TRIBHUWAN UNIVERSITY

INSTITUTE OF ENGINEERING

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Department of Civil Engineering

M.Sc Program in Structural Engineering

A

Final Thesis

On

"STRENGTH AND PERFORMANCE EVALUATION OF NEPALESE RC BRIDGE PIER USING NONLINEAR DYANAMIC RESPONSE ANALYSIS"

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064/MSSR/109

M.Sc. Structural Engineering

IOE, Pulchowk Campus

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CERTIFICATE

This is to certify that the work contained in this thesis entitled **Strength and Performance Evaluation of Nepalese RC Bridge Pier using Nonlinear Dynamic Response Analysis**, in partial fulfillment of the requirements for the degree of Master of Science in Structural Engineering, as a record of research work, has been carried out by Mr. Sikindar Kumar Chaudhary (064/MSS/R/109) under my supervision and guidance in the Institute of Engineering, Pulchowk Campus, Lalitpur, Nepal. The work embodied in this thesis has not been submitted elsewhere for a degree.

………………………

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Research is a process of advancement of knowledge for the benefit of society and mankind. It is the greatest contribution that a person can give to the society and mankind. To get an opportunity to do research is actually a great honor one can have. A feel of satisfaction that is achieved while carrying out a research work is almost impossible to express in words; can only be felt.

The research work presented in this thesis was conducted at the department of civil engineering, Pulchowk campus, Institute of Engineering, Tribhuvan University, Lalitpur, Nepal. The thesis was carried out under the supervision of Dr. Roshan Tuladhar.

First of all, I would like to express sincere gratitude to my supervisor Dr. Roshan Tuladhar for his valuable advice, guidance and kind support during the entire research work.

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ABSTRACT

Nepal lies in one of the world's most seismically active region. During the 1995 Hyogo-Ken Nanbu (Kobe) earthquake (M 7.8), highway structures were damaged or severely affected, particularly the single – column –type RC piers. During the 1989 Loma Pita earthquake (M 7.1) in California, widespread damage was reported to the region's highways and bridges. Five different conditions of Bridge Pier have been considered with investigation of all the database of design available at the DOR. Available bore-hole soil database of the Bridge Pier have been utilized. The analysis showed that the RC Bridge Pier is safer in the event of earthquakes for simulating the ground conditions encircling the pier. The performance of the RC Bridge Pier was accessed in strong ground motion records. In this study it was intended to access the seismic performance of RC bridge piers under Gazali strong motion. It was observed in the field observation that most of the bridges in Kathmandu Valley were having lowered bed due to scouring. The effect of strong vertical ground motion was investigated for possible reduction in flexural and shear strength of the pier. For this purpose, fifteen models have been developed. These fifteen models are grouped into three sets. First set consists of the five models having only horizontal direction of earthquake is applied. Second set consists of five models with horizontal & Vertical direction of earthquake is applied and third Set consist of the five models with Scouring is considered. These studies have revealed that have got profound effect on the performance of the RC structures. The lateral extent of soil mass for each of the five different bridges were fixed based on trial computations until the percentage difference in consecutive response quantities were found within the limit of 0.4%. The lateral extent for the AO is fixed as 400 m ,AW is fixed as 500 m, AP is fixed as 400 m , IP is fixed as 500 m and BPis fixed as 400 m. The light damage, considerable damage and also failure damage of the Bridge pier at different bridges for different cases (AO, AW, AP and BP).The introduction of strong vertical ground motion increase the vertical displacement by 48 % all bridges. It means horizontal ground acceleration (0.71g) and vertical ground acceleration (1.37g) both applied horizontal displacement increases as well as vertical displacement increases. The Nepalese RC Bridge Pier (In 1990-2000) is over safe. The Nepalese RC Bridge Pier (After 2000 & Before 1990) is designed under safe.

