masonry bridges throughout the world may leads to the exploration of the field.

Figure 2.6: (a) Severe damage to the stone masonry base of tower in a suspension bridge (b) Collapse of stone masonry retaining wall that resulted in failure of approach road.

A number of masonry bridges were affected in Northern Pakistan during earth quake in Kashmir in October, 2005 (Magnitude 8.5) and majority were the short span bridges with short piers. Cracking and skews of the elements of the arches, damages to the piers and overturning of the bridge walls were the prevailing damages chronicled in that pace. Figure 2.6, 2.7 and 2.8 shows the destructive seismic masonry during Kashmir earthquake.

Figure 2.7: (a) Collapse of a suspension bridge over River Neelum due to landslides

Figure 2.7: (b) Slippage of side-sway cable anchor block & failure of approach road to bridge.[S.M.Ali, 2009]

Figure 2.8: Overturning of the walls for a transport masonry bridge, after the Bhuj earthquake, 2001, India [Gisdevelopment, 2001]

Image of an 88 years old Masonry Bridge is shows another example occurred during the Bhuj earthquake signifying overturning of the walls and the falling-off of the infill material, exposing the train rails. Destruction due to overturning of bridge walls is also dominant in the earthquake occurred in the Italian regions of Umbria and Marche, in 1997.Figure shows the damage arch masonry exposing the overturning of filling material of the wall although the type of material cannot be predicted through the image. The evidences proves that the overturning of parapets and spandrel walls frequently causes damage and the failure due to interaction with infill material is really being considered as common. The study of this type of failure mechanism will be focused in the rest of the work.

Figure 2.9: Damage of a masonry bridge, after the Umbria-Marche earthquake, 1997 [Resemini and Lagomarsino, 2004]

Depending on the accuracy required, different strategies can be implemented for masonry modeling. The strategies followed in the figure comprised of detailed micro-modeling units, mortar symbolized by continuum elements while unit-mortar interface by discontinuous elements. Units, mortar and interface are modeled as a single continuum in macro-model approach. In understanding local behavior and structural particulars, micro-modeling is dominant.

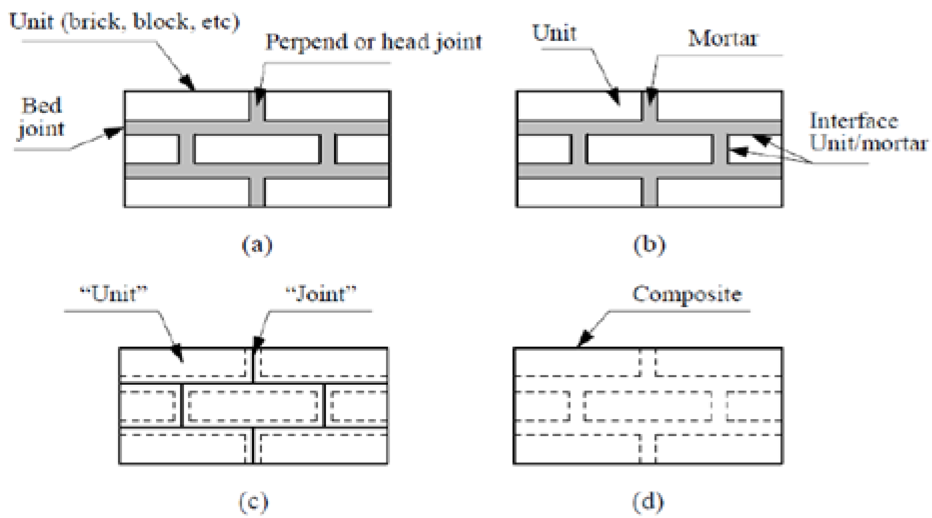

Figure 2.10: Modeling strategies for masonry structures according to Lourenco (Lourenco, 1996) a) Masonry Sample b) Detailed micro modeling c) Simplified micro modeling d) Macro modeling

The parameters from the individual components or a sufficiently large masonry specimen cancellation are utilized for obtaining parameters for macro-modeling analysis.

$$f_k = K \times f_b^a$$

Where

f_k = (characteristic strength of masonry)

f_b = (brick strength)

fm = (mortar strength)

K, α and β are constants depending on the brick and mortar material and configuration. RILEM given standard description about compressive strength of masonry analogous to the bed joints, which not only consist of masonry specimen having length of minimum 2 units and a height of minimum 5 units (Pech & Kolbitsch, 2005). Properties of masonry is dependable on factors i.e. material properties of the units and mortar, arrangement of bed and head joints, anisotropy of units, dimension of units, joint width, quality of workmanship, degree of curing, environment and age. (Lourenço, 1996).

The availability of experimental material results accurate structural analysis for the strategy of any type i.e., micro-,meso- or macro-level.3D FEM can incorporate non-linear constitutive models for masonry but lack of experimental data directly affects its reliability. Practical approach in general pertain simplified isotropic constitutive laws, defining elastic modulus, compressive (and tensile) strength and Poisson´s ratio. Plastic regime, having Post-peak behavior is often neglected but in case of safety of historic masonry structure comprised of non-linear properties, the simplifications are kept in priority. More specifically within seismic analysis, post-peak regime is kept in account for the prevention of life dent or collapse hazards.

Regarding seismic safety evaluation, current Seismic codes like the Euro-code 8 or the AASHTO 2012 suggest a variety of different analysis methods and incorporate rules for existing and new structures together with specific regulation for different building materials, including masonry. AASHTO2012 defines specific criteria for masonry entailing specific confidence factors which take into account the level of knowledge concerning the structure by means of geometric parameters, morphology and material properties. Depending on the structural characteristics of the building and/or on the trend of uncertainty within the results admitted, seismic codes recommend to use either lateral force methods, modal response spectrum, nonlinear static (or pushover) or nonlinear dynamic analysis.

The code furthermore specifies other methods like linear and nonlinear kinematic analysis for existing masonry structures (Marcari, 2012).

Variety of analysis methods and rules are proposed for the under developing structures combination with the building material and masonry, by the Seismic codes including Euro code, AASHTO and Italian NTC 08.

In case of properly planned buildings, depending on the mass and rigidity, Static lateral force methods can be utilized. The mentioned method may produce considerable seismic behavior, but still adverse results may be expected as the method cannot be considered suitable for historic masonry structures (Máca & Oliveira, 2012). Predefined lateral load-pattern with the height of the structure is put under consideration in Nonlinear Static Analysis (NSA). Till the achievement of target displacement, the load is monotonically augmented by multiplication with an incremental escalating load factor or until collapse of the formation. NSA is the mainly admired technique for performance-based design implemented in different design levels (Kalkan & Kunnath, 2007), (Magliulo, et al., 2007), (Mwafy & Elnashai, 2001), (Krawinkler & Seneviratna, 1998), (Chopra & Goel, 2002), (Chopra, et al., s.d.) and is also strongly suggested in most of seismic codes - EC8 (UNI ENV 1998-1 (Eurocode8), December 2004), FEMA 440 (FEMA 440, 2005).Choice of load patterns is indispensable for the accuracy of this approach because higher modes are not put in account. Adaptive pushover analysis is utilized for achieving improvements higher modes.

Seismic action is normally represented by a response spectrum in Modal Analysis or Response-Spectrum Analysis, serving to excite the structure to be analyzed. Low computational effort and input data is requisite for the said approach.

Chapter 3

Bridge Engineering and Seismic Hazard in Pakistan

3.1 General

It is pertinent to be mentioned that The Seismic Hazard Map used until 2007 prepared by the Geological Survey of Pakistan (GSP) and the Seismic Zoning are laid on Geophysical Center Quetta's instrumental Macro-Earthquake data of 1905 to 1979 and on Modified Mercalli Intensity (MMI) Scale (BCP, 1986)respectively. Providing quantitative measure of the true ground accelerations is not possible for MMI scale it may yield only crude estimates of the ground motion intensities as it is based on the visual observations of damage at a meticulous location recounting to the type of construction at that site. In contrast the AASHTO Standard Specification formulates a multifaceted scenario of design and construction, and pretense a complex challenge for the engineers irrespective of the local conditions, including material properties and workmanship.

The seismic hazard map may require, requiring a re-evaluation of existing bridges with the increase of the regional earthquakes database record availability. Japan indicates decreasing trend of bridge failures along mature designing experience by giving serious attention towards the seismic hazards just after the 1923 Great Kanto Earthquake of M7.9 disintegrate of only 15 bridges due to earthquakes in the pace of 71-year period until 1995 illustrated their accomplishment but the destruction of 25 bridges in Hanshin/Awaji Earthquake at Kobe alone in 1995 compelled them to revise the Japanese bridge seismic code. A comprehensive performance of in scripting a seismic code came into practice after overwhelming earthquake of October 8, 2005 in Pakistan, resulting publishing of Building code of Pakistan (BCP, 2007). Figure 2.7 shows the seismic map of Pakistan for 2007 and onwards. The Design Basis Ground

Motion with a 10% probability of exceedance in 50 years is addressed in the 2nd chapter of this code. Lying of Pakistan in Zone 2A or above in relation to a Peak Ground Acceleration (PGA) in the range of 0.08-0.16 g or higher is obvious in Seismic Hazard Map of BCP (BCP, 2007).

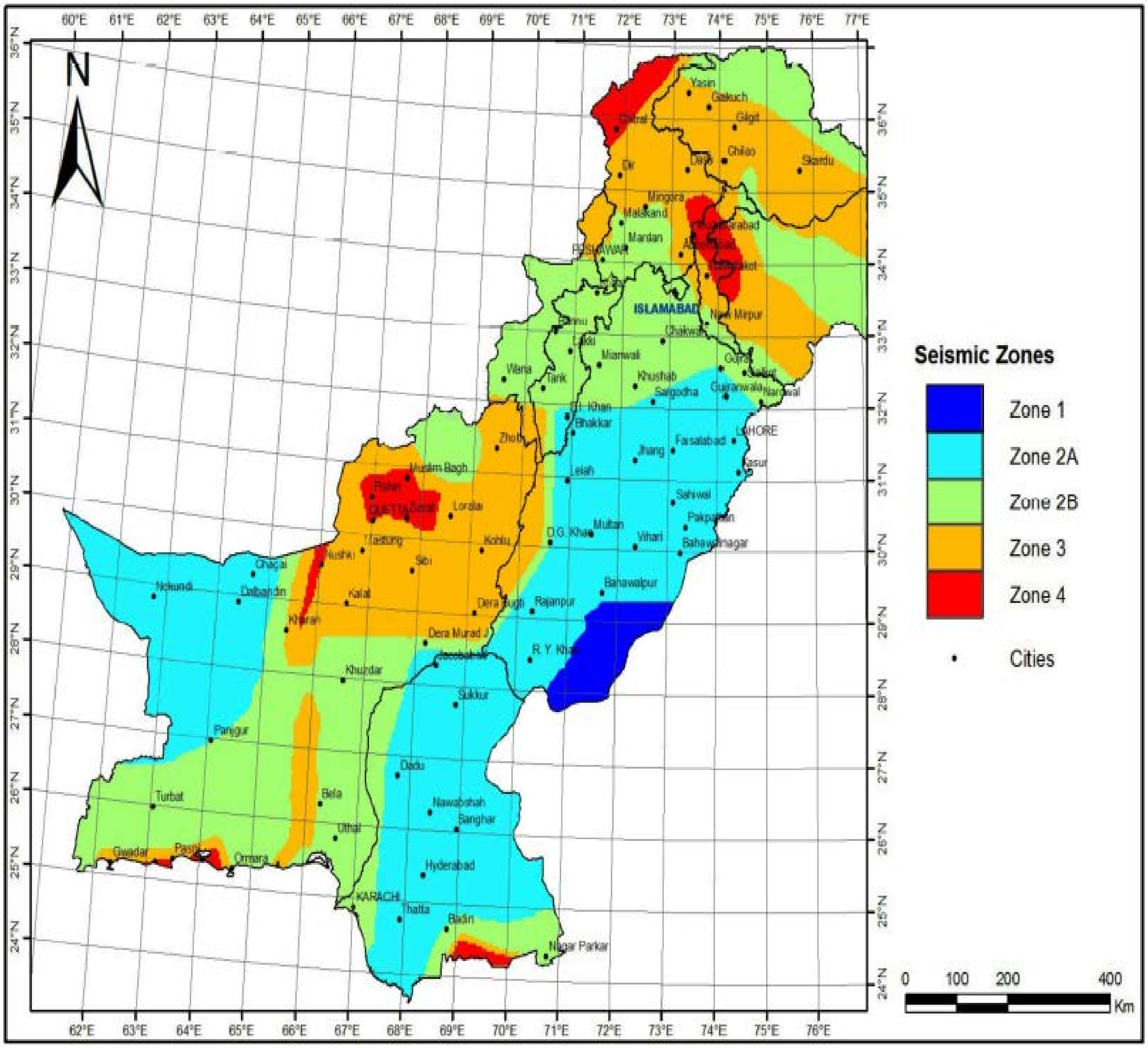

Figure 3.1: Seismic zoning map of Pakistan (BCP, 2007)

As devised in 1931, previous Pakistani seismic zoning was based on MMI scale, according to which Pakistan is divided into four zones, i.e., Zone 0, Zone 1, Zone 2 and Zone 3 depending upon the increasing ground shaking but the quantifiable measures of ground shaking intensity is not demonstrated. Zone 2 is considered as the restrained damage area analogous to an intensity of VII of the MMI scale comprising northern areas of Pakistan. In account of increasing ground acceleration, the recent publication of Seismic Hazard Map in BCP (BCP, 2007) has five Seismic Zones, namely, Zone 1, Zone 2A, Zone 2B, Zone 3 and Zone 4, illustrating that the northern part of Pakistan are placed in the zone of highest seismic hazard, i.e., Zone 4 with the a PGA ≥ 0.32g.Rest of the area composes Zone 3 with the PGA ranging from 0.24g to 0.32g. All new structures designed to withstand higher seismic loading according to the new Seismic Hazard Map is the basic requirement of revised seismic zoning map. Zone 2 is considered as the restrained damage area analogous to an intensity of VII of the MMI scale comprising northern areas of Pakistan. The bridges ought to be designed for a literal force of 2% to 6% of the dead load of the structure as recommended in Code of Practice for Highway Bridges (CPHB, 1967) Code of Practice for Highway Bridges (CPHB, 1967).The subjected range correlates to foundation resting of various forms on different soil ,taking in account of seismic zoning information. Compatibility of 1967 code compatible with the recent Seismic Hazard Map and the associated specifications is preferred to be kept in view.

Table 3.1: Seismic Zones

Seismic Zone	Peak Horizontal Ground Acceleration(g)
1	0.05 to 0.08
2A	0.08 to 0.16
2B	0.16 to 0.24
3	0.32
4	>0.32

Where “g” is the acceleration due to gravity.

The acceleration values are for rock site condition with shear wave velocity (Vs) of 760m/sec (soil profile type S_B)

AASHTO Standard Specification referring to the Seismic Hazard Map of USA is mainly followed in designing the bridges in Pakistan. It proves the utilization of arbitrary PGA values or 2%-6% of the weight as the lateral force value in the prior designs of bridges till 1979. The Seismic Hazard Map based on the MMI scale without quantifiable PGA values are employed in the bridges up to 2007. Designers also make use of arbitrary PGA values. The Seismic Hazard Map published in BCP (BCP, 2007) with applicable PGA values needs to be utilized in the evaluation and retrofitting of on hand bridges. AASHTO Standard of 1980's, simplistic elastic design procedures, that is, the Equivalent Static Force Procedure and the Response Spectrum Method were allowed in the older versions. The current AASHTO-LRFD method is comprised of various analysis procedures ranging from linear elastic, to nonlinear inelastic. The choice of an analysis practice is joined in a coherent manner depending on the importance of a bridge and its location in a particular seismic zone while keeping in view the regularity or irregularity of the bridge.

Evaluation of existing bridges is significant for safety measures against the future earthquakes. Auxiliary developmental reforms on indigenous bridge design code comprised of major revision of the 1967 Bridge Code heeds the attention of the researchers for further progress. In Pakistan ,huge portion of the cities as Karachi, Quetta, Gawadar, Peshawar, Abbottabad, Gujrat, and Islamabad falls close to the seismic zone as shown in the Seismic Hazard Map of BCP (BCP, 2007).

3.2 REVIEW OF SEISMIC ZONING FOR PESHAWAR REGION

3.2.1 General

UBC code is implemented in structural engineering of Pakistan. According to the UBC code1997Peshawar in the most severe earthquake zone-4 alongside the higher earthquake prone areas like California, Alaska, Japan, Chile, Taiwan or Turkey, being suffered from the destructive disinters in their past. Earthquake history of Peshawar, however, doesn’t indicate of even moderate desolation. Various national and international agencies illustrate conflicting view of seismic zonal conditions of Peshawar.

The review of most of the published work concerning seismic zoning of Peshawar is given in this chapter.

3.2.2 Uniform Building Code97 (UBC1997)

Till 2004 Pakistan doesn’t possessed any seismic regulation for buildings directly denying the lateral strength requirements from local practice.UBC code being a well-respected code throughout the world is referred to the Structural engineers in Pakistan as well. Peshawar, Karachi and Islamabad are in zone 4, keep up a correspondence to Peak Ground Acceleration (PGA) of above 4.0 m/sec^2 or 0.4g (UBC code., 1997).Seismic risk zoning of Karachi by UBC has already been strongly objected (Loya, A. R., Zaigham, N. A., and Dawood, M. H., 2000) by the seismic committees of Associated Consulting Engineers Pakistan and Karachi Building Code Authority, through discussions concluding that conclude that Karachi should be placed in zonc-2b instead of zone-4.

3.2.3 United States Department of State Office of Foreign Building Operations

Woodward and Clyde in 1985 prepared the report which later on, in 1992, encompassed the Peshawar in zone 3 in seismic hazard zones of international cities [US Department of States Manual., 1997].

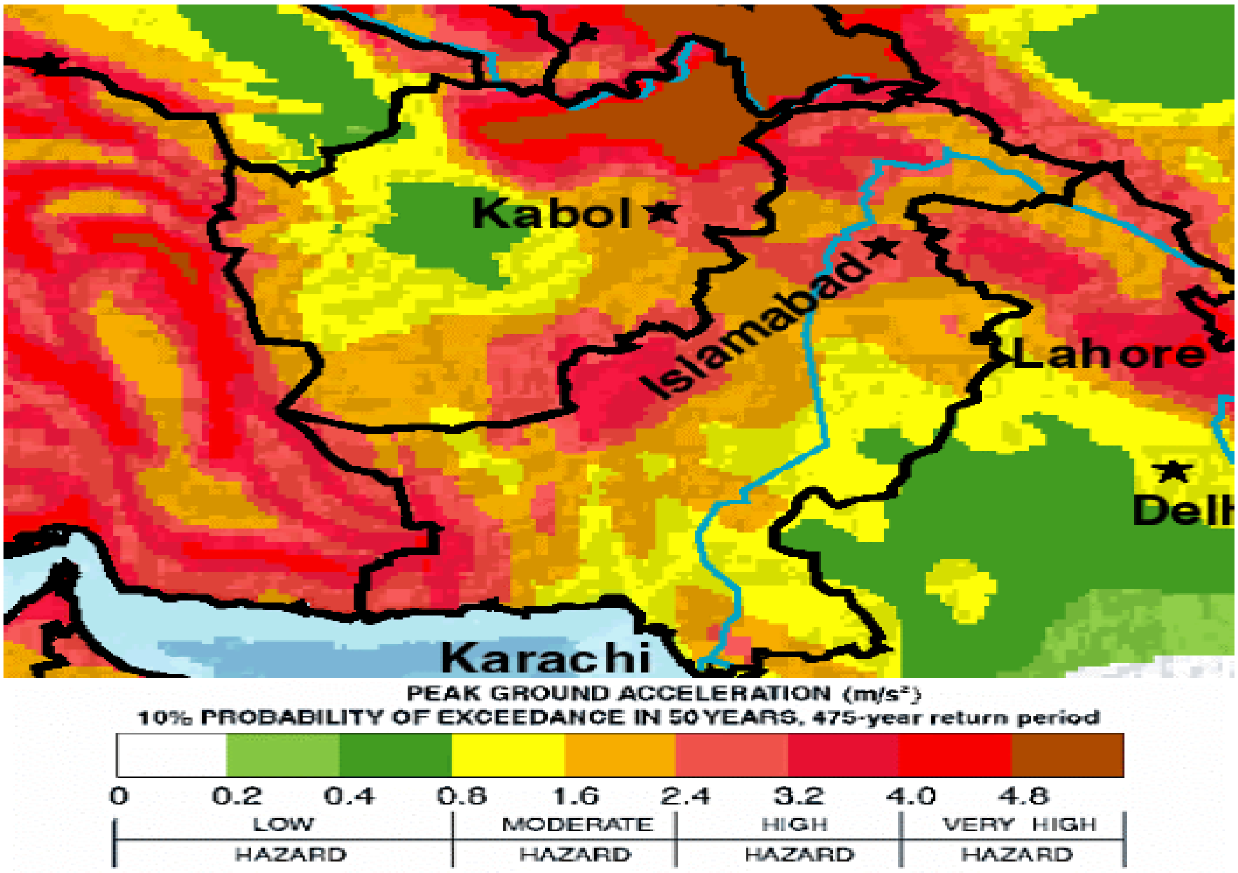

Figure 3.2: UN GSHAP map for Pakistan and Afghanistan

3.2.4 United Nations Global Seismic Hazard Assessment Program UNGSHAP

"Global Seismic Hazard Assessment Program" was launched in United Nations in 1992 to have power over the earthquake disasters of the human masses. Number of geologists and seismologists contributed throughout the world. East Asia together with Indo-Pak Subcontinent was particularly put under consideration. GSHAP in December 1999 [GSHAP IUSGS., 1992 to 1999] published detailed report and seismic risk zonal distribution criteria. UNGSHAP map for PGA values in m/sec2 with 10% probability of exceeded in 50 years analogous to return period of 475 years is shown in figure. According to map, Peshawar in “Moderate Hazard Zone” consequent to PGA values of 1.6 to 2.4m/ sec^2 or 0.16g to 0. 24 g.