List of Symbols used

G_o	Initial shear modulus
μ	Poisson's ratio
N	Corrected standard penetration test value
V_s	Shear velocity
f_{ck}	Uniaxial compressive strength of concrete
f_c'	Uniaxial compressive strength of concrete
f_t	Tensile strength of concrete
f_y	Yield strength of steel
K_n	Spring constant for normal spring
K_s	Spring constant for shear spring
σ_n	Normal stress
ε	Normal strain
τ_y	Shear stress
I_1	First strain invariant
I	mean inelastic strain

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List of Abbreviations used

RC	Reinforced Concrete
FEM	Finite Element Method
PGA	Peak Ground Acceleration
FE	Finite Element
2D	Two Dimensional
SPT	Standard Penetration Test
AO	After 2000 Open
AW	After 2000 Well
AP	After 2000 Pile
IP	Int. 1990-2000 Pile
BP	Before 1990
H	Horizontal Acceleration
HV	Horizontal plus Vertical Acceleration
HVS	Horizontal plus Vertical Acceleration with Scouring

INTRODUCTION

Background and Motivation

Nepal lies in one of the world's most seismically active region. The Himalayan range is the youngest mountain range on the earth. It was formed by the convergence of the Indian plate below the Tibetan plate. The Indian plate is constantly moving to the north and converging below the Tibetan plate by 20 mm annually (Upreti, 2001) and therefore the building process of the Himalayan range is still under progress. The process of convergence of the two plates is known as the tectonic activity. Large amount of energy is accumulated over the region and its eruption in the form of earthquake is today or tomorrow in geological terms. It is this process which makes the Himalayan and the surrounding region one of the seismically active regions on the earth.

All structures that are to be constructed in Nepal must be designed taking into the effect of earthquakes. In light of the worldwide damages that occurred in different kinds of structures, safety of these structures has become a prime concern. Detailed investigations of the seismic performance of the structures that were already constructed in the past or that are currently under construction and the structures that will be constructed in future have to be made with different considerations in seismic design.

Nepal is a mountainous country and there is lots of variation in geology. Nationwide transportation network require a large number of bridges to connect the road on side of the river. Development of transportation infrastructure is the top priority of the GoN and hundreds/thousands of bridges are on its way of design/construction phase.

In Nepal before1980 there is no any specific code provisions or government regulations to design the structure for the earthquake effect. At that time there is lot of variation in design being followed. For identical bridge structure different designers have drastically different outputs: there are no rules for the earthquake design. Seismic performance evaluation and understanding is highly desirable for the DOR for these bridges. In Nepal there is large number of examples of failure of bridges quite earlier than expected design life time.

There are many bridges which showed earlier collapse due to scoring and lower of bed level. The bed lowering is especially a local problem of bridges on the rivers of Kathmandu Valley due to unauthorized excavation of sand for urban construction.

Fig.1.1 Sinamangle Bridge (The Bridge collapsed and fell below the road level).

Fig.1.2 Sinamangle Bridge (scouring; exposure of pier cap)

Fig. 1.3 Balkumari Bridge at Koteshawar (Pile cap is completely exposed above the bed level. Pile is exposed above 1.6m from the bed level)

Problems and Issues

In Nepal before1980 there is no any specific code provisions or government regulations to design the structure for the earthquake effect. At that time there is lot of variation in design being followed. For identical bridge structure different designers have drastically different outputs: there are no rules for the earthquake design. Seismic performance evaluation and understanding is highly desirable for the DOR for these bridges. In Nepal there is large number of examples of failure of bridges quite earlier than expected design life time.

२७ पुल मर्मत गर्न रकम माग

मकर श्रेष्ठ

डिभिजन सडक कार्यालयले मर्मतका लागि सिफारिस गरेको अरनिको राजमार्गको तीनकुनेस्थित बाग्मती पुल । तस्बिर: कान्तिपुर

Fig. 1.4 Kantipur News. (More bridges in Kathmandu valley any time collapse due to scouring; river width reduces and sand excavation etc.)

There are many bridges which showed earlier collapse due to scoring and lower of bed level. The bed lowering is especially a local problem of bridges on the rivers of Kathmandu Valley due to unauthorized excavation of sand for urban construction.

1. Wide Variation in reinforcement exists for almost identical bridges.
2. Variation in types of foundation / soil characteristics adopted exists due to variation in geological / soil characteristics.
3. Many Urban bridges have problem of Scouring and subsequent bed lowering with increased effective length/span of the bridge pier.
4. Nepal is prone to severe earthquake but, seismic designs are not entertained. Bridges are seismically vulnerable. Soil structure interaction and non-linear RC response under strong ground motion needs to be understood for Nepalese bridges for its seismic performance evaluation. So “what is the condition of bridges of Nepal?” needs to be answered with high level of confidence.

Objectives and Scope of the study

Overall Objective and Scope

The seismic performance assessment of typical Nepalese RC bridge piers is the main objective of this thesis work. The research work is limited to typical bridge pier systems-open-well and piled. The non-linear dynamic analysis is performed in two dimensions only using WCOMD.

Specific Objectives

The specific objectives of this research work are as follows:

1. Evaluate the seismic performance of Nepalese RC Bridge Pier Systems under horizontal as well as horizontal plus vertical strong ground motion.
2. Assess the different failure conditions and modes for the typical Nepalese RC bridge pier systems.
3. Identification of the critical condition/-location in the Nepalese RC Bridge Pier for probable retrofitting.
4. To determine the effect of scouring and bed lowering on Nepalese RC Piers.
5. To analyze the performance of Nepalese RC bridge pier in different time span.

Methodology

The different country codes of bridge analysis & design, necessary loading data, geotechnical investigation reports, structural design report and the drawings will be collected from the Bridge Unit, department of road (DOR), Babarmahal, Kathmandu. Field observation of different bridges of Kathmandu Valley, to find out average representative bed lowering of bridges due to scouring problem. Select some typical bridges from different region of Nepal.

Nonlinear finite element analysis using WCOMD software will be carried out to identify the real behavior of the structure subjected to static and dynamic loading conditions. The comparison of the results of the linear elastic analysis based design and that of nonlinear seismic performance analysis will then be made to identify the deficiencies and gap between the real structural behavior and the presently employed methodologies of analysis and design.

The RC nonlinear analysis will be coupled with soil-structure interaction in time history; the structure will be subjected to strong ground motion with strong vertical recorded motion with the aid of computer software WCOMD.

The flow diagram for the thesis work is shown in figure 1.5.

Data Collection from DOR
(436 RCC Bridges: Before 1990, 30; 1990 to 2000, 142 and after 2000, 264 bridges)

↓

Their Foundations (Open: 158; Pile: 72 and Well: 126 bridges

↓

Their Lanes (Single: 202 and Double: 158 Bridges)

↓

Design loads (IRC class AA: 10; class A 398 and class B 28 bridges)

↓

Selected Five RC Bridges located at different regions of Nepal constructed on different Years and having different foundation systems; but all designed with Class 'A' loading

- After 2000 open, Madi Khola Bridge, Rolpa District. East Consultant, May 2008:18 m span , single lane
- After 2000 Well, Ghardha River Bridge, Saptari District, JV: MEH Consultants, Tulsi Engineering Consultants and Multi Lab Pvt.Ltd., October 2006:18m span, single lane
- After 2000 Pile,Sankata Bridge over Bagmati River along Gusingal-Tripureshwor Road in Lalitpur District JV MEH Consultants, Tulsi Engineering Consultants and Multi Lab Pvt. Ltd. ,October 2006: 21.7 m span,
- 1990-2000 Pile, Jangha Bridge, Mahottari District, MULTI Displinary Consultancy Pvt. Ltd. ,July 1995: 22.64m span, double lane
- Before 1990 Pile, Garun Bridge, SILT consultant, June 1989:18.27m span, double lane