Hindu Kush region is positioned in the dangerous hazard zone in relation to PGA values of above 4.0 m/sec^2.Per year average of four magnitudes 5.0 is documented in this region. Towns in the regions of Badakhshan and Takhar provinces of Afghanistan are destructed in the earlier period justifying its placement in the high hazard seismic zone. The map is shown in figure 3.2.

3.2.5 Geological Survey of Pakistan: Seismic risk map of Northern Pakistan

The National Geo-Data Center issued Geological Survey of Pakistan in 1988 [Mirza, M. A. et al., 1988] prepared by Mohammad Ali Mirza, et al. The map is deficient in the information regarding peak ground acceleration as requisite by the structural engineers. According to Mercalli scale of earthquake intensity is determined through the maps, illustrating the Peshawar in major damage zone and District Dir, of high seismic region, in a moderate damage zone bring into being qualms in the authenticity of the map.

Maps prepared by GSP cannot be suggested for earthquake measures by engineers as GSP don’t have its own strong motion recording system.

3.2.6 Geophysical Center Quetta, Meteorology (Met) Department of Pakistan

Geophysical Center Quetta, Met Department of Pakistan has issued a colored seismic zoning map of Pakistan which has categorized Pakistan in four damage zones relating PGA values specified for each zone. Keeping in view PGA value of .05g to 0.0667g Peshawar is in consequential damage zone. The compilation and propagation of information in Pakistan on the subject of natural hazards and earthquakes is the official accountability of Geophysical Center Quetta, having sophisticated network of seismometers installed in major cities and has already uploaded 100 years archives of Pakistan's seismic history on a global internet at the web site: www.met.gov.pk

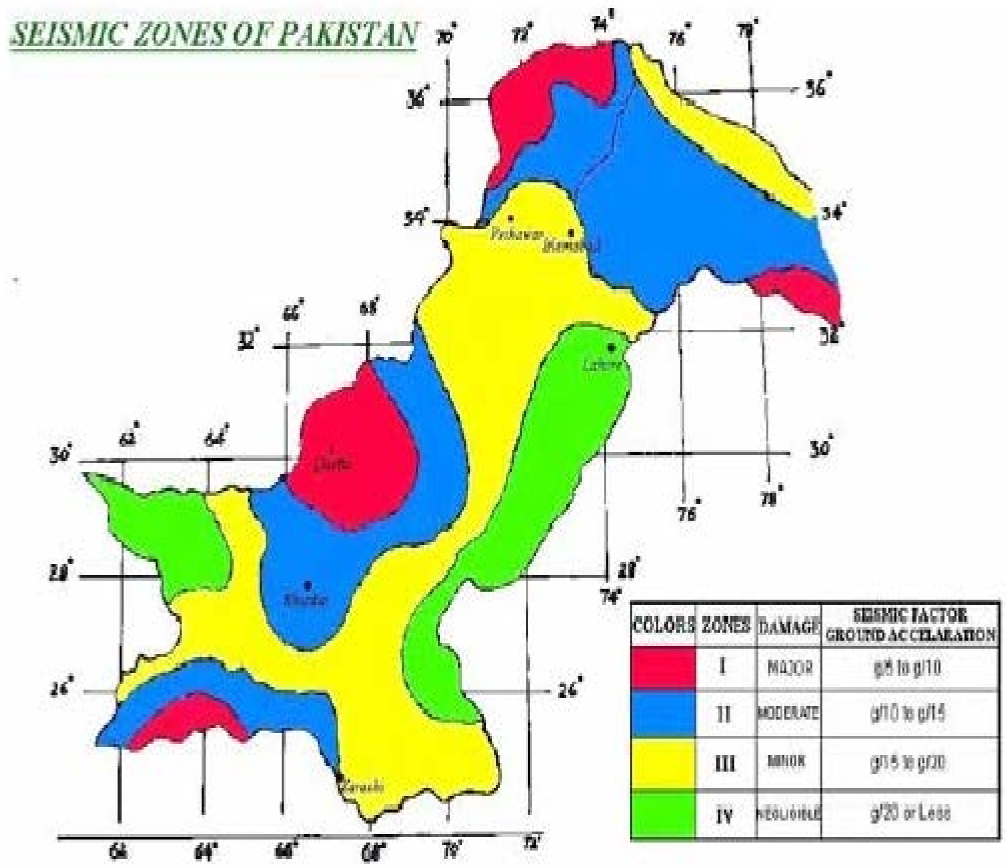

Figure 3.3: Seismic zoning map of Pakistan issued by Geophysical Centre Quetta, Met Department of Pakistan (Rafiq, M., 1999)

3.2.7 Study carried out at Quaid-e-Azam University Islamabad

An area of 200 km around Peshawar has been alienated into 2913 grids of 0.065° x 0.065° size for seismic risk analysis. From USGS instrumental data of Mb≥4 .5 is utilized comprised of 2207 events that occurred during1905 to 1998.The work out recommends 0.06g value of PGA for Peshawar and adjoining areas, figure 3.4 (Lisa, M., and Khwaja, A. A., 2002).

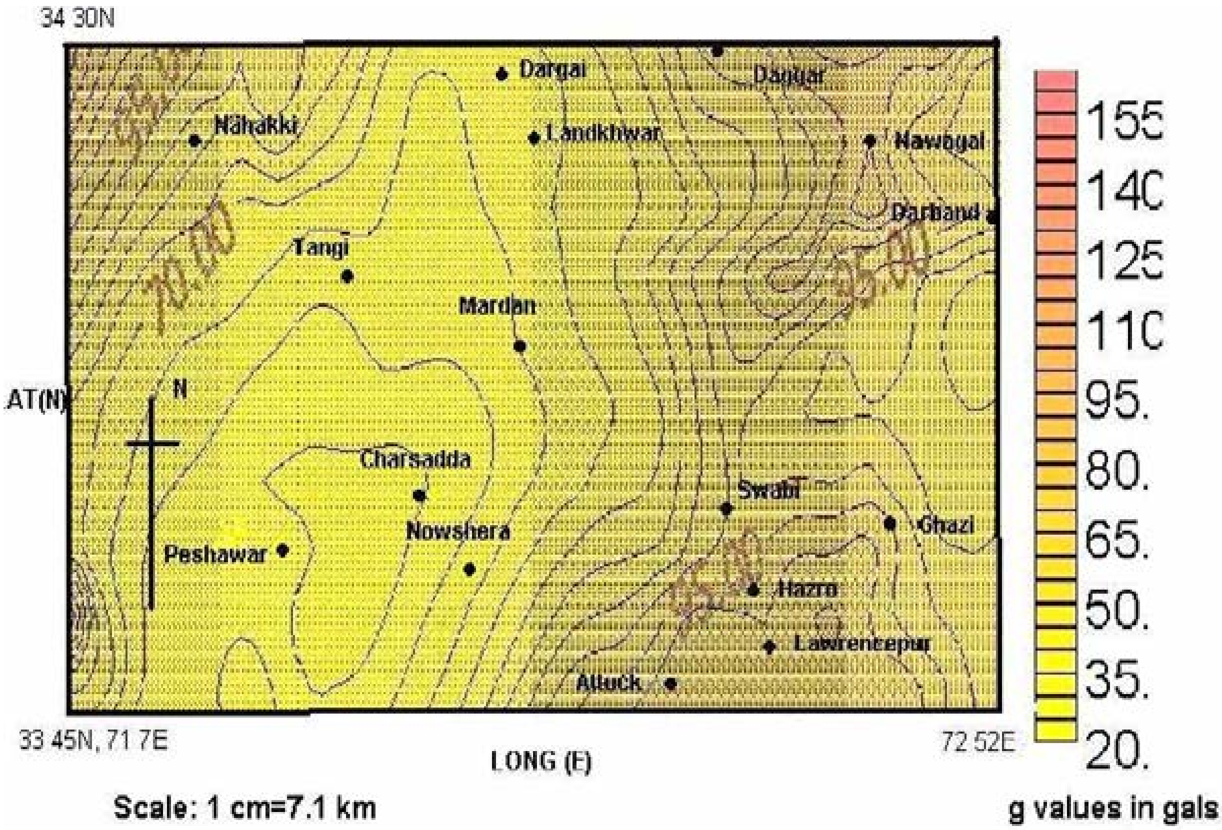

Figure 3.4: Map showing PGA values in gal(1gal=1cm/sec^2)for Peshawar and Adjoining Areas

3.2.8 Conclusions

A huge conflict is tinted between the resource suggesting seismic hazard zoning for Peshawar and adjoining areas indicating that the PGA value of 0.4g by UBC and 0.1g by Met Department of Pakistan has no match. A moderate way is approached by the UNGSHAP suggesting 0.16g to 0.24g.The insufficiency of locally accessible strong motion data and instrumented study of the seismic activity of the local tectonic faults proves to be the conceivable grounds for the divergence. Peshawar and adjoining areas cannot be positioned in a seismic hazard zone at least equivalent to Hindu-Kush Region (HKR) is basically evident by restrained damage earthquake history of Peshawar. As recommended by the Met Department of Pakistan and Mona Lisa the presence of low seismic activity in the vicinity of the city, suggests higher seismic hazard zoning with the fact that Peshawar is regularly rattled by earthquakes originating from HKR.

3.3 EXPERIMENTAL AND NUMERICAL TECHNIQUES

3.3.1 General

In past times for seismic recital evaluation of masonry structures various experimental and numerical techniques have been considered. Static monotonic, quasi-static (slowly reversed cyclic), pseudo-dynamic and dynamic tests using shake table (earthquake simulator) are encompassed in the experimental techniques. Through subversive explosions further alternatives are impact table and excitation of soil. Heterogeneous and homogenous models with finite element and/or interface elements are included in numerical techniques. Stepwise portrayal of first experimental and then numerical techniques with merit and demerits of each method are given below.

3.3.2 Experimental techniques

3.3.2.1 Static *monotonic* and quasi-static loading

Static loading is applied in one direction in monotonic loading whereas in quasi static loading the specimen is subjected to force or deformation loading cycles of predetermined amplitude until failure occurs. Figure 3.5 shows static monotonic/quasi static loading facility at Department of Civil Engineering NWFP UET. Recent study indicates that momentous distinctions in the consequences may be attaining in case of different displacement patterns used to analyze equal masonry wall specimens (Tomazevic M. et al., 1996), investigated the experimental imitation of seismic behavior. It’s significant that testing measures for assessing the seismic recital should replicate the actual seismic behavior of masonry buildings as differences in the results exceed the in general abide scattering of quality of masonry materials. Both static and quasi-static techniques are deficient of this effect.

The researchers in the topical precedents have preferred utilization of pseudo-dynamic and shake table test methods for evaluation of seismic resistance of masonry structures along with the progress of powerful, computer controlled hydraulic actuators, larger and robust High Performance Seismic Simulators (HPSS).

Figure 3.5: Hydraulic actuators in its original form with fix base suitable for monotonic testing for pushing only

3.3.2.2 Dynamic tests using shake table

Structure to be tested is resolutely secured on a table in this technique. Secondly the table is shacked in the desired direction is made possible through Computer controlled actuators proficient of simulating earthquake events. In order to evaluate the vibrant distinctiveness and seismic capacity of structures, it's the only technique might be followed.

As 8 meter x 8meter table, providing six degrees-of-freedom shaking and capable of simulating earthquake measures with displacements of± 600 mm and velocities of 2 m/s is available. Accelerations of 2g's are possible with 300-ton test specimens. At reduced accelerations, specimens up to 300 ton can be experienced on the table figure 3.6

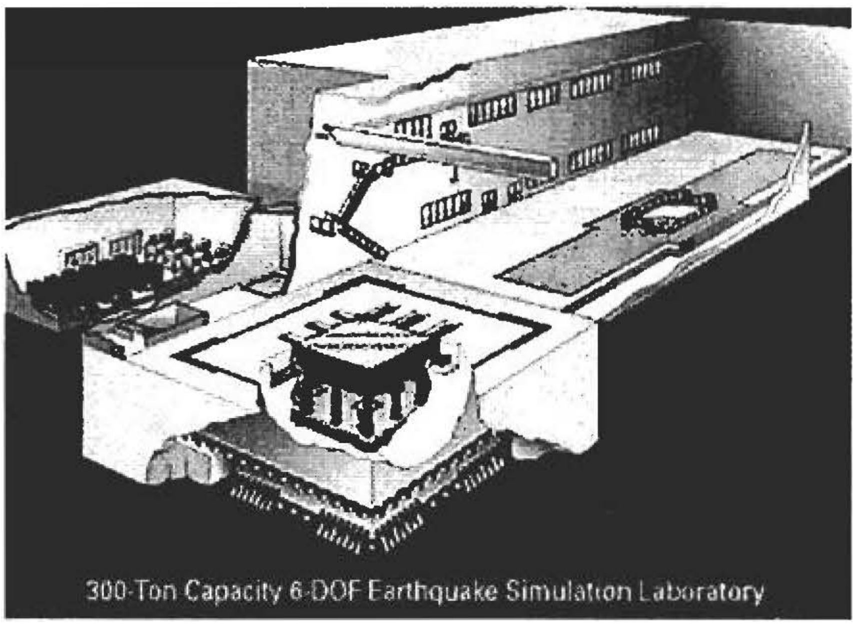

Figure 3.6: High performance seismic simulator at the earthquake simulation laboratory, ministry of construction Tsukuba-Shllbaraki-Ken, Japan

The NWFP UET will be shortly installing its first shake table in the Department of Earthquake Engineering. The table will resemble that given to University of Columbia USA, figure 3.7.

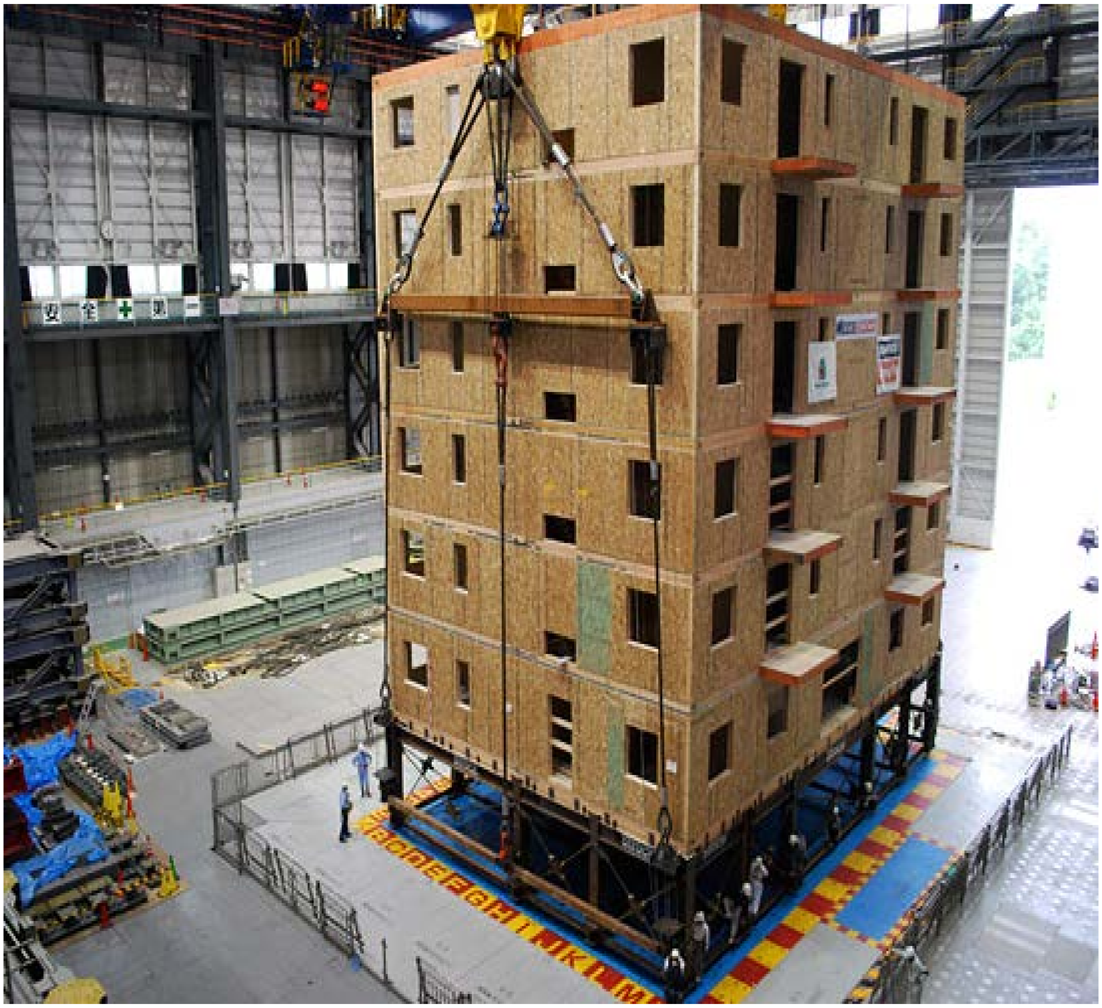

Figure 3.7: Shake Table at University of Columbia, USA

3.3.2.3 Pseudo-dynamic method

The figure given, schematically demonstrate the principle of the pseudo-dynamic scheme. The horizontal displacements at floor represent the kinematics of the building by small number of degrees of freedom. A testimony of an actual (or artificially generated) earthquake ground acceleration account is specified to the computer. The horizontal displacement d of the floor is premeditated through (explicit or implicit) numerical integration of the equation of motion is carried primarily where inertia and glutinous forces are modeled analytically (matrices M and C).Through servo-controlled hydraulic actuators attached to their action wall, displacement is implemented to the structures. The forces F necessary to achieve the required deformation (structural restoring forces) is measured through Load-cells on the actuators are then used to determine the next displacement d to be applied to the building.

Stereo typically, an earthquake of ten seconds is replicated pseudo-dynamically in about one hour. Performing tests in real time scale is of no importance as the initial forces are signified analytically. Throughout the testing procedure, countenance of close monitoring of the damage in the structure, is the chief advantage assured by the pseudo-dynamic method over shaking table testing. The above cited test may be ceased at any stage for detailed evaluation or for preventing a complete collapse. Testing very large models with a modest hydraulic power, in comparison to the conventional test method (shaking table) is the second advantage of pseudo-dynamic method to be considered.

Few developed countries have these high-tech equipment installed in their laboratories as both pseudo-dynamic and shake table facilities are expensive enough and is unaffordable for Pakistan and other under developed countries.

3.3.2.4 Impact table:

A test structure to a series of base excitation pulses through the use of impacting railroad wagons illustrates charge effective way of endangering a test structure [Keightley, W.O., 1981].The test structure positioned in the epicenter of wagon, strikes with wagon undulating down a slope and in turn knockout a stationary wagon.

University of Roorkee in India is admissible for this type of testing service, generating two widely placed pulses in this manner. The facility enables in divulging rough half-sine uni-axial acceleration pulses of 0.15 second duration, up to 2g amplitude, to the base of 20-ton test structure.

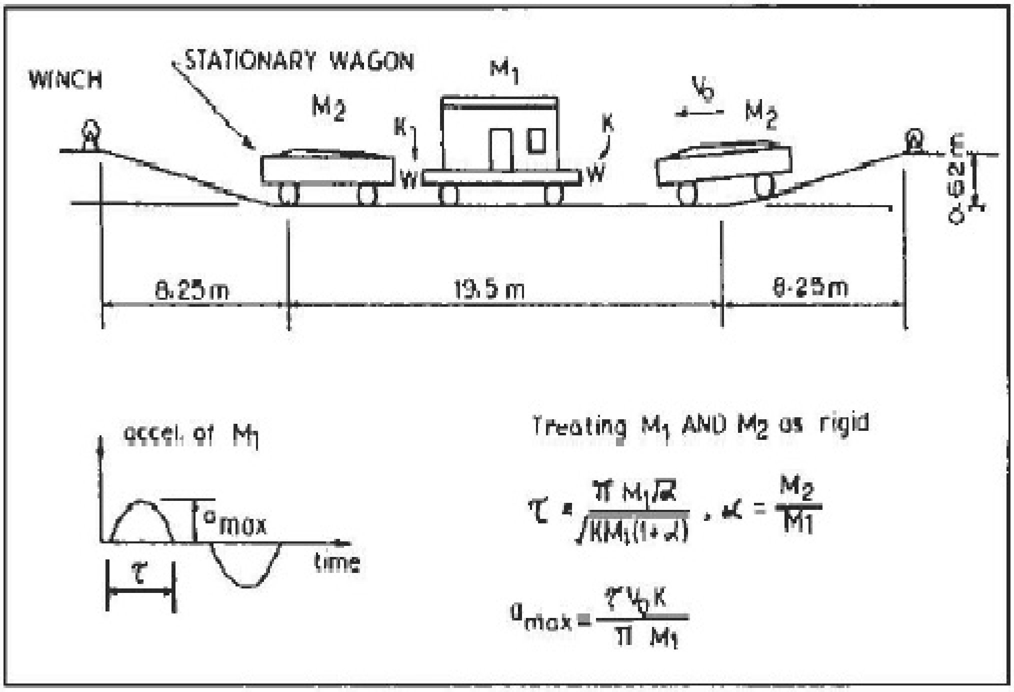

Figure 3.8: Railway wagon shock test facility

The degree of accuracy of dynamic response to a continuous earthquake record can be simulated by the response to distinct and widely spaced pulses are the evident that might arise. An analytical pilot study utilizing linear and bi-linear single degree of freedom systems [Krawinkler, H., and Tolles, E.L., I984] illustrated the facts about the issues involved. In 1952 The SDF systems were endangered primarily to the Taft record, scaled to peak ground acceleration of 0.4g, and to series of individual pulses signifying the foremost acceleration pulses delimited in the record.