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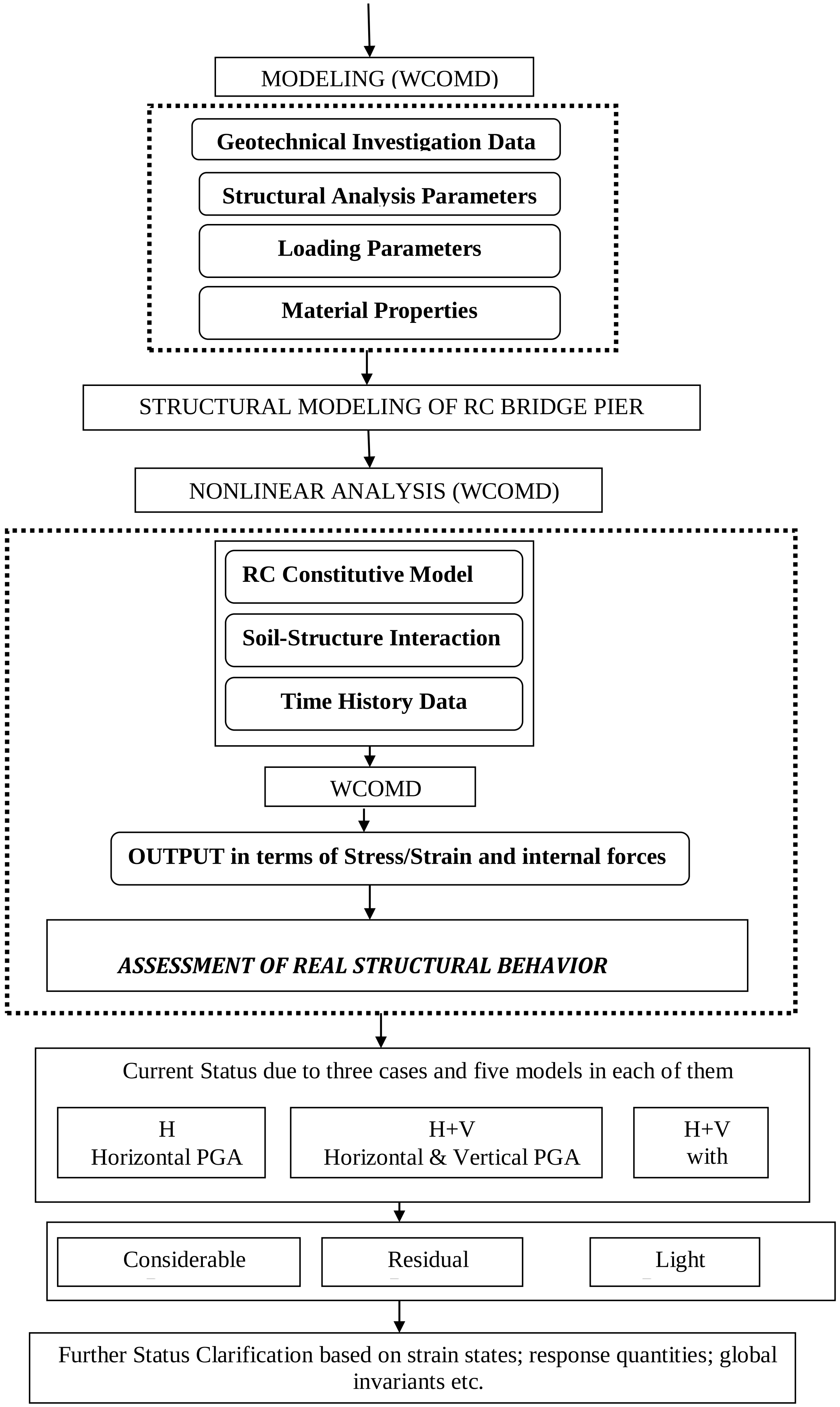


Fig. 1.5 Flow Diagram

Organization of the thesis

The contents of this thesis have been divided into seven chapters as follows.

Chapter 1 presents the introduction to this research work. The motivation and scenario of Nepalese RC Bridge Pier behind this research work is explained in this chapter. Based on problem statement objectives are listed. The methodologies followed are also summarized in the form of flow charts in this chapter.

Chapter 2 provides the literature review related to the past researches on RC Bridge Pier and description of soil-structure interaction problems. The chapter also discusses the various issues in finite element modeling of the soil-structure system and discusses various methodologies used by different authors for finite element modeling of soil-structure interaction problems.

Chapter 3 describes the dynamic of the coupled RC Pier-Soil system. The constitutive model for soil that has been adopted in the analysis is explained in this section. Also, the constitutive model for joint element is presented here.

Chapter 4 is concerned with the analytical parameter of the RC Bridge Pier. The geological profiles, location & geometry of the structure, material properties, input static & dynamic loads and different analysis cases; all are described in this chapter.

Chapter 5 presents the results of various parametric studies that have been conducted during the research work and also presents the interpretation of the obtained results.

Chapter 6 presents the conclusions drawn after carrying out the research.

Chapter 7 gives the list of related topics that can be considered for the future research works.