In utmost of cases the variance between response to the incessant ground motion and the pulse loading is often dominant. Typically the spectral acceleration of pulse loadings response was less than half that of the ground motion response particularly for elastic systems.

Contingent to the pulse shapes, the natural period of the structural system and the extent of inelastic behavior probable in the structure, earthquake loading and pulse loading may fallout in immensely different comebacks as cleared from the indications. In order to mature a technique for amending pulse shapes and peak accelerations in a manner heavenly efforts in analytical work is required in turn aggregating consequences of a series of pulses signifies unceasing earthquake ground motion.

3.3.2.5 Underground explosions

Underground explosions is the solitary means for imitating earthquake like ground motions in a field environment is diminutive in making for an actual earthquake to occur. Testing in a field environment can come up with the advantages by bearing in mind the soil foundation interface and soil-structure interaction as important facets of seismic response.

Synchronized firing of a planar array of vertical line sources is confined in a method that can produce ground motion. High-pressure explosion products like a rubber bladder, which in turn coerces the surrounding soil [Bruce, J.R., and Lindberg, H.E., 1981], are produced ensuing by the explosion took place in a steel canister in a line source.

Large accelerations along with small velocity and displacements are engendered by Ground motion obtained from firing single charge. Certain a seismic ground motion compressed in time and equally exaggerated in acceleration reassemble those generated motions. The technique is considered feasible for the field testing of models of masonry buildings, showing more appealing if considering that a series of closely set apart pulses can be engendered by setting off synchronized charges in a series of canisters in each line source.

3.3.3 Numerical techniques:

The use of numerical techniques to model brick masonry building system involves consideration of following aspects.

3.3.3.1 Numerical models

Numerical models for masonry being categorized as heterogeneous and the homogenous models, analyze the masonry walls discretizing bricks and mortar through finite element and/or interface elements. An appropriate constitutive rapport is then presumed for each component, taking in account the precise accuracy, characteristics of mortar joints, playing vital role in the global behavior of masonry. When a real wall or building must be analyzed the principal constraint of these models entailing the high computational effort is required (Papa, E. 2001).The properties are obtained from in-situ lab tests or empirical equations in the homogenous models, masonry is assumed as a continuum material, assisting the computational study of real masonry buildings.

3.3.3.2 Method of analysis

The analysis procedure could be either static or dynamic proportionate to on the type of the external actions. The case of irregular structural configuration in plan or elevation is assessed through dynamic method. Significant variances can be pragmatic between static and dynamic analysis force level and their distribution along the height of the structure. Useful information such as modes of vibration and natural periods of the structure, regarding dynamic characteristics of structure is obtainable from Dynamic analysis while static analysis can also be applied successfully for the case of rather regular structures with limited heights.

3.3.3.3 Stress-strain relationship

Non-linear analysis should be of major consideration in case of real stress-strain relationship of masonry, as it possesses non-linear attribute. The areas having analogous stress-strain relationship, the technique if applied, indicates to quite precise fallouts, for the case of reinforced masonry. The qualms in material response lead to ambiguity in assessments of non-linear analysis specifically for unreinforced masonry (URM) exhibiting a fragile conduct. However, linear analysis is kept pertinent in case of URM [Syrmakezis, C. A., and Sophocleous, A. A., 2001].

3.3.3.4 Structural model

The real response of the structure particularly alongside the torsional possessions is preferred droving to the reliability constraint, for masonry structures constructed in seismic areas, 3-Danalysis.

3.3.3.5 Finite element choice

Shell elements, taking into account in and out of plane bending have been used for the masonry models successfully (Genna, F., and Ronca, P. 2001). However six and eight nodded brick elements have also been used, obviously with more computational effort (Melbourne, C., and Gilbert, M., 2001).

3.3.3.6 Simulation of seismic actions

The preeminent replication of the subsequent estimated earthquake ground motion to attack the structure is based usually on the series of accelerograms acquired from the previous seismic events. The subsequent option comes out on the probabilistic basis artificial accelerograms developed from the former seismic events. Seismic road evaluation for the period of exploration technique modern seismic codes is utilized for a design spectrum, as an sachet of certain different accelerograms allied to real earthquakes expected to befall at the place of structure location.

3.3.3.7 Material characteristics

The materials characteristics are comprised of critical input data encompassing the compressive-tensile strengths of the materials, the modulus of elasticity, Poisson’s ratio, and shear modulus having the crucial significance. To assess the subjected parameters Laboratory material tests conferring to the standard procedures are evaluated.

3.4 Types of analysis

On masonry structures it is possible to carry out numerous analysis types. They are summarizable in three groups: linear Analysis, non linear Analysis and limit analysis.

3.4.1 LINEAR ANALYSIS

It is the simplest analysis type in which the material obeying to the Hooke's law is assumed. Therefore the elastic properties of the material and the maximum allowable stresses are necessary. The obtainable results are the deformed shapes and the stress distribution in the structure. In case of stress redistribution it is possible to assume a reduced stiffness in correspondence of the cracked areas. A linear analysis can help in the comprehension of the behavior of a construction with regard to service loads, when the material still shows an elastic behavior. On the other hand, it is not useful into the establishment of the collapse limits. The linear model is particular effective into the identification of the global behavior tendency of the building and the individuation of the points where the structure is subjected to tension stresses able to break the continuity of masonry elements.

In seismic areas, linear Analysis are applicable also in the calculation of structures in presence of seismic forces. More in particular, it is possible to carry out two types of Analysis: the linear static and the modal dynamic ones, as described in the following.

3.4.2 LINEAR STATIC ANALYSIS

The linear static analysis consists in the application of a force system distributed along the height of the building assuming a linear distribution of the displacements. In case of buildings made of a series of floors, these forces are applied at each slab where it is assumed that forces can be concentrated. In case of masonry monumental buildings, like churches (lacking slabs if not on the roof) the problem is overcome in a different way. Whether the walls are modeled with bi-dimensional elements, the horizontal forces, proportional to the weight, can be introduced directly on the shells. In this way, every single geometric variation, like the presence of openings or different thickness in the walls, will be taken into account.

This type of Analysis has been carried out on three-dimensional models of the four study cases.

3.4.3 MODAL DYNAMIC ANALYSIS

The modal analysis, associated with the design response spectrum, can be performed on bi or three-dimensional structures in order to obtain the stresses values in the elements. In this analysis, all the vibration modes with a participating mass bigger than 5% should be considered summing up a number of modes so that the total participating mass is larger than 85%. In order to calculate stresses and displacements in the structure, SRSS or CQC combination rules may be used.

Also this type of Analysis has been considered in the study of the four study cases.

3.4.4 NON LINEAR ANALYSIS

It is possible to study the complete response of the structure from the elastic field through the cracking, until the complete collapse. Different types of non linear behavior exist: *mechanical* (connected to the non linearity of the material), *geometrical* (connected to the fact that the application point of the loads changes increasing the actions) and *of contact* (connected to the interaction of two corps). It is also possible to carry out non linear Analysis with damage models very useful into the evaluation of the stiffness loss at global and local level. This type of analysis requests the elastic and inelastic properties and the strength of the material. The results that can be gained are the strain behavior, the stress distribution and the collapse mechanism of the structure.

In addition to the vertical ones, in presence of horizontal actions, a non linear static analysis can be carried out.

3.4.5 NON LINEAR STATIC ANALYSIS

The non linear static analysis consists into the application on the structure of the vertical loads (self weight and dead loads) and a horizontal forces system monotonously increasing until the reaching of the limit conditions. The method, used also in the evaluation of the bearing capacity of existing buildings, is comprised in the last seismic codes.

This type of analysis has also been carried out in this study on bi-dimensional elements extrapolated from the whole structures of the four study cases.

3.4.6 LIMIT ANALYSIS

This analysis type is aimed at the evaluation of the collapse load. The theoretical principles that allow making a seismic check through the limit analysis are conceptually simple but result of complex and delicate application for the following reasons: it is not useful to interpret the cause and the extension of the cracks, strains or other damages not directly related to the collapse generation; furthermore its use is fairly difficult in complex structures with a lot of elements.

The two theorems of the limit analysis, due to Godzev (1938) and Drucker, Prager and Greenberg (1952), are:

1) Static theorem: the plastic collapse load multiplier γp is the largest of all the multipliers $\gamma\sigma$ correspondent to the statically admissible set ($\gamma p \geq \gamma\sigma$). For statically admissible set, a stress distribution in equilibrium with the external forces that in no point violates the plastic conditions is intended.

2) Kinematic theorem: the plastic collapse load multiplier γp is the smallest of the entire multipliers $\gamma\sigma$ correspondent to possible collapse mechanisms ($\gamma p \leq \gamma\sigma$). For kinematically admissible set, a kinematism or a distribution of velocity of plastic deformations, related to the distribution of plastic hinges, which satisfies the condition of kinematic compatibility, is intended.

These theorems generate two correspondent calculus methods of the collapse multiplier:

1) Static method: it consists into assuming a distribution of statically admissible stresses dependent by a certain numbers of parameters and searches them so that the correspondent load multiplier is maximum.
2) Kinematic method: it consists into assuming a collapse mechanism dependent on some geometrical parameters and in the following minimization of the correspondent multiplier to the considered mechanism.

According to the uniqueness theorem, a multiplier that is statically and kinematically admissible coincides necessarily with the collapse multiplier.

3.4.7 LIMIT ANALYSIS APPLIED TO MASONRY STRUCTURES

The masonry constitutive model is of fragile type with a high value of collapse in compression compared to tension. The collapse tension stress is not only small but is characterized by a high uncertainty of evaluation because of the great scattering of the experimental results as well. This is the reason why in limit analysis a simplified diagram of linear indefinite elasticity on compression side and null collapse strength on tension is admitted. The masonry constitutive model coincides with materials called NRT (non resistant tension) and is defined by the following relationship:

$$\varepsilon = C\sigma + \varepsilon^{(f)}$$

$\sigma \leq 0$ (lack of tension)

$\varepsilon^{(f)} \geq 0$ (cracking strains)

$\sigma\ \varepsilon^{(f)} = 0$ (lack of internal dissipation)

The condition of convexity and the respect of the normality condition to the limit surface from the cracking strains, imply a tight connection between the theory developed by NRT materials and the classic theory of perfect plastic materials.

The study of masonry structures through limit analysis investigates the very essential aspects of the behavior at collapse and, at the same time, seems to match modern analysis techniques with geometrical static principles rising from traditional theories. The applicability of limit analysis to masonry structures has been firstly investigated by [Coulomb, 1773], in which a theory on the collapse behavior of masonry elements was formulated. More recently, [Koorian, 1953] demonstrated how stone masonry can be dealt with through plasticity theorems. However, the main contribution in this regard is by [Heyman, J. 1966, 1969, 1995], who clearly stated some hypotheses on the mechanical behavior of masonry, (already adopted, though implicitly, in the traditional pre-elastic theories). The following assumptions regarding material properties are made:

1) *Masonry has zero tensile strength*. Although this statement is conservative, it has to be considered that even if stones have a certain resistance, only very small tension forces will be transferred across joints because of the weakness or absence of the mortar.
2) *Infinite compression strength of the blocks*. In most cases, collapse of masonry structures is not governed by compression failure, but is due to cracks opening and mechanisms formation: this assumption is slightly un-conservative.
3) *Sliding of a stone, or of part of the structure, upon another cannot occur*. Based on the experimental evidence that, generally, the angle between the thrust line and the sliding surface is greater than the friction angle.

With these assumptions, the only possible collapse mode is the rotation of adjacent blocks about a common point, so that masonry behaves as an assemblage of rigid bodies held up by compressive contact forces. The collapse is characterized by the formation of hinges among the single parts.

Uniqueness and safe theorems can be then respectively expressed as follows:

"If a thrust line representing an equilibrium condition for the structure under certain loads lies fully within the masonry, and allows the formation of sufficient hinges to drive the structure into a mechanism, then the structure is about to collapse. Further, in case of proportional loads, the load proportionality factor at collapse is unique."

"If a thrust line, in equilibrium with the external loads and lying wholly within the structure, can be found, then the structure is safe."

With these statements and under the outlined hypotheses, collapse analysis of masonry structures basically consists in seeking a thrust line, which is actually the graphical representation of equilibrium conditions, passing through a number of hinges sufficient to transform the structure into a mechanism.

Though the approach is conceptually simple and well posed from a theoretical point of view, a few points on its applicability and reliability can be arisen. First of all, infinite compression strength is assumed, while experience has shown how structures made of materials with poor mechanical properties often do not develop mechanism-like collapse, rather large portions of masonry crush. Possibly, finite values of compression strength can be accounted for by moving the position of the hinges from the external boundary towards the inside of the masonry. Secondly, it must be said that though limit analysis actually reveals the weakest points of the structure and provides a bound of the admissible horizontal action, it neglects, due to material assumptions, a few structural inelastic capacity issues, so that the safety assessment turns out to be fairly pessimistic.

The application of limit analysis in studying the collapse of structural elements under seismic-type lateral loadings seems to be very appealing. As a matter of fact, on account of the hypotheses assumed regarding the material properties and the mechanism formation schemes, the horizontal bearing capacity of a masonry element can be derived as a function of the geometry alone. In this regard several authors have made use of limit analysis for treating simple masonry elements, since complex buildings are often seen as assemblages of elementary structural schemes, so that the overall capacity can be somehow derived from the ones of the components. Although in complex structures it might be difficult to find the correct failure mechanism by limit analysis, it is outlined that this simplified modeling combines, on one side, sufficient insight into collapse mechanism, ultimate stress distributions and load capacity, and on the other, simplicity to be caste in a practical computational tool.

Another appealing feature of limit analysis is the reduced number of necessary material parameters, given the difficulties in obtaining reliable data for historical masonry.

Chapter 4

Development of The FEA Model for Bridge

4.1 Introduction

Real Bridge study is thought-out for the finite element procedure distinctively subject to the masonry configuration analysis of seismic activity. Destructive, non-destructive and experimental testing techniques are followed to value the material assigned for micro –analysis. Young' Modulus is adopted in macro-analysis. The specified assessment for a prism comprised of two blocks and a mortar joint may evaluated in correspondence with the homogenized masonry in view of the equations from literature or analytical evaluation. Masonry's response is subjective to the Young's Modulus of Elasticity. Isotropic or orthotropic masonry material is kept in view in case of 2-D or 3-D elements implementation depending on exhibited distinct directional properties. Finite Element Program is carried out for modeling [MIDAS Civil, 2013].

4.2 Type of elements used

Solid element is used in this research. There are 4, 6 or 8 nodes in a three-dimensional space define a solid element. The element is generally used to model solid structures or thick shells. A solid element may be a tetrahedron, wedge or hexahedron. Each node retains three translational displacement d.o.f. The element is formulated according to the iso-parametric Formulation with Incompatible Modes. Element d.o.f., ECS and Element types are described as under.

The ECS for solid elements is used when the program calculates the element stiffness matrices. Graphic displays for stress components are also depicted in the ECS in the post-processing mode. The element d.o.f. exists in the translational directions of the GCS X, Y and Z-axes. The ECS uses x, y & z-axes in the Cartesian coordinate system, following the right hand rule. The origin is located at the center of the element, and the directions of the ECS axes are identical to those of the plate element, plane number 1. There are three types of elements, i.e., 8-node, 6-node and 4-node elements, forming different shapes as presented in Figure 1.33. The nodes are sequentially numbered in an ascending order starting from N1 to the last number.

Solid Element is used for modeling three-dimensional structures, and its types include tetrahedron, wedge and hexahedron. Pressure loads can be applied normal to the surfaces of the elements or in the X, Y, and Z-axes of the GCS. The use of hexahedral (8-node) elements produces accurate results in both displacements and stresses. On the other hand, using the wedge (6-node) and tetrahedron (4-node) elements may produce relatively reliable results for displacements, but poor results are derived from stress calculations. It is thus recommended that the use of the 6-node and 4-node elements be avoided if precise analysis results are required. The wedge and tetrahedron elements, however, are useful to join hexahedral elements where element sizes change.

Solid elements do not have stiffness to rotational d.o.f. at adjoining nodes. Joining elements with no rotational stiffness will result in singular errors at their nodes. In such a case, MIDAS/Civil automatically restrains the rotational d.o.f. to prevent singular errors at the corresponding nodes. When solid elements are connected to other elements retaining rotational stiffness, such as beam and plate elements, introducing rigid links (master node and slave node feature in MIDAS/Civil) or rigid beam elements can preserve the compatibility between two elements. An appropriate aspect ratio of an element may depend on several factors such as the element type, geometric configuration, structural shape, etc. In general, it is recommended that the aspect ratio be maintained close to 1.0. In the case of a hexahedral element, the corner angles should remain at close to 90°. It is particularly important to satisfy the configuration conditions were accurate analysis results are required or significant stress changes are anticipated. It is also noted that smaller elements converge much faster.

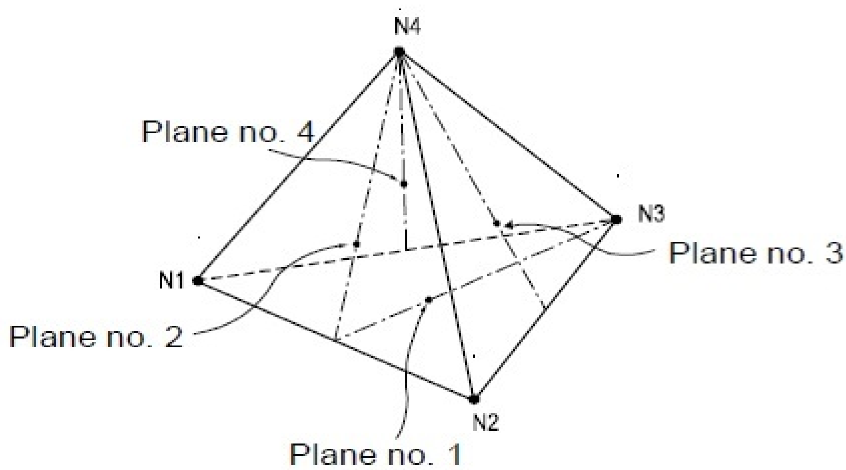

Figure 4.1: 4 Node Element (Tetrahedron)

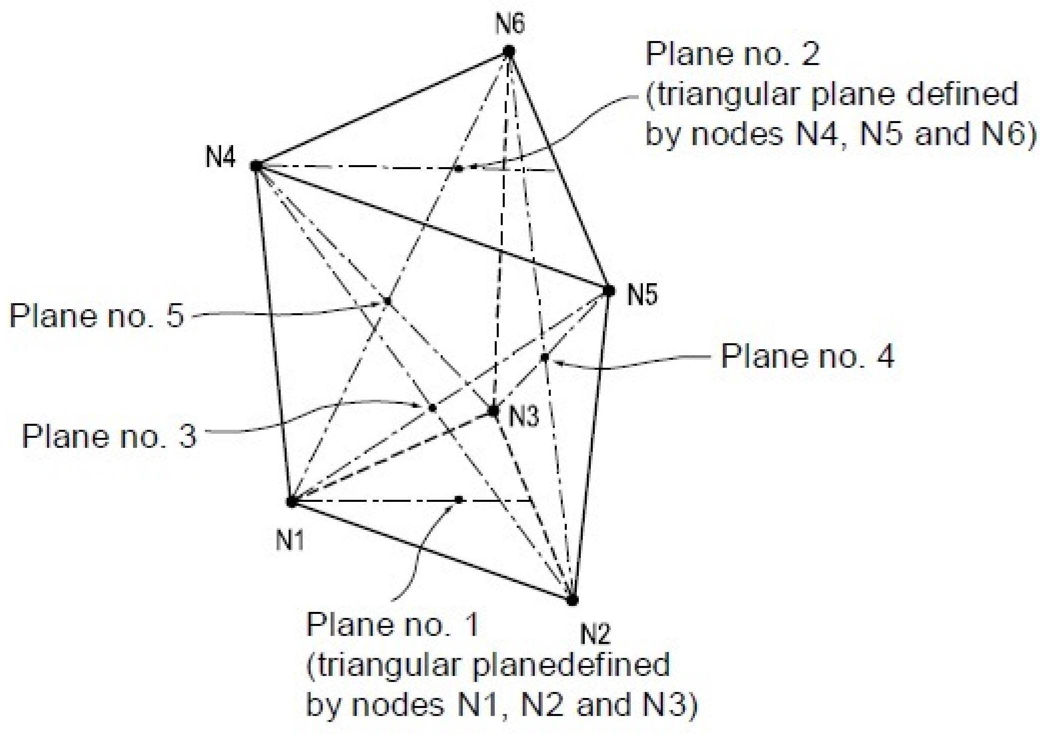

Figure 4.2: 6 Node Element (Wedge)

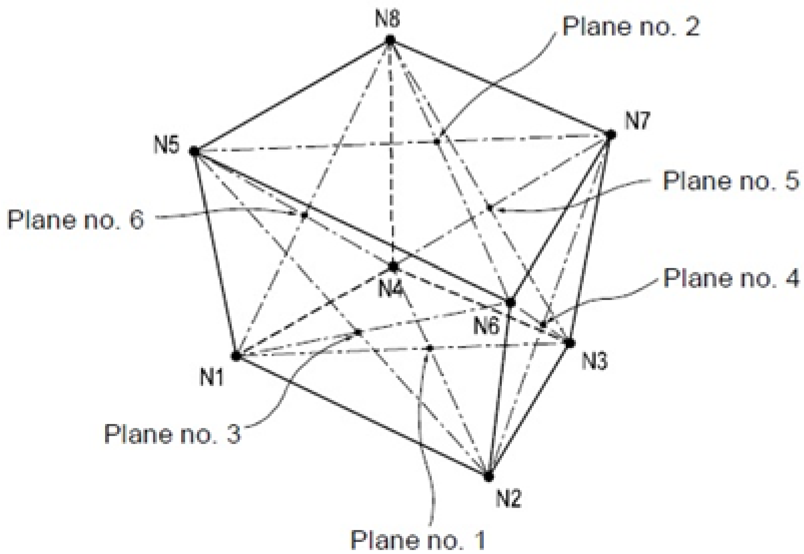

Figure 4.3: 8-Node Element (Hexahedron)

4.3 Application of the FE method

An assumption that characteristics of masonry's geometry and loading conditions consent it to be precisely simulated by plane structural members, provides base to the practice of two-dimensional FE for masonry structures. Particularities in the works hinder assumptions, especially when standards for high analysis results are imposed for chronological buildings and monuments' analysis. FE meshing and apprehend grid density is vital for the accuracy of stress output. Stress distribution variation is intense in openings and members' intersection area as compared to the rest of wall surface, eventually leading for attaining comprehensive results by two dimensional FE. Shells final geometry contradicts their two-dimensional character and FE thickness i.e. wall width is proportional to their area dimensions. Architectural particularities of wall width variations demonstrate the presence of inadequate model formulation. Using up of two-dimensional elements for analysis may leads to inaccuracy in stress analysis results which literally can be overcome by the implementation of three-dimensional "solid" FE. In the matter of abutment of bridge specified for hydrodynamic pressure or earthquake, "solid" elements are implemented, supporting the attainment correlated data of stress variation. 4-node tetrahedron pyramid, the 6- node pentahedron wedge and the 8-node hexahedron brick are the standard “solid” FE. Mid-side nodes are considered for accomplishing refinement. Jacobian matrix’s computation and inversion are required for formulation of strain-displacement matrix. Displacement nodes are obtained from stiffness of matrix FE. Total number of nodes and elements are 7843 and 7136 respectively as shown in the figure**.**

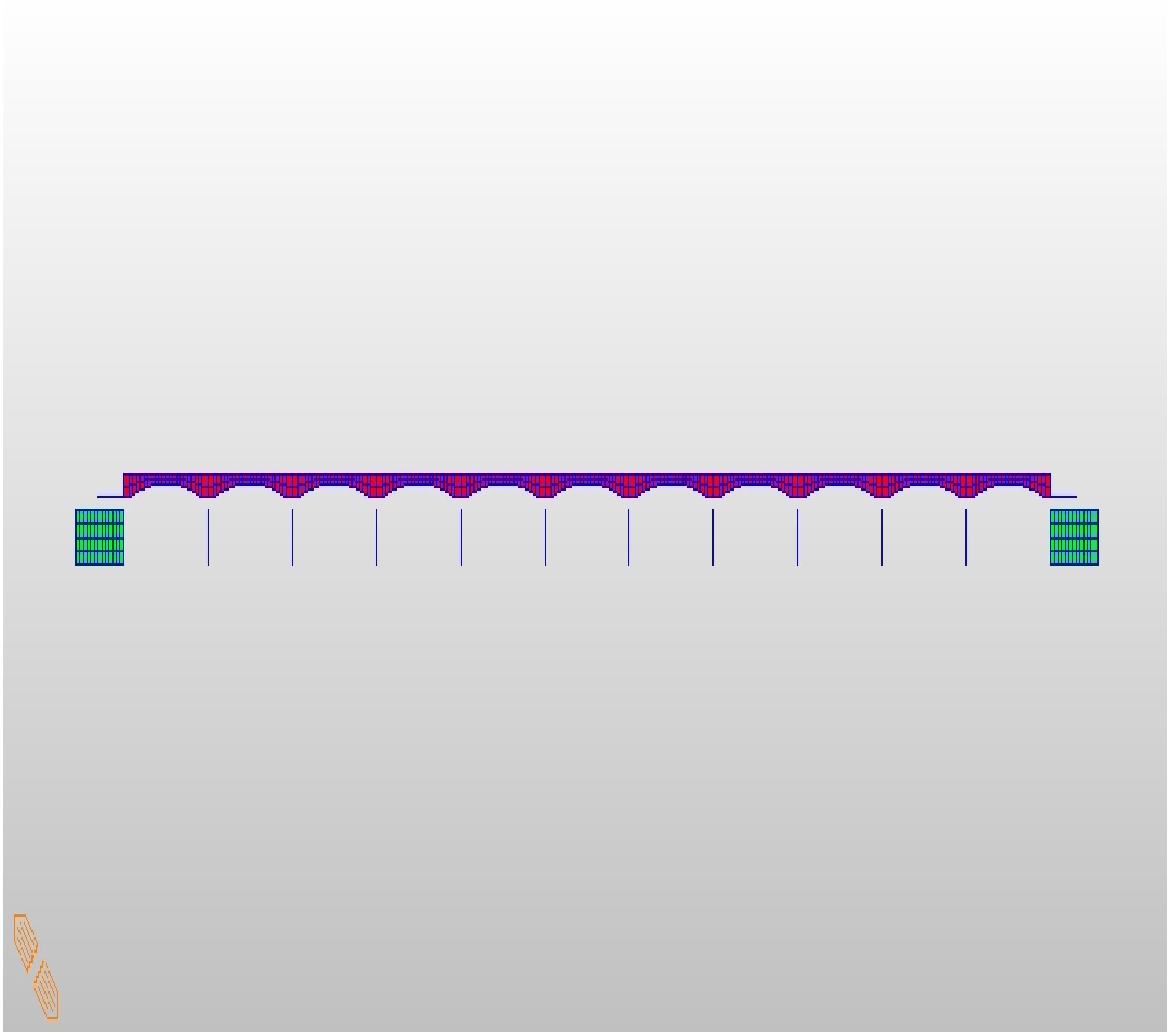

Figure 4.4: Front View of model bridge

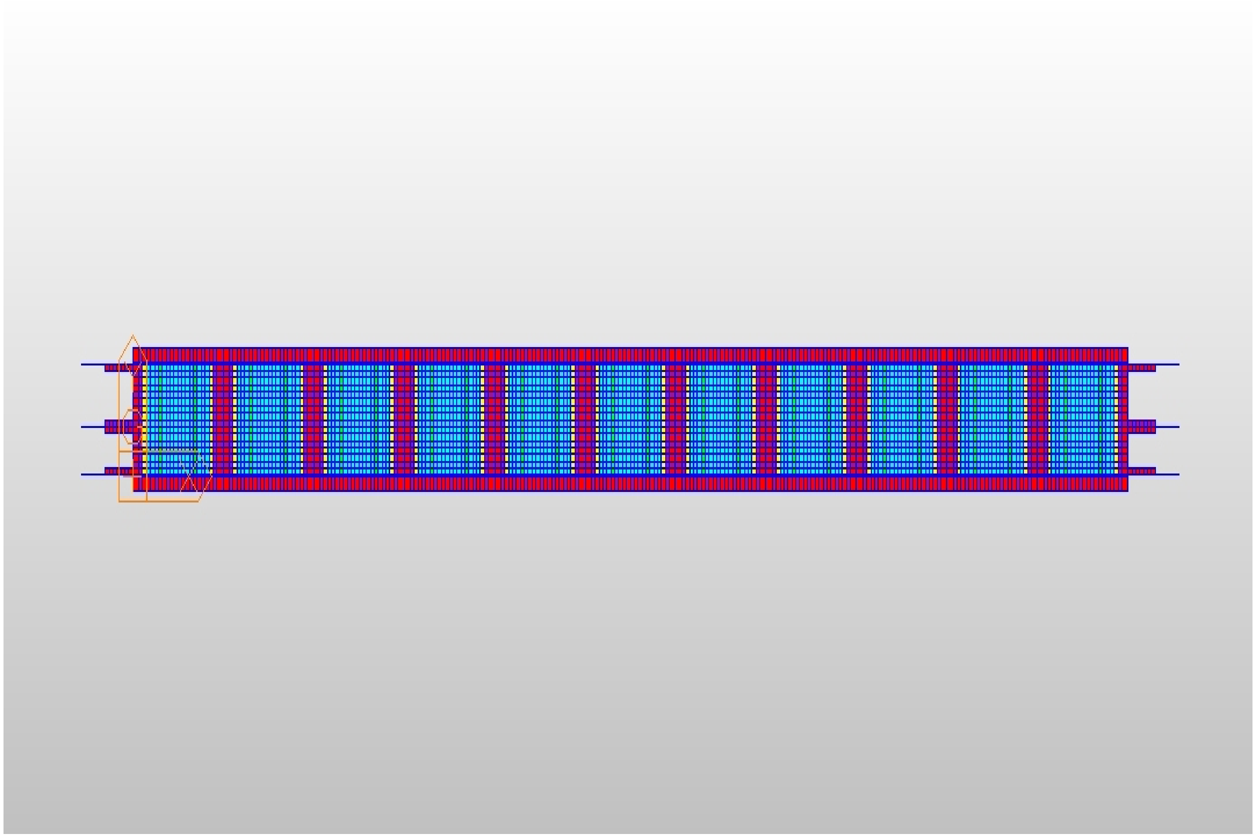

Figure 4.5: Top View of Model Bridge

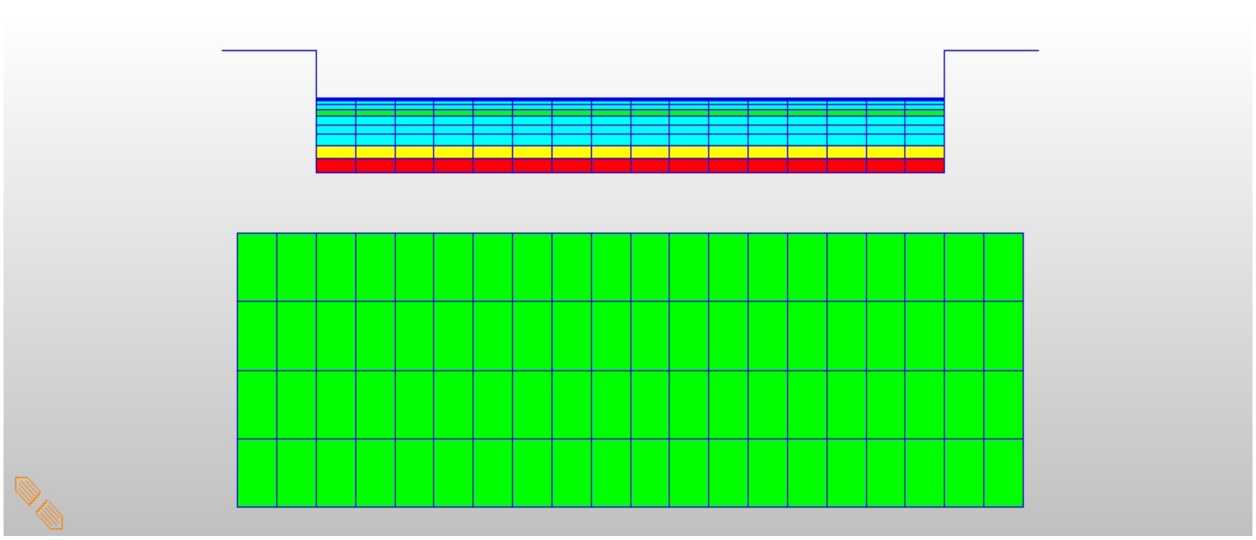

Figure 4.6: Side view of Model Bridge

Three dimensional models have been developed for such bridge analysis. Bridge structure of full scale model has been analyzed in the given figure. Ground weight is considered at the buttresses irrespective of the static load. Perfect joints are observed along with the rigid links.

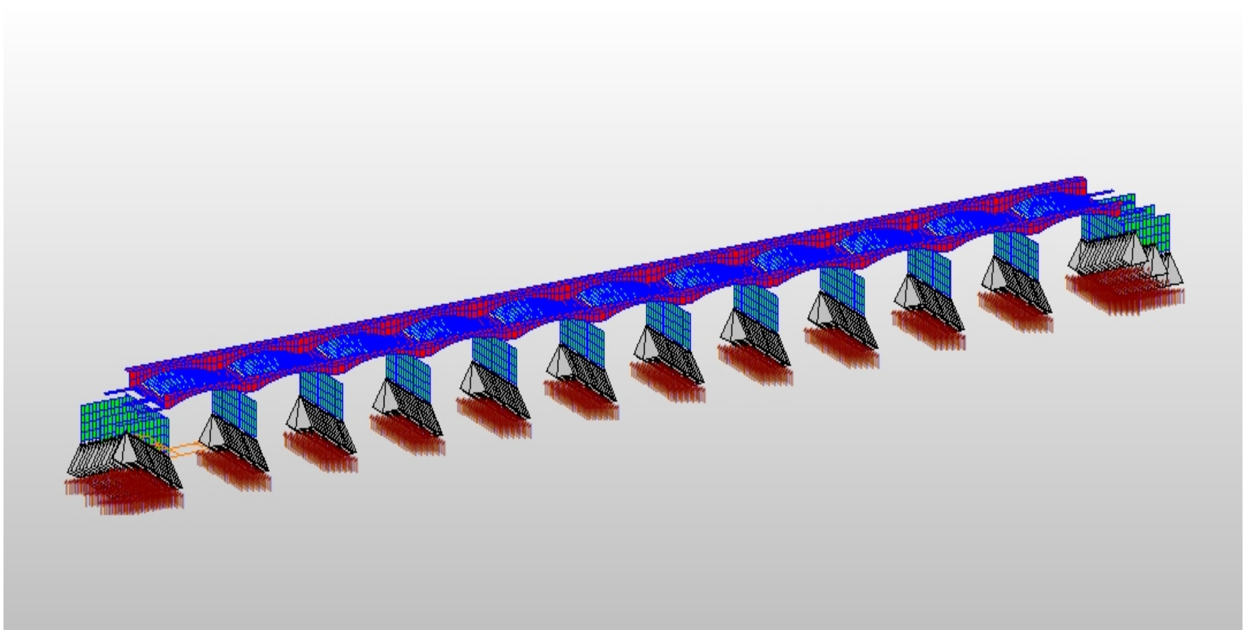

Figure 4.7: Bridge Modal showing support conditions

4.4 Materials

Table 4.1: Material properties used in analysis

Type	40	25
Material	Concrete	Concrete
Code	AASHTO(RC)	AASHTO(RC)
Elasticity	3.60E+07	2.85E+07
Poisson	0.2	0.2
Material Type	Isotropic	Isotropic

Material Type 40

Characteristic Resistance (R_{ck}) = 5800 psi

Cylindrical Resistance f_{ck} = 4800 psi

Young`s Modulus (E) = 5228600 psi

Material Type 25

Characteristic Resistance (R_{ck}) = 3600 psi

Cylindrical Resistance (f_{ck}) = 3000 psi

Young`s Modulus (E) = 4133500 psi

Concrete

Compression resistance R_{cm} = 7700 psi

Weight for unity of volume (inclusive armors) (γ)= 2.5 kN/m^3

Level of limited knowledge (*LC2*)

Factor of confidence = 1.20

Steel

The characteristics of the steel are is constituted by FeB44K.

Yielding stress: f_{yk} = 62336 psi

Level of limited knowledge (*LC2*)

Factor of confidence = 1.20

4.5 Seismic Parameters for FEA Bridge Model

Recently Pakistani territory is being categorized on seismic grounds. Distinctions among the earthquake designs of national zones are covered with the seismic codes directly curing the serious gaps of risks regarding to civil protection. The basic aim concerned with value of Ag characterized from of return than at least 475 years. In 50 years, 10% probability of seismicity is observed. The absorption of seismic energy and the rescue efforts related to the post-seismic needs to be kept in consideration for low cases. Subsequent objectives correlates with the value of Ag specified from return of 100 years. This seismic value comes up with 50% in 50 years, concerning with the negligence of small damages, not disturbing the traffic.

Characterization of Ag value from of 475 year-old return for catastrophic earthquake, comes up with crucial status. The probability of the catastrophic allied to the 2% in 50 year. It doesn’t have concern with the work collapse, it can’t be used anymore and the determination its collapse is made possible through it.

AASHTO LRFD in the Region is kept in view for seismic verification. It provides fundamentals for the Civil Protection during seismic events also requiring safety evaluation estimated from the AASHTO LRFD.

Three states limits PGA of Collapse (CO), Severe Damage (DS), Limited Damage (DL) along with their relationship with the acceleration, defining its three levels is the main objects to be achieved. Pakistani Bridges belonging to the Zone 2B structures have useful life of 100 years. As described in the AASHTO LRFD, elastic response spectrum is used for verification development. Seismic parameters of the bridge are expresses as maximum (Ag) acceleration having probability of 10% in 50 years, characterized by shear velocity Vs> 800 m/s [AASHTO LRFD, 2012]. Ag = 0.25g, acceleration has been assumed. For horizontal and vertical directions parameters from Table 4.2 and Table 4.3 are considered for elastic response spectra of the horizontal components. The response spectrum in according to AASHTO LRFD with Peak Ground Acceleration 0.25 g as shown in Fig.4.8 and Fig.4.9 for horizontal and vertical direction, respectively.

Figure 4.8: Design Spectrum for Bridges (Horizontal), AASHTO 2012

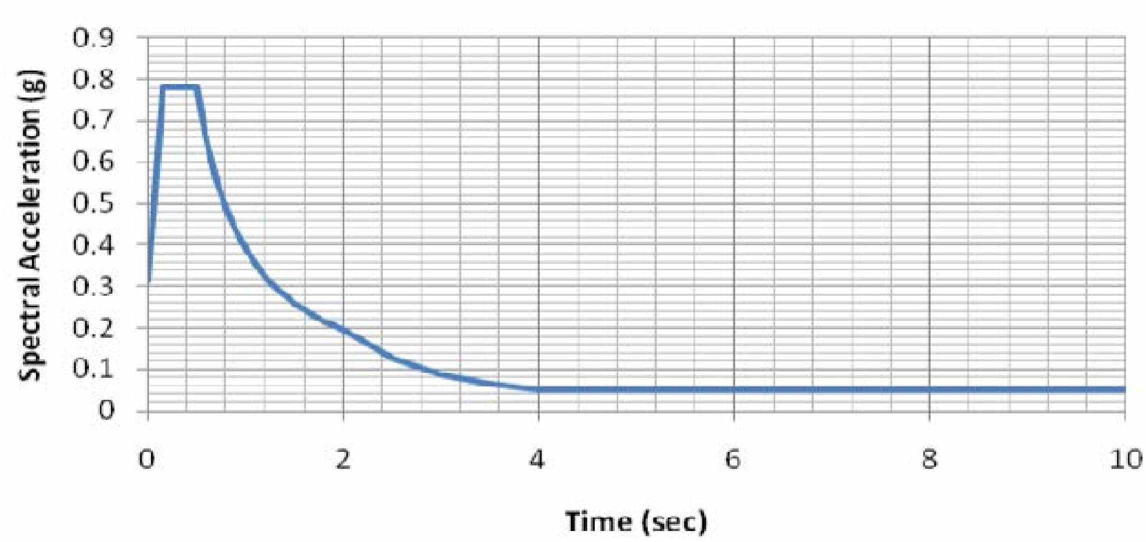

Table 4.2: Response Spectrum Parameters (Horizontal Direction)

<table>
<tr><td>S.No</td><td rowspan="2">Soil Type</td><td rowspan="2">C</td></tr>
<tr><td>1</td></tr>
<tr><td>2</td><td>Damping Ratio</td><td>5%</td></tr>
<tr><td>3</td><td>G</td><td>B</td></tr>
<tr><td>4</td><td>S</td><td>1.25</td></tr>
<tr><td>5</td><td>T_b</td><td>0.156</td></tr>
<tr><td>6</td><td>T_c</td><td>0.553</td></tr>
<tr><td>7</td><td>T_d</td><td>1.922</td></tr>
<tr><td>8</td><td>A_g</td><td>0.25g</td></tr>
</table>

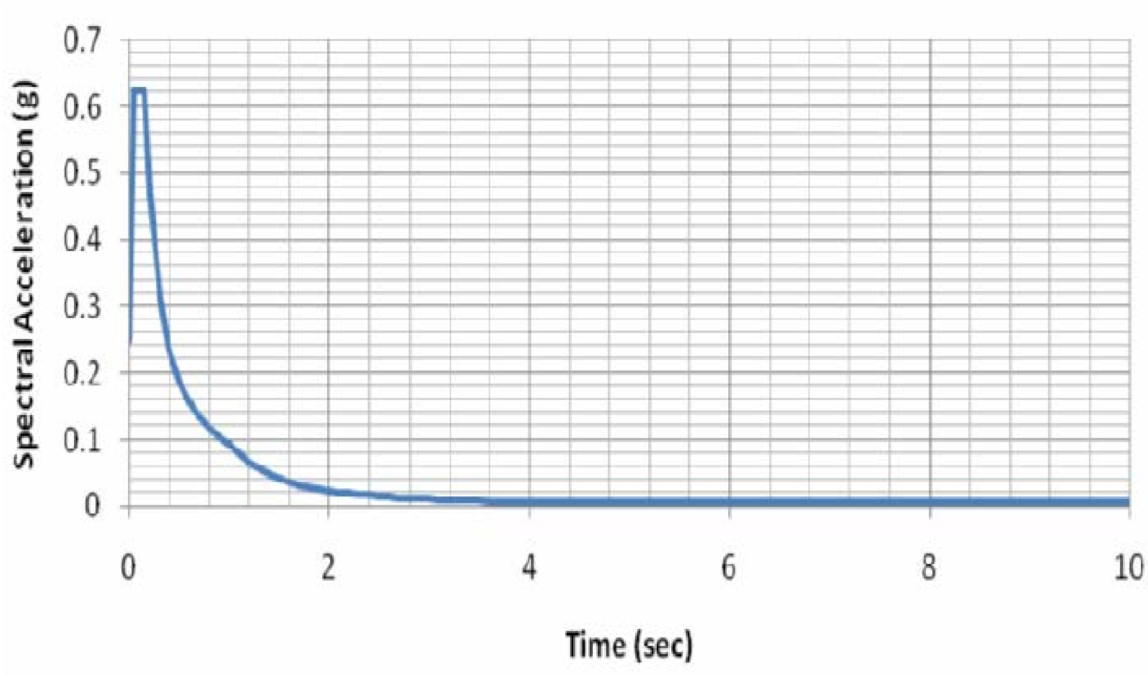

Figure 4.9: Design Response Spectrum Parameters (Vertical Direction), AASHTO 2012

Table 4.3: Response Spectrum Parameters (Vertical Direction)

S.NO		
1	Soil Type	C
2	Damping Ratio	5%
3	G	B
4	S	1.0
5	T_b	0.049
6	T_c	0.154
7	T_d	1.002
8	A_g	0.25g

4.6 Simulated Time History Response

Adoption of a simulated time history is referred as the alternative approach. The code SIMQKE [Gasparini and Vanmarcke, 1979] is used for the signal generation. With smoothed response spectrum SIMQKE builds a power spectral density function producing sinusoidal signals of random phase angles and amplitudes. In a specific time domain, iterative filtering of a series of white noise with a trapezoidal function of amplitudes is performed. Due to having inadequate low frequency content and excessive energy content, researchers illustrated doubts about artificial accelerograms.

The coefficient of viscous damping:

$\zeta = 5\%$

The strategic interest of the bridge it is assigned a factor of importance:

$\gamma = 1.4$

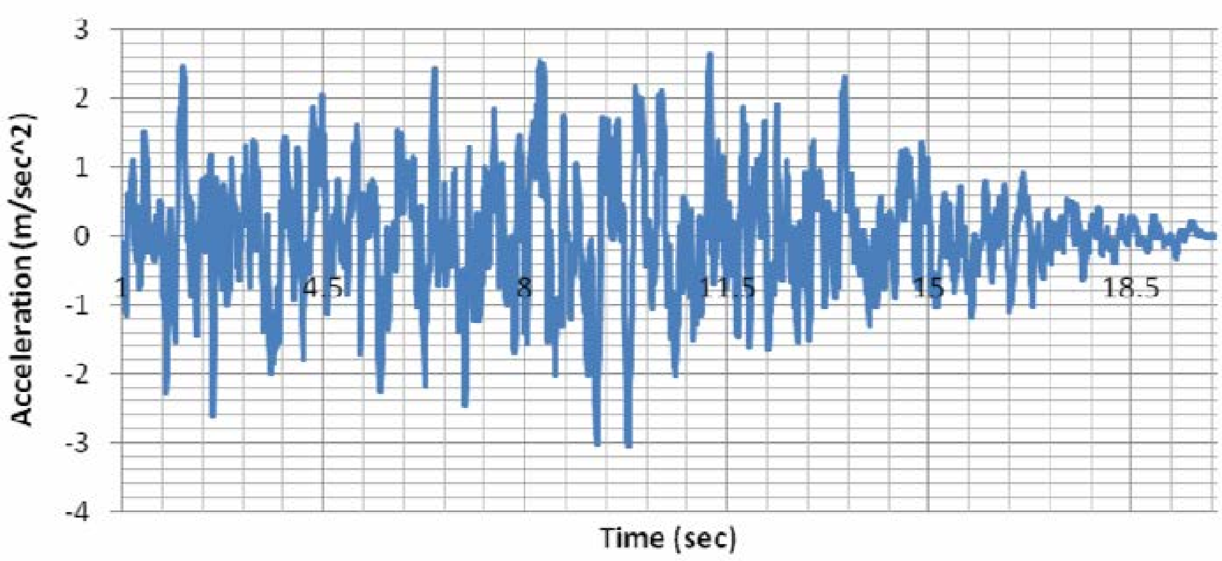

Figure 4.10: Simulated Accelerogram, Time History

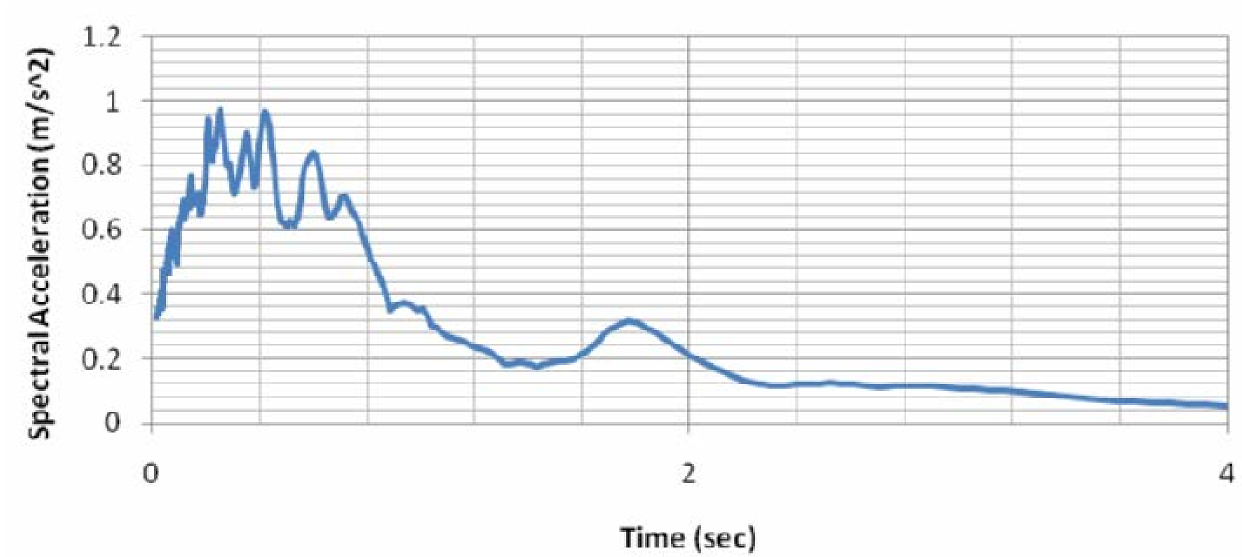

Figure 4.11: Simulated Accelerogram, Response Spectrum

1.5 with the SL of Severe Damage (DS) is multiplied with the accelerations of the elastic response spectrum for the State Limit (SL) of Collapse (CO) shown in the following Table 4.4:

Table 4.4: Three Levels Peak Ground Acceleration

Acceleration of Three SL		
SL_{DL} PGA $_{soil}$ 50% [g]	**0.175**	$SL_{DS}/2.5$
SL_{DS} PGA $_{soil}$ 10% [g]	**0.438**	A_g. γ. S
SL_{CO} PGA $_{soil}$ 02% [g]	**0.657**	$SL_{DS} \times 1.5$

In figure, the detail graphic elastic spectrum adopted for modal dynamic analysis is shown.

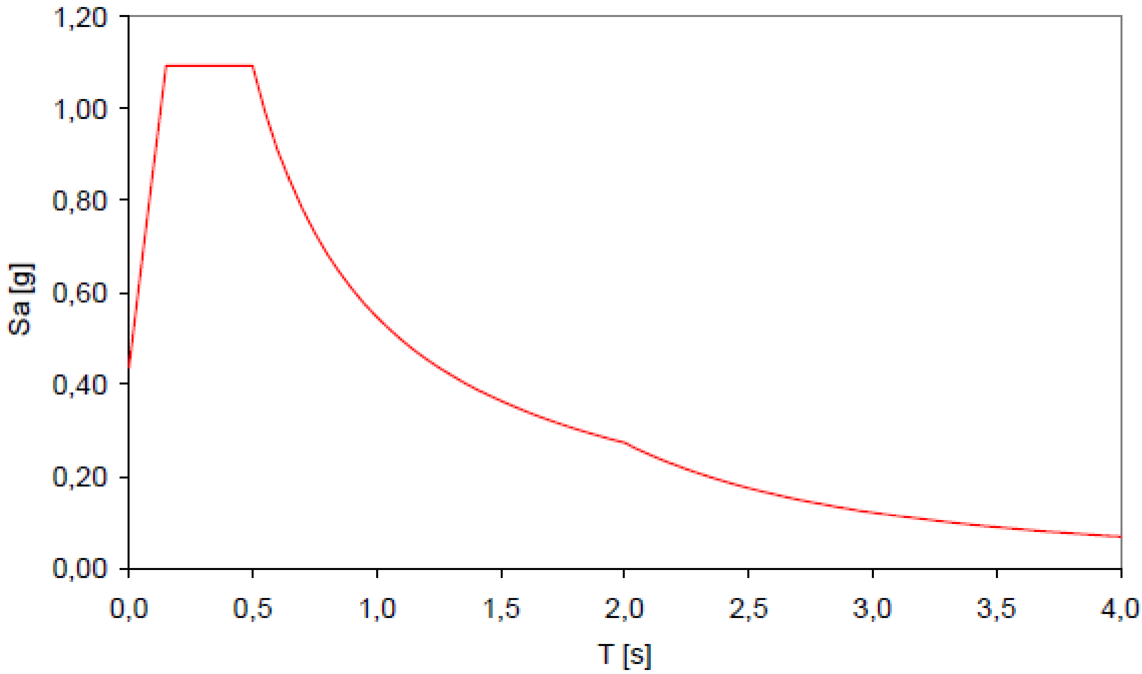

Figure 4.12: Pseudo-Spectral Acceleration

4.7 Analysis of the Seismic Response of Structure

Safety evaluation with seismic action is the investigation of construction of concerned bridge about its code and withstand against seismic damage. Verification is based on three limit states. The present study is based on the analysis of road infrastructure in the provincial and town plans of emergency. Level 1 being considered as less severe along with three state limits (SL of Collapse, SL of Severe Damage, and SL of Limited Damage) and related acceleration probability of 2%, 10% and 50% in 100 years. Level of knowledge adjusted (LC2) influences the absence of tests and verifications in site.

The typology of the bridge object of verifications taken in account for the Preliminary analysis and the vulnerability of the same bridge is associated to the vulnerability of the components of structures.

Table 4.5: Mode No Vs Mass (%)

Mode No	TRAN-X MASS(%)	Mode No	TRAN-Y MASS(%)	Mode No	TRAN-Z MASS(%)
1	76.15	9	0.02	2	0.12
4	0.05	11	0.01	5	0.05
10	0.05	13	0.06	11	0.19
12	0.63	15	0.18	13	0.06
14	0.37	17	0.15	17	0.18
16	0.81	19	1.65	19	0.06
18	0.23	21	25.03	21	0.15
22	1.54	23	0.02	23	4.24
25	0.32	24	0.03	24	6.86
27	0.23	28	0.22	26	2.25
31	0.06	30	0.03	28	0.02
33	0.32	49	0.08	30	0.21
35	0.71	50	0.04	32	0.16
37	0.04	52	4	34	0.72
39	0.03	54	35.98	36	0.84
40	0.01	55	0.06	38	0.01
45	0.02	56	1.8	44	0.04
51	0.02	60	1.54	47	0.25
62	0.01	61	7.98	50	0.09
65	2.24	63	0.64	52	20.96
67	1.03	64	3.25	54	1.53
68	0.91	69	0.3	55	0.87
74	0.03	71	0.06	56	0.03
76	0.01	72	0.14	60	0.12
78	0.17	73	0.43	61	0.05
79	0.47	75	0.03	63	0.06
85	0.01	77	0.09	64	0.01
86	0.01	80	0.12	72	0.02
91	0.32	83	0.01	73	0.07
95	0.01	84	0.41	75	0.74
96	0.01	87	1.13	77	12.75
97	0.16	89	0.17	80	0.01
98	0.93	92	0.42	82	0.05
100	0.28	95	0.31	94	0.03
∑Mass(%)	88.6		86.44		54.06

Table 4.6 : Mode No. Vs Time Period

Mode No	Frequency (cycle/sec)	Period (sec)	Mode No	Frequency (cycle/sec)	Period (sec)	Mode No	Frequency (cycle/sec)	Period (sec)
1	16.76447	0.05965	9	23.83283	0.041959	2	20.60935	0.048522
4	21.47066	0.046575	11	24.46808	0.04087	5	22.42326	0.044597
10	24.4497	0.0409	13	25.34696	0.039452	11	24.46808	0.04087
12	25.08282	0.039868	15	26.54673	0.037669	13	25.34696	0.039452
14	25.97091	0.038505	17	27.3137	0.036612	17	27.3137	0.036612
16	26.86096	0.037229	19	27.97359	0.035748	19	27.97359	0.035748
18	27.47588	0.036396	21	28.63797	0.034919	21	28.63797	0.034919
22	30.92127	0.03234	23	30.94984	0.03231	23	30.94984	0.03231
25	31.82736	0.03142	24	31.79174	0.031455	24	31.79174	0.031455
27	32.2144	0.031042	28	32.61305	0.030663	26	32.17555	0.031079
31	32.92856	0.030369	30	32.70549	0.030576	28	32.61305	0.030663
33	33.3237	0.030009	49	36.27133	0.02757	30	32.70549	0.030576
35	33.61589	0.029748	50	36.56944	0.027345	32	33.12817	0.030186
37	33.95409	0.029452	52	36.83289	0.02715	34	33.58542	0.029775
39	34.04192	0.029376	54	37.28273	0.026822	36	33.84448	0.029547
40	34.08086	0.029342	55	37.51954	0.026653	38	34.02145	0.029393
45	35.40173	0.028247	56	37.56693	0.026619	44	34.97492	0.028592
51	36.6487	0.027286	60	38.38352	0.026053	47	35.87901	0.027871
62	39.34162	0.025418	61	38.43217	0.02602	50	36.56944	0.027345
65	40.35691	0.024779	63	40.25146	0.024844	52	36.83289	0.02715
67	41.19496	0.024275	64	40.33279	0.024794	54	37.28273	0.026822
68	41.36051	0.024178	69	41.46564	0.024116	55	37.51954	0.026653
74	42.79285	0.023368	71	41.98554	0.023818	56	37.56693	0.026619
76	43.26972	0.023111	72	42.072	0.023769	60	38.38352	0.026053
78	43.53997	0.022967	73	42.19606	0.023699	61	38.43217	0.02602
79	44.49284	0.022476	75	42.82593	0.02335	63	40.25146	0.024844
85	47.97643	0.020844	77	43.28842	0.023101	64	40.33279	0.024794
86	48.11007	0.020786	80	44.54691	0.022448	72	42.072	0.023769
91	49.57215	0.020173	83	46.90857	0.021318	73	42.19606	0.023699
95	50.27814	0.019889	84	47.01625	0.021269	75	42.82593	0.02335
96	50.84159	0.019669	87	48.45931	0.020636	77	43.28842	0.023101
97	51.1143	0.019564	89	49.10377	0.020365	80	44.54691	0.022448
98	51.40737	0.019452	92	49.6526	0.02014	82	45.93596	0.021769
99	51.61278	0.019375	94	49.93076	0.020028	83	46.90857	0.021318
100	51.80902	0.019302	95	50.27814	0.019889	94	49.93076	0.020028

In following three directions seismic actions are combined.

X + 0.3Y + 0.3Z

0.3X + Y + 0.3Z

0.3X + 0.3Y + Z

4.8 Interpretation of Results

Spectral analysis was used to assess the modal of the given structure, which modeled as a system of “multi degree of freedom”. This uses the concept of the response spectrum to estimate the behavior of the structure (Displacement, velocity, stress, etc.). In the specific case, the analysis of the vibration modes did emerge as the structure is characterized by low periods; its acceleration is next to the acceleration of the ground while the maximum displacement is very small. This is therefore a very rigid system which moves rigidly with the ground. The masses participants and periods derived playing an Eigen value Analysis considering the number of frequencies of 100, are summarized below.Keeping in mind the above results, the structure is placed in response spectrum as shown in figure.

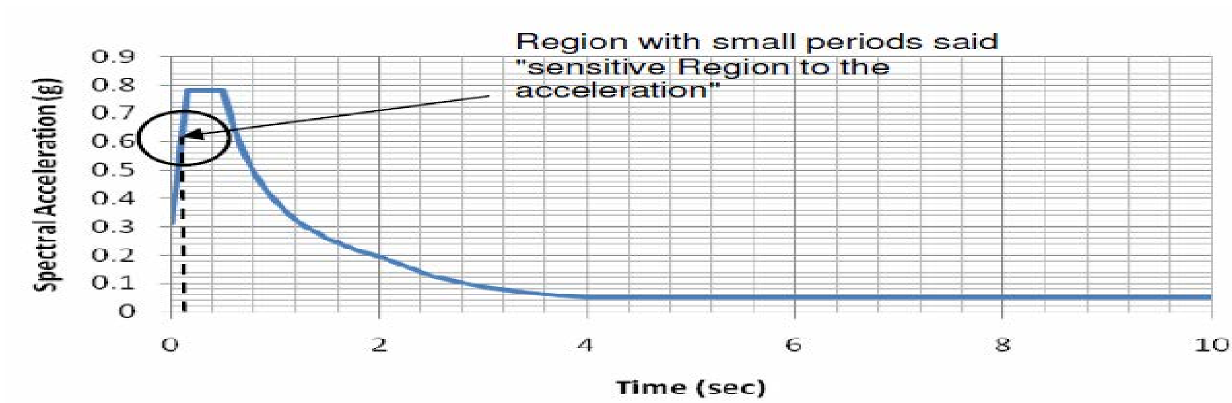

Figure 4.13: Location of PGA at collapse

The confirmation of the high rigidity of the structure is discussed in next chapter.

4.9 Methods of analysis

Pushover analysis, dynamic time history analysis and response spectrum analysis are followed, utilizing MIDAS and SAP2000 in order to acquiring strength capacity of the structure. In the specific time domain, dynamic time-history analysis is considered as the sequential solution of multi degree of freedom equation, a reliable tool for structural seismic response. Evaluation of anticipated seismic performance of structures is attained by the records of motion though significant computational efforts are required for it. In order to estimate the strength capacity in the post-elastic range and dig out the potential weak areas in the structure, inelastic static pushover analysis is considered as an alternative. The process implements predefined lateral load pattern distributed all along the bridge height. Awaiting the failure of the structure (Mwafy and Elnashai, 2001), lateral forces and displacement control are increased with the constant proportionality. The assumption of the pushover analysis response is equivalent to the single degree of freedom system implying that the response is dependent on the single mode and its shape remains constant throughout the time history response (Krawinkler and Seneviratna, 1998).

Modal analysis is performed before pushover or dynamic analysis, yielding the mode shapes and natural frequencies being used for the selection of base accelerogram in dynamic collapse analysis and determination of lateral load distribution in the pushover analysis.

Rayleigh damping model was used in the analysis and the damping matrix is given by

$$C = \alpha M + \beta K$$

Damping is considered proportional to the initial stiffness (K) as damping of higher modes is unpredictable. The value of α is taken as zero and β is calculated by the program to give 5% critical damping in the first mode of the structure. Lateral loads are functional in a single comportment at the master nodes and augmented until the collapse of the structure in the pushover analysis. In vertical direction incorporated with the option of MIDAS Civil 2013, is considered according to the AASHTO-2012.

The loading shape for pushover analysis is calculated by keeping in view the fundamental period of modal analysis. A small step of predefined lateral load is of 0.01s to ensure the insignificant inertia forces. The time step is verified to a check with convergence of results. One value of maximum displacement and maximum base shear cancellation be generated on the single value of input ground accelerogram generated from an earthquake. Series of maximum responses generated from the analysis and a scaled simulate varying intensity of the ground motions are based to determine the capacity of the structures. In dynamic collapse analysis iteration time step is of 0.005s, verifying by progress of several Analysis with different time steps, to test out convergence of the results.

Chapter 5

Evaluations of Seismic Assessment

5.1 FEA Results and Interpretations

FEA modeling and the techniques to assess seismic adequacy of Bridge will be discussed in this chapter. The following figure shows that the structure is strong enough to resist the design earthquake. The results obtained from Finite Element Modeling are shown from the Figures 5.1 to 5.8.

5.2 Interpretation of the Results

Response spectrum method is followed in modeling the bridge with degree freedom. In certain situations the structure exemplifies by low periods by means of acceleration of the ground with very low maximum displacement, the time history analysis has been performed therefore treating a rigid system sustained with the ground. The Figure 5.2 shows the mode shapes from Eigen value Analysis and frequencies acquired.

To get hold of the natural frequencies and the mode shape, earlier than the conduction of foremost analysis the modal analysis was accomplished. The prophecy acquired from the modals analysis of 0.00596s, 0.00485s and 0.00472s, generated1st, 2nd and 3rd modal natural periods respectively for bridge. The literature designates these values nearer to the actual ones. The dynamic property of the model is embodying by FEA model.

Results presented in figure confirm the structural rigidity of the bridge. Figure 5.3 having maximum value of 3.5 mm in severe condition shows the deflection in longitudinal direction.

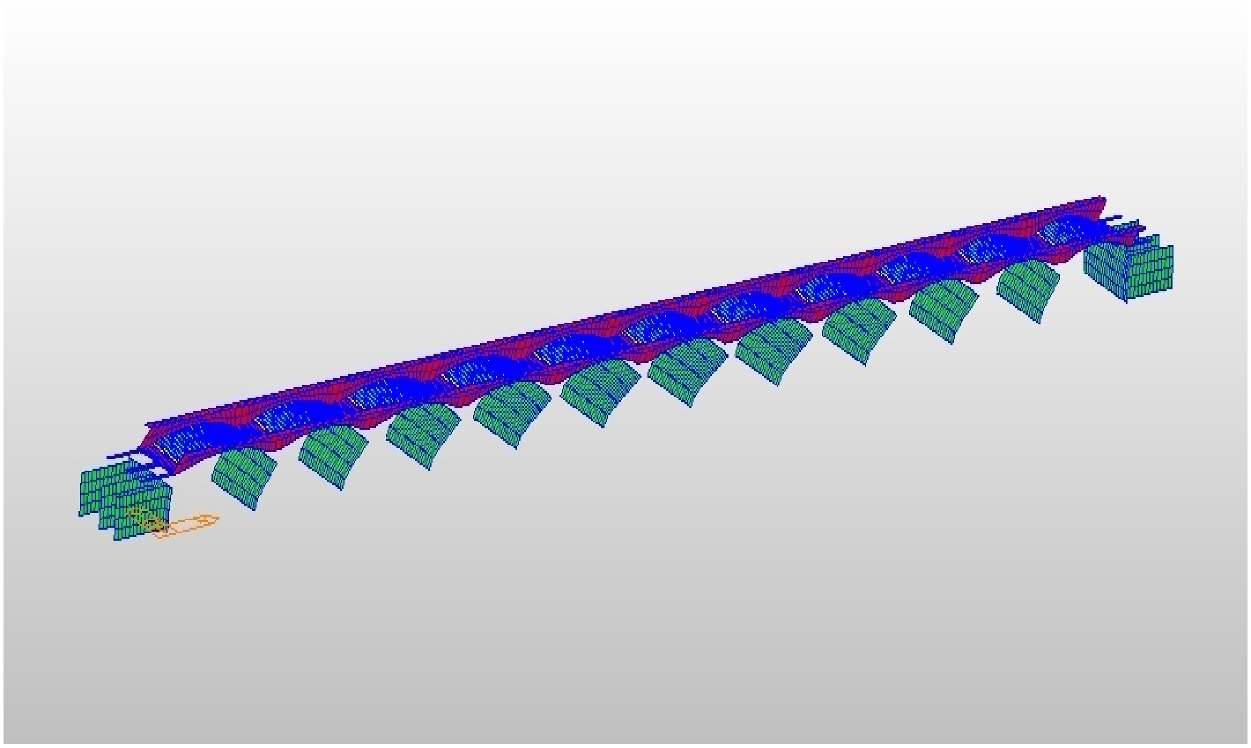

Figure 5.1: Deflected Shape

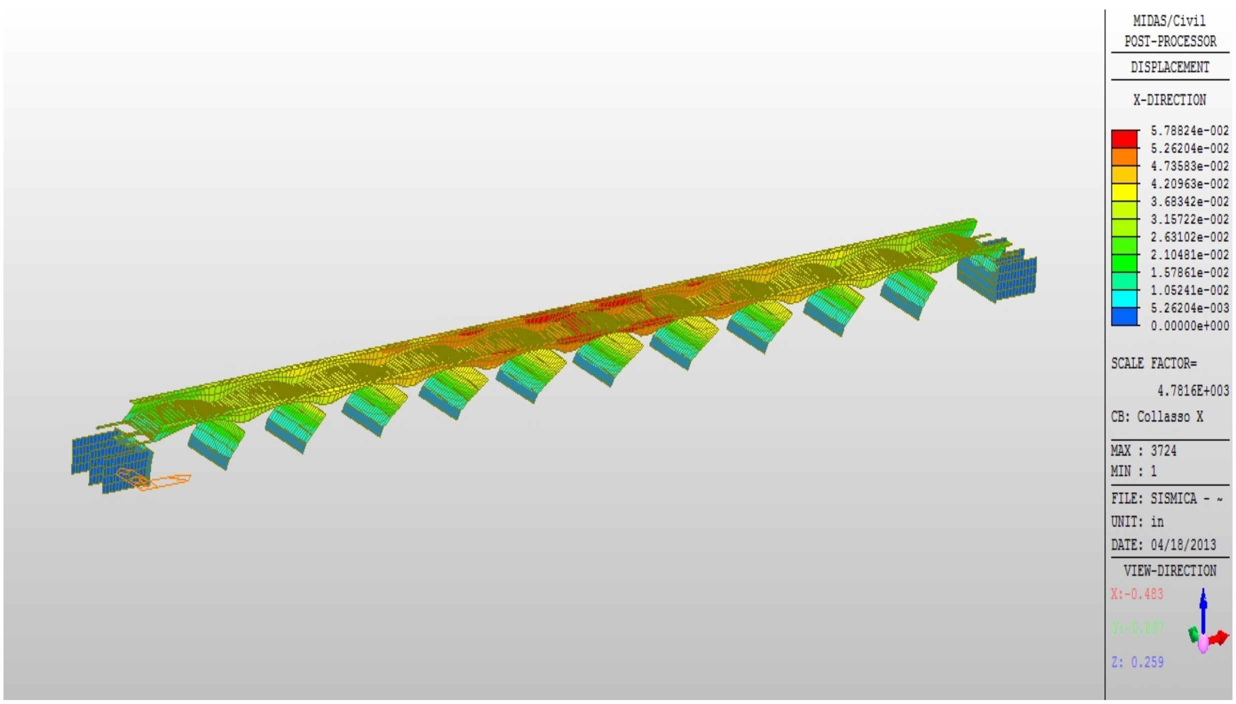

Figure 5.2: Displacement Contours

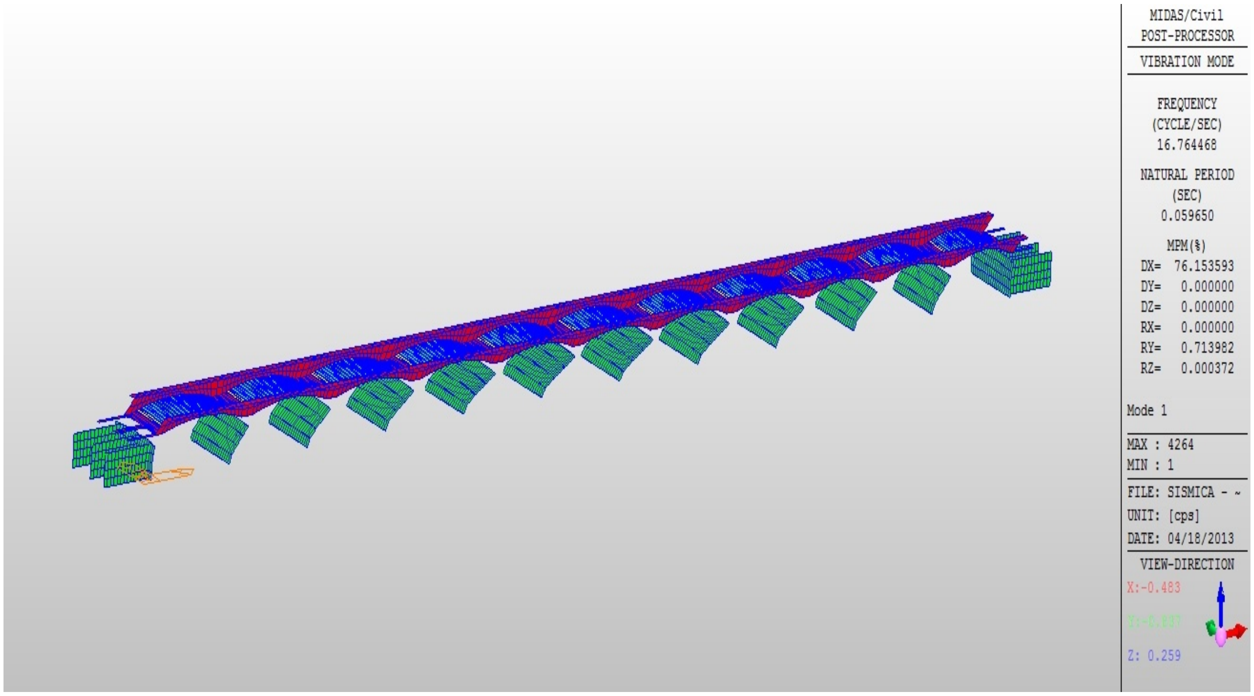

Figure 5.3: Vibration Mode Shape 1 (Natural Time Period = 0.0059 Sec)

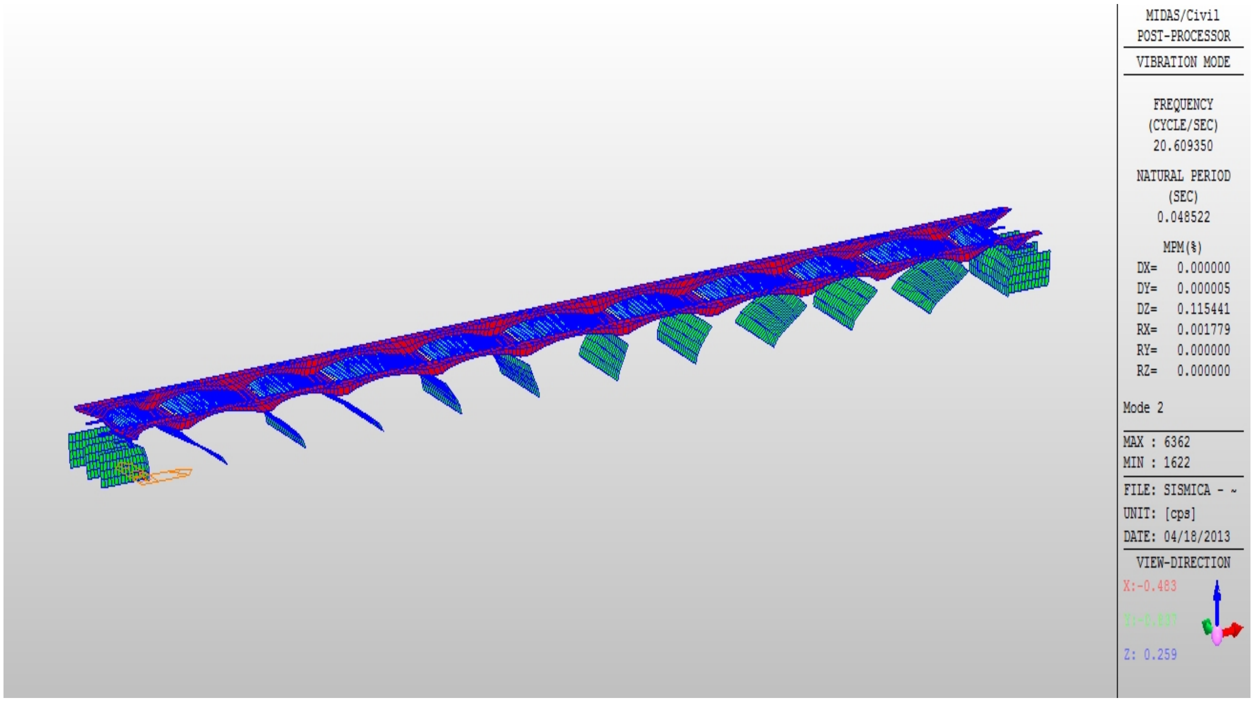

Figure 5.4: Vibration Mode 2 (Natural Time Period = 0.0048 Sec)

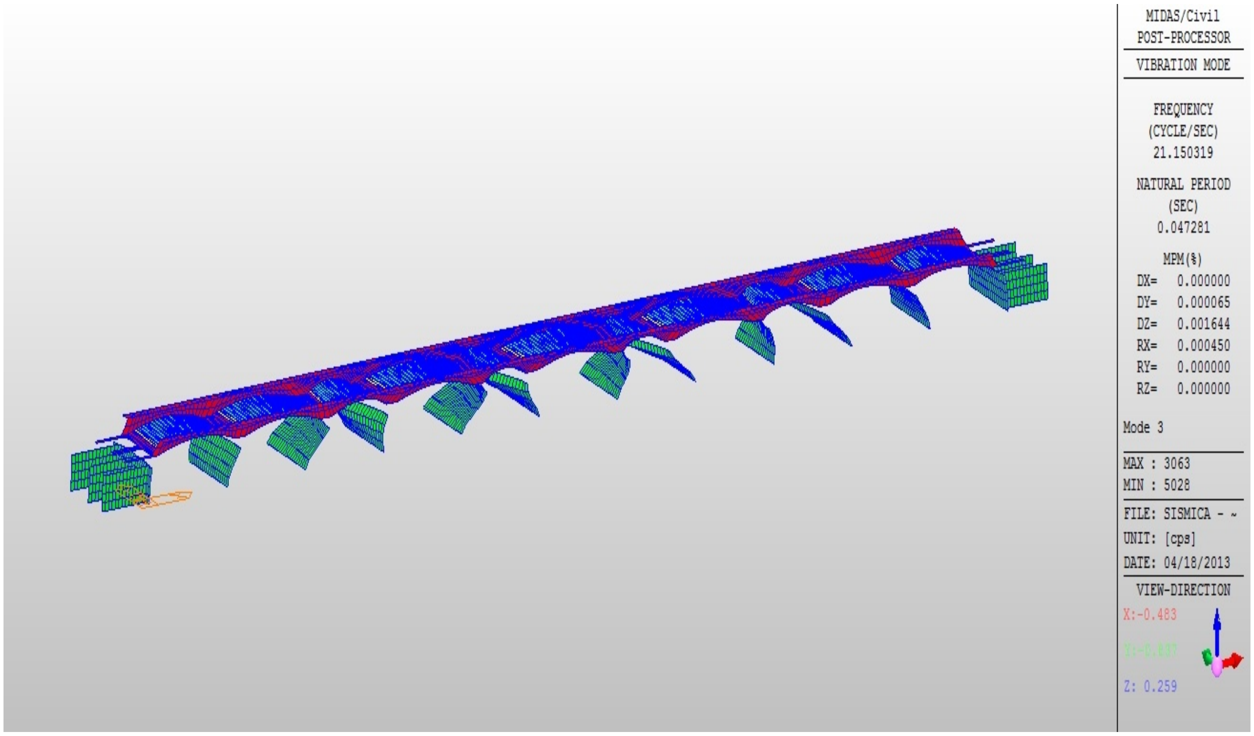

Figure 5.5: Vibration Mode Shape 3 (Natural Time Period = 0.0047 Sec)

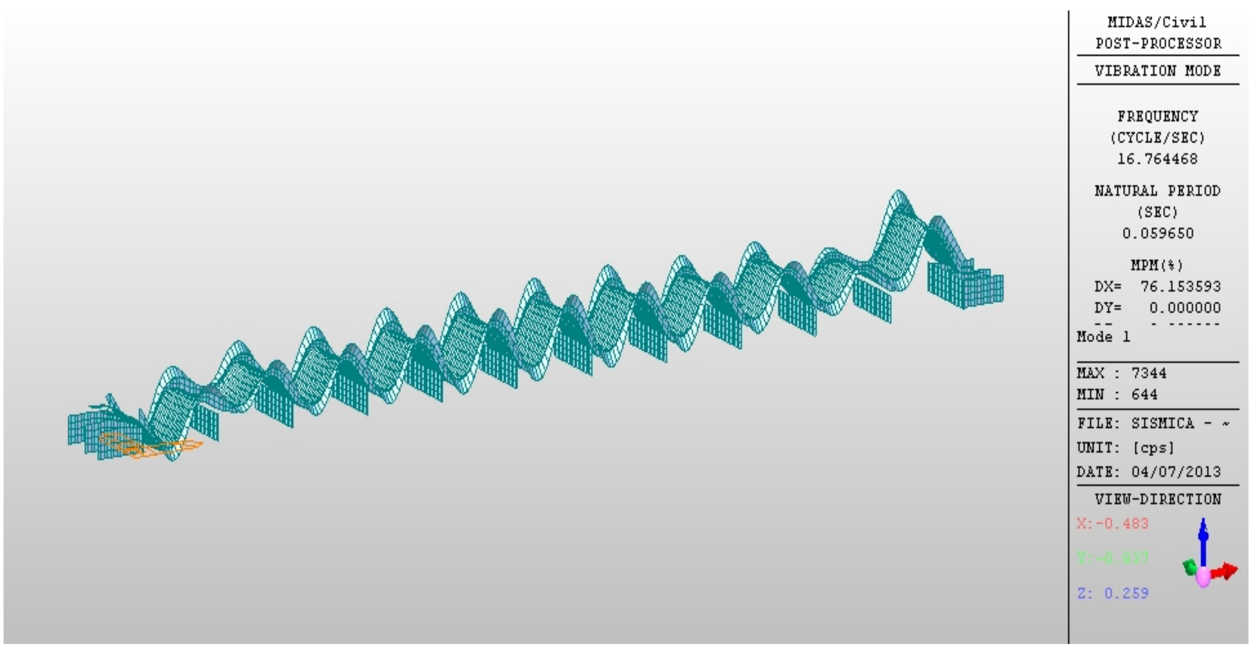

Figure 5.6: Vibration Mode shape 1 for Mz

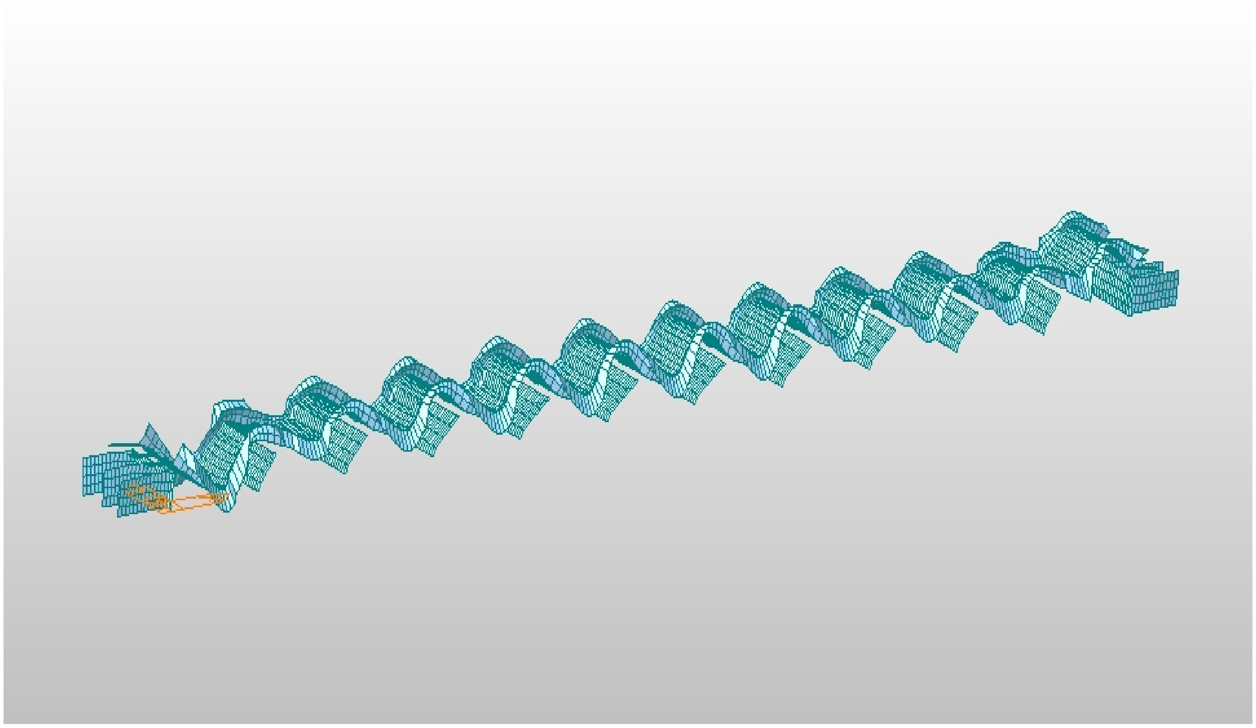

Figure 5.7: Vibration Mode shape 1 for all three directions

5.3 Arch Verification

Conduct of arch is deliberated in premier phase. The two sections of in Centre and near support are considered for verification of longitudinal section.

Arch along with wall make up the considered resistance. It’s being observed that careful evaluation of the constructive sketches, in case of seismic actions, wall doesn't undergo to endanger its functionality as it also support a part of the sidewalk.

The capacity and demand has been calculated for the sake of resistant PGA. Dividing capacity PGA with demand PGA gives the safety coefficient FS. The said procedure is being followed in the longitudinal section of arch. The figure 5.8 shows the shape of the longitudinal moment along the arch in combination with the prevailing longitudinal earthquakes.

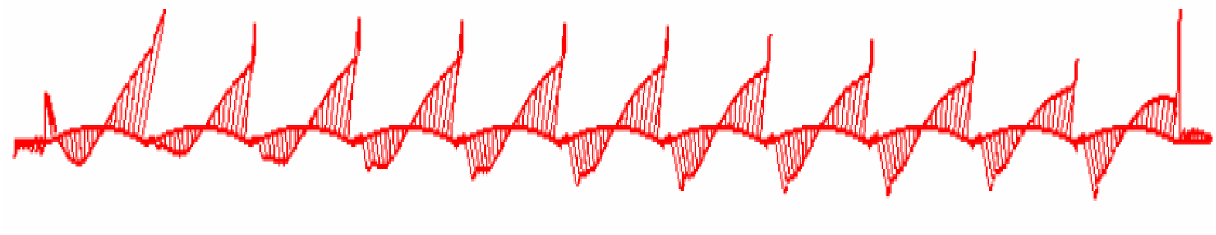

Figure 5.8: Bending Moment Diagram for all spans of Bridge Model

The figure 5.9 shows the shape of the detail of only one span.

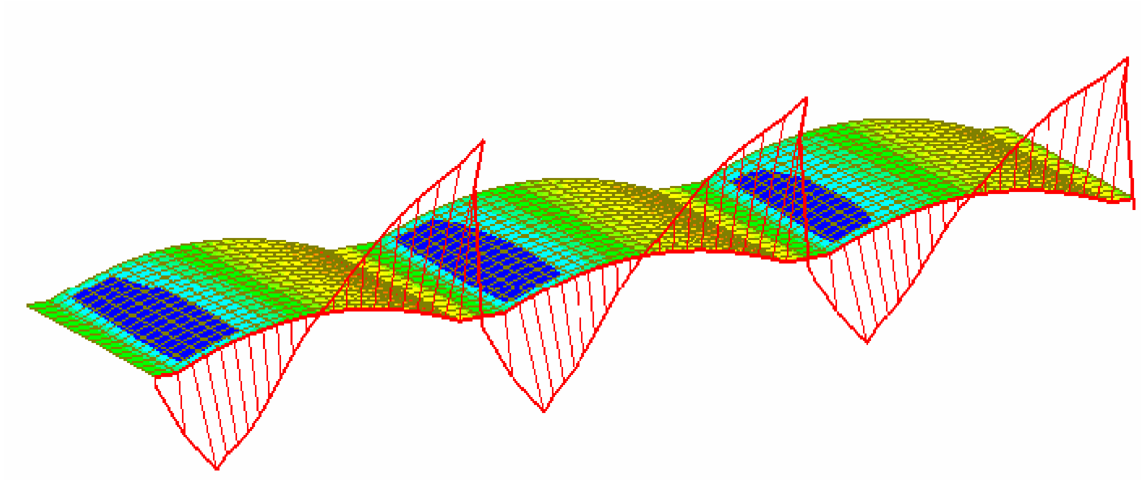

Figure 5.9: Bending Moment Diagram for mid span of Bridge

5.4 Safety verification of arch at Centre of bridge

The section concludes discussion on the values of the resistant PGAs to the state level (SL) of collapse (CO).SL (DS) and SL (DL) are calculated and peak to ground values of acceleration obtained from certain seismic zones are multiplied for the safety coefficient

Table 5.1: Table Safety coefficient obtained from the worst combination for mid span arch

Coefficient Mx/My=Constant		Coefficeint N=Constant	
Coefficient γ	2.957	Coefficient	4.356
Mx[K.ft]	-5575970	Mx[K.ft]	-5575970
N[Kip]	5941991.5	N[Kip]	3781286.5
Mux[K.ft]	-16493080	Mux[K.ft]	24292911.5
Nu[Kip]	175756981	Nu[Kip]	16271659.2

Table 5.2: PGA capacity of the mid span Arch

Bending-State Level of Collapse (SLCO) Longitudinal	**PGA_{cap} [g]** γ_{min} × SL_{co} PGA_{soil} 2%	**PGA cap [g]** **1.943**
Bending-State Level of Severe Damage (SLDS) Longitudinal	γ_{min} × SL_{ds} PGA_{soil} 10% (0.438)	**PGA cap [g]** **1.295**
Bending-State Level of Limited Damage (SLDL) Longitudinal	γ_{min} × SL_{dl} PGA_{soil} 50% (0.175)	**PGA cap [g]** **0.517**

Figures 5.10 to 5.12 shows the three state limits and graph of the resistant PGA to the SL (CO), SL (DS) and SL (DL).

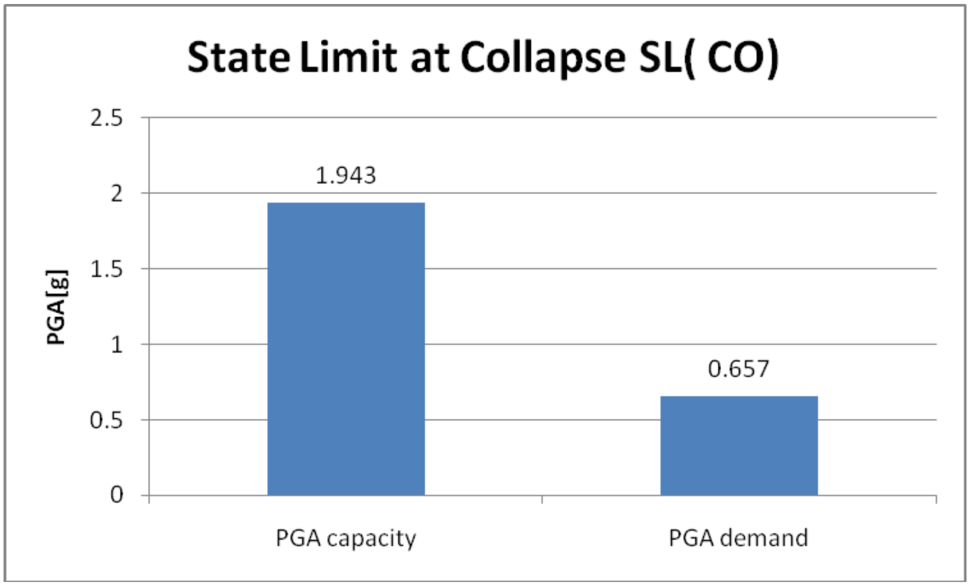

Figure 5.10: Comparison of actual PGA capacity and required PGA at Collapse

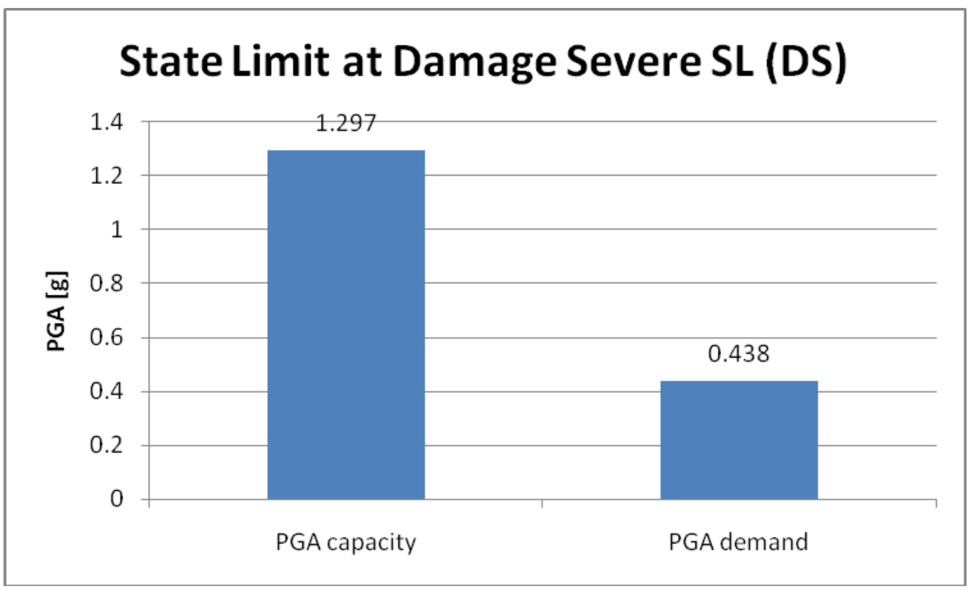

Figure 5.11: Comparison of actual PGA capacity and required PGA at Severe Damage

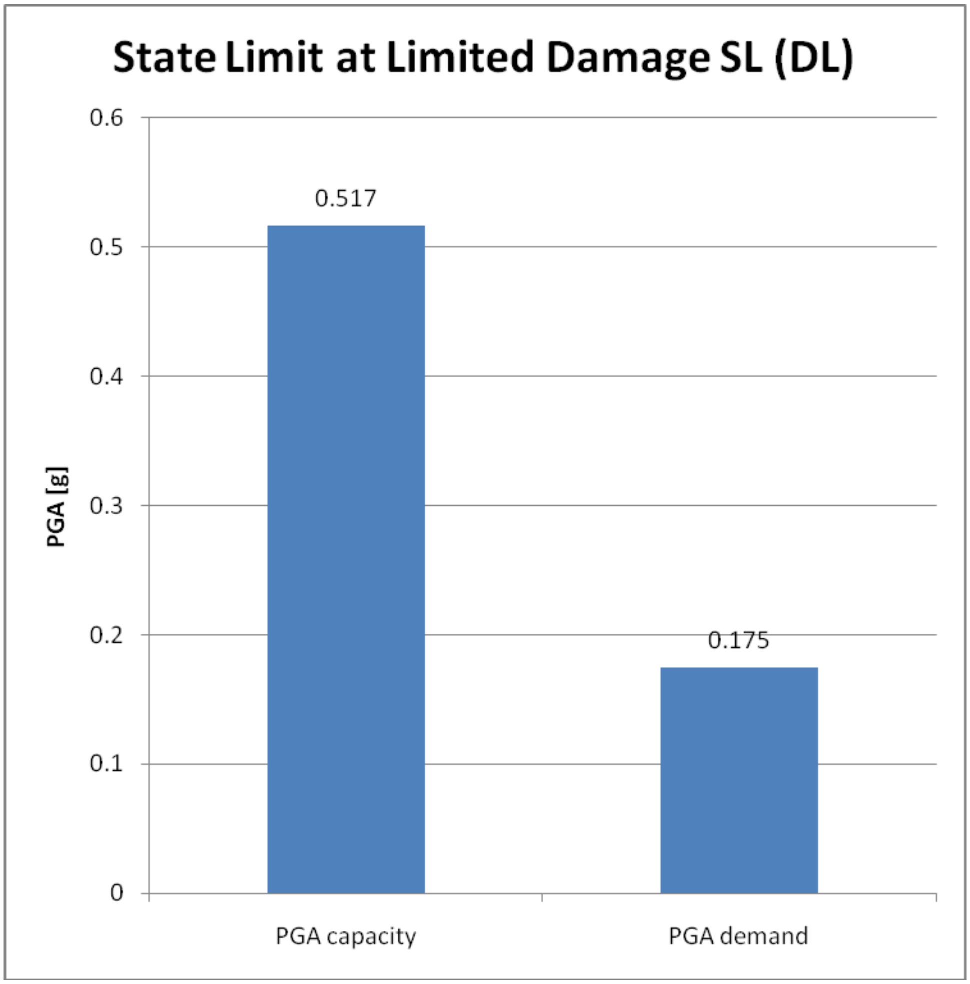

Figure 5.12: Comparison of actual PGA capacity and required PGA at Limited Damage

5.5 Safety verification of arch near support of bridge

Table 5.4 presents values of to the SL (CO), SL (DS) and SL (DL) related to the sate Limits. Table 5.3 Shows Safety coefficients obtained from the worst combination at near support.

Table 5.3: Safety coefficient obtained from the worst combination at near support

Coefficient Mx/My=Constant		Coefficient N=Constant	
Coefficient γ	2.109	Coefficient	2.759
Mx[K.ft]	-29812261	Mx[K.ft]	-29812261
N[Kip]	7616527	N[Kip]	7616527
Mux[K.ft]	-14137945	Mux[K.ft]	82242788.5
Nu[Kip]	21349475	Nu[Kip]	210116369

Table 5.4: PGA capacity at support span arch

<table>
<tr><td rowspan="2">Bending-State Level of Collapse (SLCO) Longitudinal</td><td rowspan="2">PGA_{cap} [g]
γ_{min} × SL_{co} PGA_{soil} 2%</td><td>PGA cap [g]</td></tr>
<tr><td>1.385</td></tr>
<tr><td rowspan="2">Bending-State Level of Severe Damage (SLDS) Longitudinal</td><td rowspan="2">γ_{min} × SL_{ds} PGA_{soil} 10% (0.438)</td><td>PGA cap [g]</td></tr>
<tr><td>0.923</td></tr>
<tr><td rowspan="2">Bending-State Level of Limited Damage (SLDL)Longitudinal</td><td rowspan="2">γ_{min} × SL_{dl} PGA_{soil} 50% (0.175)</td><td>PGA cap [g]</td></tr>
<tr><td>0.369</td></tr>
</table>

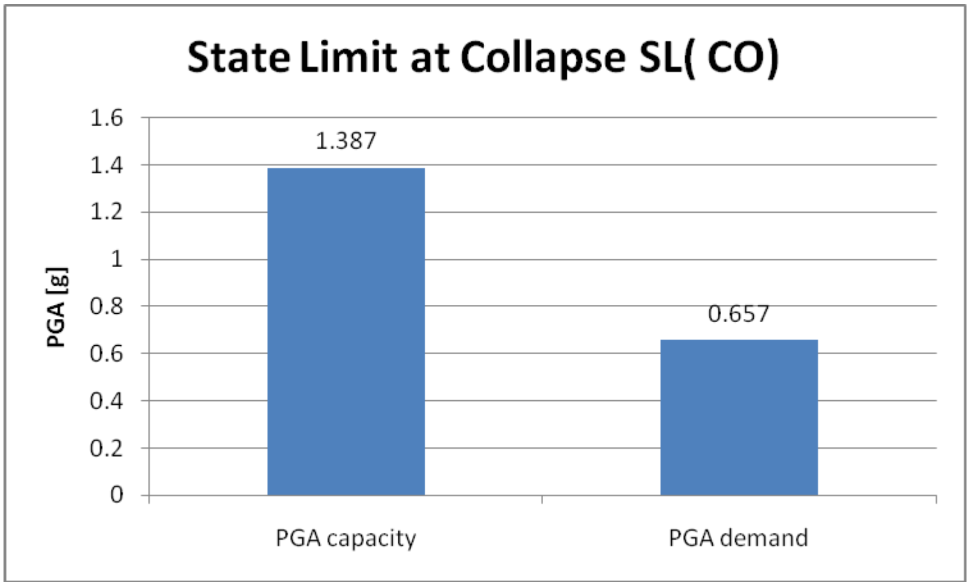

Figure 5.13: Comparison of actual PGA capacity and required PGA at Collapse near support

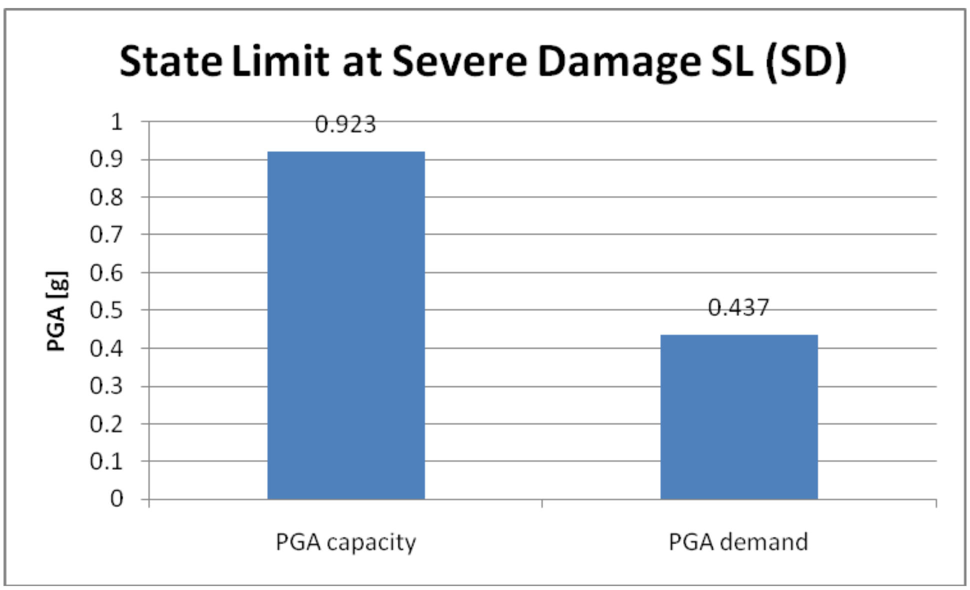

Figure 5.14: Comparison of actual PGA capacity and required PGA at Severe Damage near support

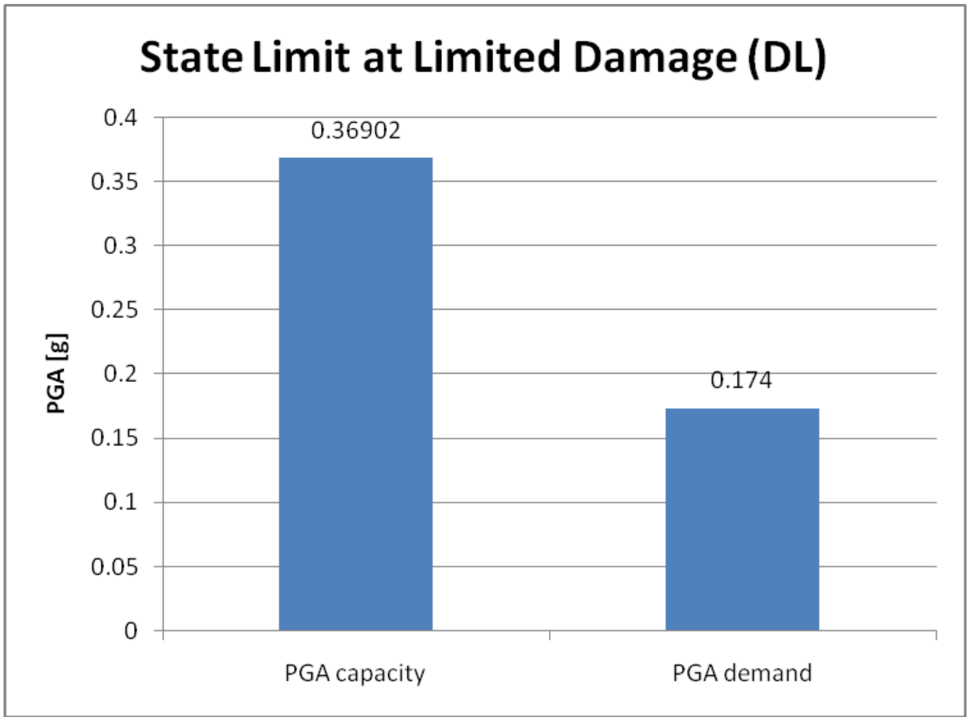

Figure 5.15: Comparison of actual PGA capacity and required PGA at Limited Damage near support

Above figures shows that under the sever condition of state limit at collapse (SL CO), severe damage (SL SD) and limited damage (SL LD),the arch is sufficient capacity to resist the seismic action.

5.6 Safety Verification of Piles

The piles will be checked in at bending. In a manner similar to what was seen for the arch, shows the PGA resistant and safety coefficients. The trend of the bending moment is indicated in the figure

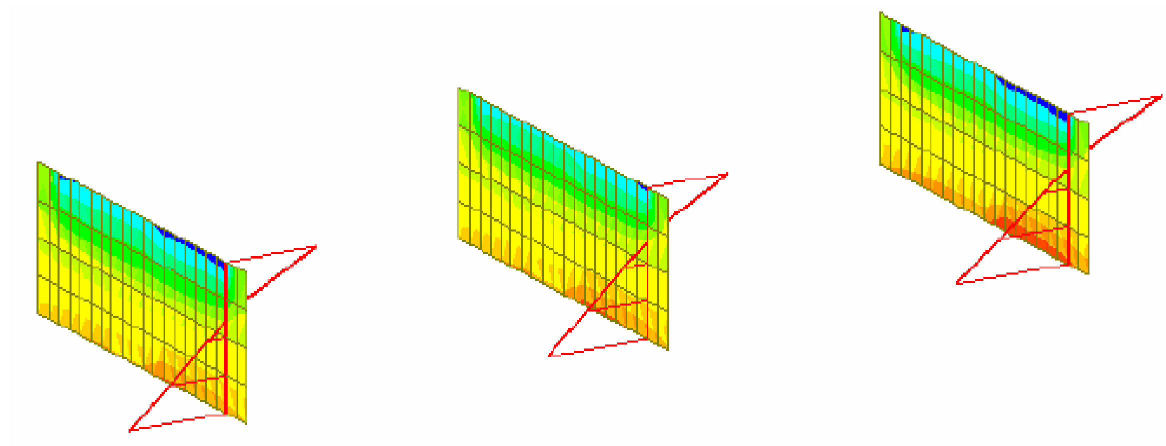

Figure 5.16: Bending moment of Piles

Verification of the ultimate limit state in the worst combination – buckling

Table 5.5: Safety Coefficient obtained for Piles

Coefficient Mx/My=Constant		Coefficient N=Constant	
Coefficient γ	0.678	Coefficient	0.733
Mx[K.ft]	-14150000	Mx[K.ft]	-14150000
My[K.ft]	0	My[K.ft]	0
N[Kip]	-55400	N[Kip]	-55400
Mux[K.ft]	-9600350	Mux[K.ft]	-10382040
Mux[K.ft]	0	Muy[K.ft]	0
Nu[Kip]	-37587	Nu[Kip]	-55400

The values of PGA-resistant SL CO relating to the stack are shown below. They are obtained by multiplying the peak ground acceleration, obtained from the seismic hazard map for the relative safety factor FS.

Table 5.6: PGA capacity for Piles

Bending-State Level of Collapse (SLCO) Longitudinal	**PGA_{cap} [g]** γ_{min} × SL_{co} PGA_{soil} 2%	**PGA cap [g]**
		0.445
Bending-State Level of Severe Damage (SLDS) Longitudinal	γ_{min} × SL_{ds} PGA_{soil} 10% (0.438)	**PGA cap [g]**
		0.487
Bending-State Level of Limited Damage (SLDL) Longitudinal	γ_{min} × SL_{dl} PGA_{soil} 50% (0.175)	**PGA cap [g]**
		0.779

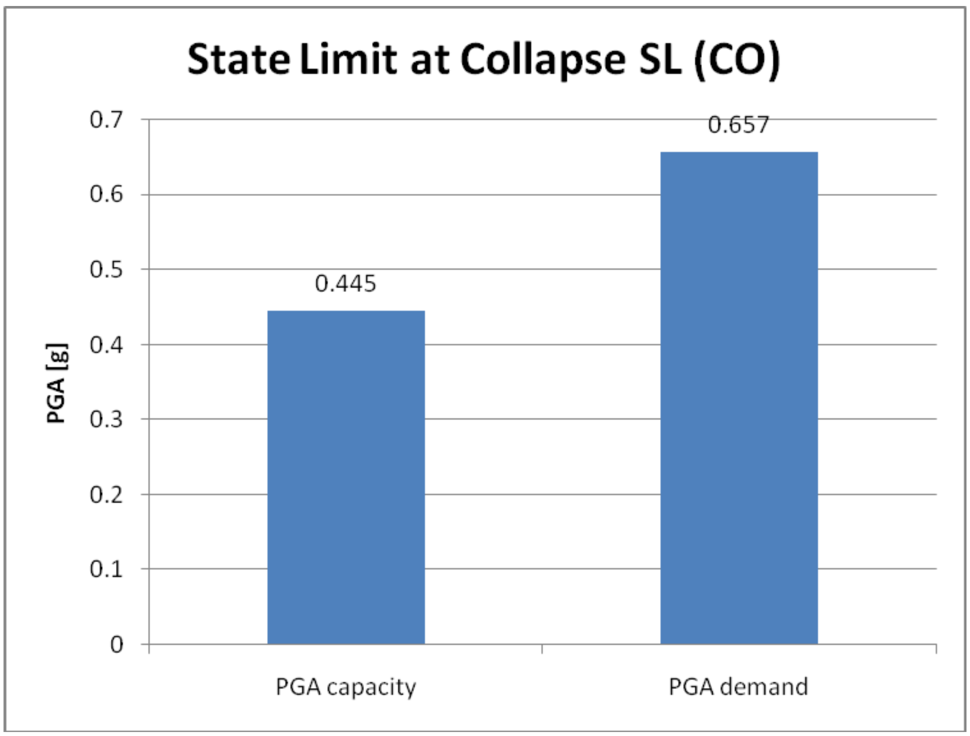

Figure 5.17: Comparison of actual PGA capacity and required PGA at Collapse for piles

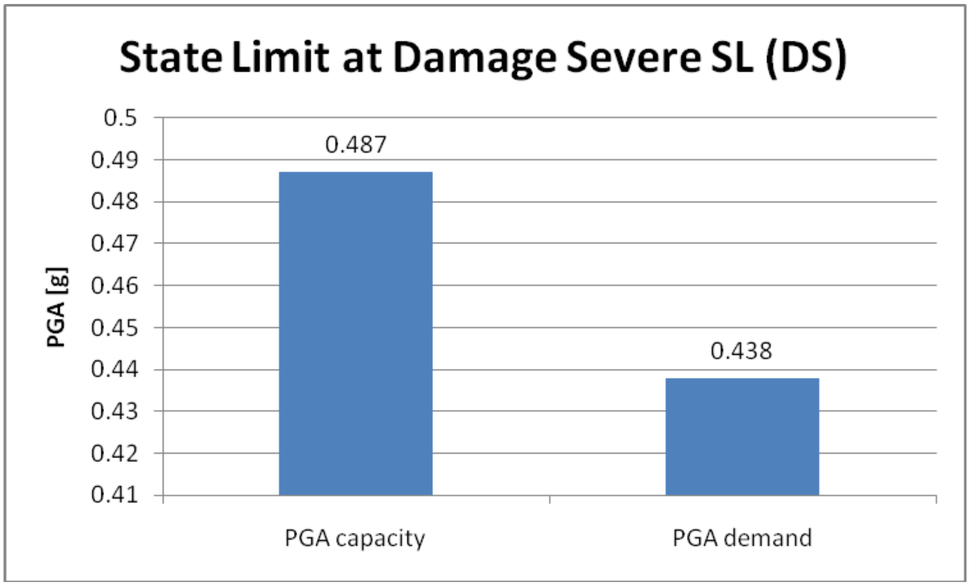

Figure 5.18: Comparison of actual PGA capacity and required PGA at Severe Damage of Piles

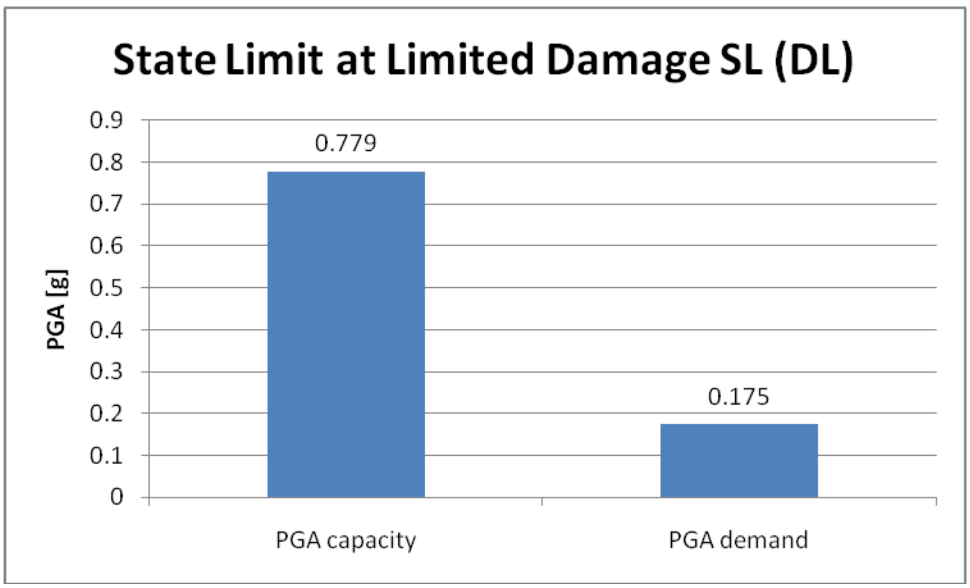

Figure 5.19: Comparison of actual PGA capacity and required PGA at Limited Damage of Piles

5.7 Checking The Shoulder

The finite element model developed to calculate the stresses is shown in Figure

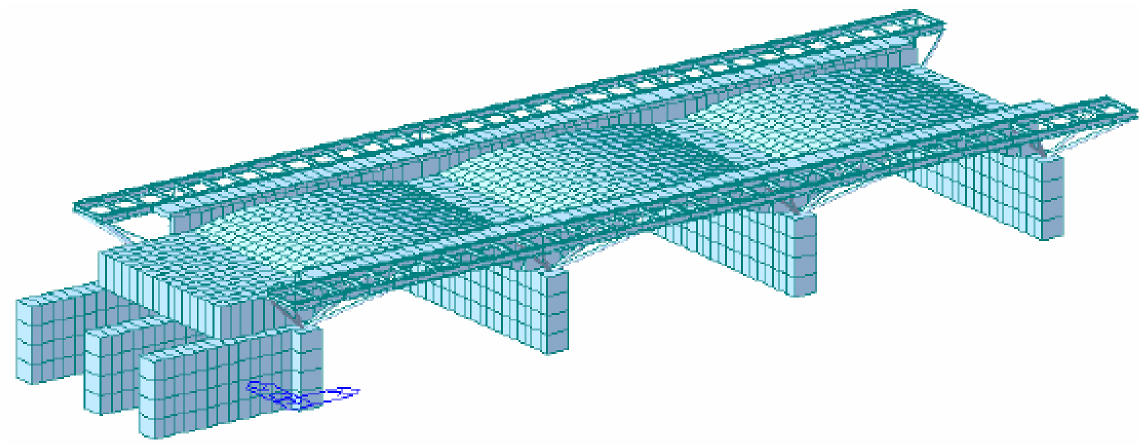

Figure 5.20: Shoulder Model

Shoulders were considered only three bays as the effects of the loads on the other are not appreciably noticeable. In addition to the above mentioned applied in the previous section for the calculation of arches and piles, has been added to the pressures on the various elements of the lands where they go to press.

The piles as well as the front wall of the shoulder are stuck to the ground, while for the spurs was adopted the model of Winkler, thus applying a linear relationship between load applied at a point and its failure, independent loads (bed of springs independent). As for the piles, it will be verification to buckling and cracking. Given the type of structure, we have omitted the tests overturning and sliding of the shoulder. In the picture you can see the stripes elementary treated as continuous beams in buckling checks.

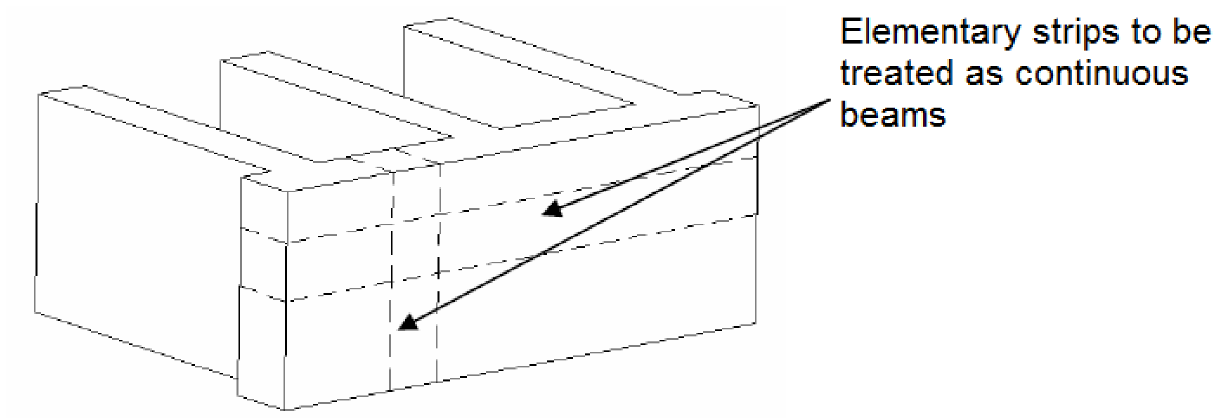

Figure 5.21: Elementary strips to be treated as continuous beams

Considering the structural type examined, it was decided to treat the shoulders as if they were the piles, adding the horizontal thrust of the land.

Table 5.7: Safety Coefficient obtained for Shoulders

Coefficient Mx/My=Constant		Coefficient N=Constant	
Coefficient γ	0.803	Coefficient	0.683
Mx[K.ft]	-6150000	Mx[K.ft]	-6150000
My[K.ft]	0	My[K.ft]	0
N[Kip]	80700	N[Kip]	80700
Mux[K.ft]	-4941992	Mux[K.ft]	-4202272
Mux[K.ft]	0	Muy[K.ft]	0
Nu[Kip]	64848	Nu[Kip]	80700

Table 5.8: PGA capacity for Shoulders

Bending-State Level of Collapse(SLCO) Longitudinal	**PGA_{cap} [g]**	**PGA cap [g]**
	γ_{min} × SL_{co} PGA_{soil} 2% (0.657)	**0.447**
Bending-State Level of Severe Damage (SLDS) Longitudinal	γ_{min} × SL_{ds} PGA_{soil} 10% (0.438)	**PGA cap [g]**
		0.515
Bending-State Level of Limited Damage (SLDL) Longitudinal	γ_{min} × SL_{dl} PGA_{soil} 50% (0.175)	**PGA cap [g]**
		0.544

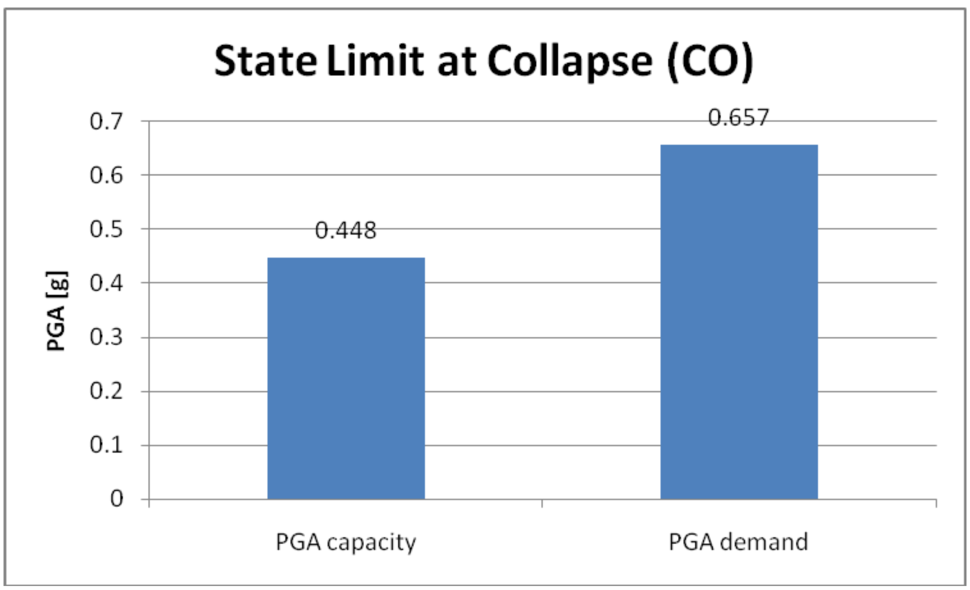

Figure 5.22: Comparison of actual PGA capacity and required PGA at Collapse for shoulders

5.8 Nonlinear Static Analysis

Gradual aggregate in horizontal load under constant gravity loading is observed in applying Nonlinear static analysis (NSA) i.e. pushover analysis to approximate seismic anxieties for structures. If directive to consent comparison to a spectral demand, the subsequent base shear –displacement pushover curve can be assimilated into spectral acceleration and displacements. The perception followed is based on the conjecture of equivalent response figures of the response of the structure and the response of an equivalent SDOF system. Krawinkler (Krawinkler & Seneviratna, 1998) introduced strong simplifications in the analysis with the emphasis that the shape of the mode remains constant as structure´s response is characterized by a single mode. One mode structure the analysis has proven to generate acceptable results and various patterns can be followed for enhanced evaluations.

Kalkan (Kalkan&Kunnath, 2007) or Chopra and Goel (Chopra &Goel, 2002) have drawn similar conclusions in comparison with others regarding combining a variety of load patterns. All modes of vibration alongside momentous impact to the seismic demand (normally the first two-three modes) were encompassed in their later development. Later on Chopra, Goeland Chintanapakdee (Chopra, et al., s.d.) augmented the modal pushover analysis in which, while computing the response contributions of higher modes, the structure is presumed to be linearly elastic. The negligence of the higher modes and worth of the inelastic response under intense excitation in the first mode was the prior suggestion by the researchers.

When the structure is made-up to undergo unadorned non-elastic displacements, concerning the application purposes, NSA is particularly convenient for demand predictions at low performance levels hereafter contributing towards the security assessment and collapse prevention (Krawinkler&Seneviratna, 1998). It is pertinent to be mentioned that the qualms familiarized by the model, the applied load pattern and the demand spectra are kept in account NSA. The outcomes considerably as explained by (Pelà, et al., 2009), (Pelà, et al., 2012), and (Leprotti, et al., 2010) are affected by the arbitrary control node selection. The conclusions expressed in terms of an energy-displacement curve and their contrast with energy demand spectra as described by Mezzi (Mezzi, et al., 2006) are considered to overcome the respective affects.

The standard techniques elucidated in Eurocode8 (UNI ENV 1998-1 (Eurocode8), December 2004) or FEMA 440 (FEMA 440, 2005) analysis should be conceded smearing not less than two vertical disseminations of the lateral loads. The two lateral forces distribution are calculated in the linear analysis, based on proportional to mass and one modal pattern rendering to AASHTO LRFD-2012.As explained in FEMA 440 (FEMA 440, 2005) another load patterns embrace concentrated load, triangular, first mode, code distribution or adaptive load.

The technique entailed in smearing the self-load in a first step (actually it was applied in two steps each attaching 50% of the self-load, but one would yield the same result) and consequently toting incrementally horizontal forces proportionate to the mass distribution in x direction (= transversal direction). Irrespective of the commendation in (Krawinkler & Seneviratna, 1998), (UNIENV 1998-1 (Eurocode8), December 2004), (FEMA 440, 2005) alternative load pattern has not been applied. Precise approximations for the global and local inelastic deformation structures vibrating in one crucial mode will be shown relating to the inferences of Krawinkler (Krawinkler&Seneviratna, 1998). The prime mass participation in transversal direction of 76.05% in the first mode should be perceived in the HAJI AYUB Bridge. The employed load pattern in this direction may occur due to subsequent failure mechanism that may take place for forces in transversal direction. The third mode exhibits 25.04% and mode exhibits 41.56% in longitudinal and transversal direction respectively .The mass participation in the longitude final direction, considered vitally in seismic response of the bridge evaluation, can be neglected as the present study only in relates with the response in transversal direction.

The point should be enlighten that the requirements of the AASHTO LRFD-2012 (CS.LL.PP. 2012) within the scope of this study are abandoned, and the modal is load pattern is kept under consideration, comprising an effective modal mass participation of at least 85%. Through building with high mass contribution by the slabs but literally no mass contribution from the elements in between a modal mass participation of 85% might be achieved with the dominate mode for a structure. Constructions with depressed mass distribution can barely approach the 85% of effective modal mass. For the highest modal mass participation for the predominant (1st) mode in the HAJI AYUB bridge in transversal direction (x-direction) accounts to 76%.

Geometrical and material non-linearity is kept in view for the numerical accomplishment of the NSA. The mass distribution is multiplied by applied steps (varied from 0,1 to 1e-10).After the reduction load steps to 1e-10, analysis is stopped in case of inconvenient to way out. Loading comes up with the displacement control. The model with 8 respectively 6-node elements comprises 150 maximum iterations.

5.9 NSA Results and Discussion

Comparison of the pushover curves for arbitrarily chosen control nodes is accomplished for determination of different constitutive models on the bridges seismic performance indirectly assessed by the nonlinear static analysis. Direct displacement-acceleration curves is preferred rather than typical displacement-base shear curves in turn assisting assessment with response spectra and nonlinear dynamic exploration consequences. Node 5774 is situated at the façade on the top of the parapet between the pier and the cornice of the middle arch and the nodes 5447 and 5605 are located at the façade at the strut close to the middle arch, very close to the center of mass of the structure. Node 5774 not encompassed in the inferences describes a local response of the parapet.

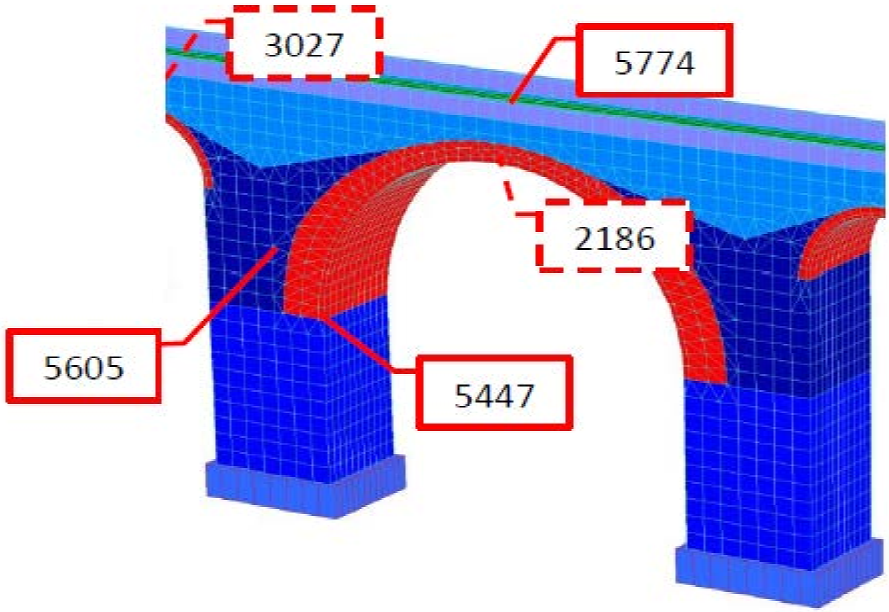

Figure 5.23: Control Node Position - Solid line around the number indicates node position on the facade, dashed line indicates a position in the middle of the arch (longitudinal axis of the bridge)

The highest displacements at the last load-step applied is illustrated by node 5774 in a sequence along with node 2186, 3027, 5605 and 5447.The displacement shown by the node 5774at the last load-step which are three times higher than node 5447.Approximate 2.5 is beard by node 2186 regarding to node 5447 while the node 5605 is still showing approximate 50% higher result than node 5447.The said analysis is followed in all analysis.

Chapter 6

Results and Recommendations

6.1. Results and Conclusions

The bridges designed in Pakistan without seismic provisions undergo from earthquake are kept in focus for evaluating seismic vulnerability of bridge structures. In order to conclude the capacity of full scale bridge and for bridge under seismic loading, FEA analytical model and 3D nonlinear macroscopic FEA model using MIDAS were endorsed respectively, showing the predictions of the model with dynamic properties and reasonable accuracy.

In Pakistan, Comparison of demand and capacity curves is considered for evaluating the seismic vulnerability of historical bridge. The pits scenario (design earthquake) the seismic demand for historical bridge in Pakistan was conceded based on the statistics of sturdy replicated earthquake.

FEA Model was utilized for the estimation of bridge capacity. The study had evaluated that worst possible earthquake leads to the damages of these historical bridges. Retrofitting of the whole bridge needed to be done. Special attention must be given to piles and shoulders, as the present study shows that they are more vulnerable to earthquake. Further precedents are based on the predictions.

RECOMMENDATION

The study herein is primarily based on numerical criteria. To gratify the performance-base criteria, rational numerical models for seismic retrofitting with actual soil investigation at the subsoil of the bridge and experimental investigation may be carried out.

Following are the suggestions for further investigation:

1) Local Failure can be replicated by the alteration of Finite element model, defining interface between different structural elements of the bridge particularly the materials with low (filling) and high (spandrel walls, vaults) cohesion.

2) Working out complete set of code corresponding real ground motions records.

3) To determine the upper and lower range of the material parameters, computation of elastic parameters for each material and safety evaluation through detailed study.

4) Meticulous evaluation of the (MIDAS) material models in relation to shear, their feasibility for masonry and consequence for the safety valuation.

5) To analyze the behavior in the post-peak range, decreased load-steps and displacement controlled iterations is applied through a definite “pushover”.

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