LITERATURE Review

Soil - Structure Interaction

(Rosenblueth, 1980; Kramer, 1996; Kausel, 1983)

If the motion at any point on the soil structure interface differs from the motion that would occur at the same point in the free field when the structure was not present, there is an interaction between the soil and the structure and is referred to as soil-structure interaction. There are two reasons for this difference in the motion. First reason is the inability of the structure (substructure) to comply with the free field deformation. The second reason is that the dynamic response of the structure itself will also induce the deformation of the supporting soil. There are in general two approaches to analysis of soil-structure interaction effects: the **direct approach (Single Step)** and the **multi step approach**. In the direct approach (often referred to as total solution), the entire soil-structure system is modeled and analyzed in a single step. The analysis may be done in time domain or frequency domain. In the time domain analysis, the system of differential equation is solved directly through step-by-step integration with respect to time. In frequency domain analysis, transfer function of any desired effect is obtained by solving at each frequency. System of algebraic equations and time history response is then computed through the use of Fourier transforms. The time domain analysis allows the treatment of nonlinear behavior of the material whereas the frequency domain analysis is limited to linear problems only. In the multi step approach, the analysis is divided into two parts: kinematic interaction and inertial interaction. The total solution is then obtained by the superposition of the two effects. As the method relies on the superposition principle it is limited to linear problems only.

Seismic Bed Rock

(Tadanobu, 2000)

Bedrock is a relatively hard solid rock that underlies softer rocks, sediments and soils. The location of this bedrock can be at the depths of few kilometers to several hundreds of kilometers from the ground surface. While conducting the soil-structure interaction analysis we need to locate the bedrock. But consideration of this bedrock at the actual depths is not practical because it is very difficult to investigate the properties of the soil layers at these large depths. Further, when conducting the analysis using the numerical techniques such as FEM, it is practically not possible to construct huge mesh incorporating such depths. For this reason, a simple concept of the seismic bedrock has been developed. This seismic bedrock does not necessarily coincide with the geological bedrock. Several proposals have been developed for setting the seismic bedrock. However, there is a lack of uniformity in the treatment of the seismic bedrock. The deepest seismic bedrock has been set at the layer of granite experiencing a shear wave velocity of 3 kilometers per second. Similarly, the shallowest seismic bedrock has been set for a layer having N value of about 50 (Japanese SPT value). The consideration of this seismic bedrock is also dependent on the objective of the investigation that is to be carried out. The philosophy behind the setting of the seismic bedrock is quite simple. Seismic bedrock is a physical boundary that separates the effect of the localized soil conditions at the point of observation from the other factors affecting the ground motion. In conclusion, we can say that the setting of the seismic bedrock depends in the objectives of the analysis, analytical model used, importance of the structure and other engineering considerations.

Finite Element Modeling Issues

Today, finite element technique is the most widely used numerical technique for solving various kinds of structural problems including underground structures. The two and three-dimensional dynamic response analysis and the soil-structure interaction problems are usually carried out using the dynamic finite element analysis (Kramer, 1996). Despite of its wide use, extreme care has to be exercised during the formulation of the problem for arriving at the acceptable solution. Further, the solution is always approximate and is subject to careful interpretation. While dealing with underground structures which has to take into account the soil-structure interaction effects, factors such as size and shape of the elements, modeling of the internal damping of energy dissipation and reproduction of semi-infinite soil mass through the use of appropriate boundary condition in a finite domain of finite elements have to be considered (Kramer, 1996; Kausel, 1983; Maekawa et al., 2003).

Discretization Considerations (Kramer, 1996; Kausel, 1983): The use of coarser mesh can result in the filtering of high frequency components whose short wave length cannot be modeled by widely spaced nodal points. However, it is possible to use larger elements far away from the region of interest at a considerable distance from the structure.

Far field Effect (Rosenblueth, 1980; Kramer, 1996; Kausel, 1983; Maekawa et al., 2003): Determination of lateral extent of soil mass and selection of appropriate boundary condition to simulate the behavior of semi-infinite soil domain with a finite domain of finite element mesh is an important factor that should be considered in the analysis of soil-structure system. The lateral extent of the soil mass should be such that there is a considerable length of soil beyond which free field deformations are recovered. This fact is clearly depicted in the figure 2.1 (Rosenblueth, 1983).

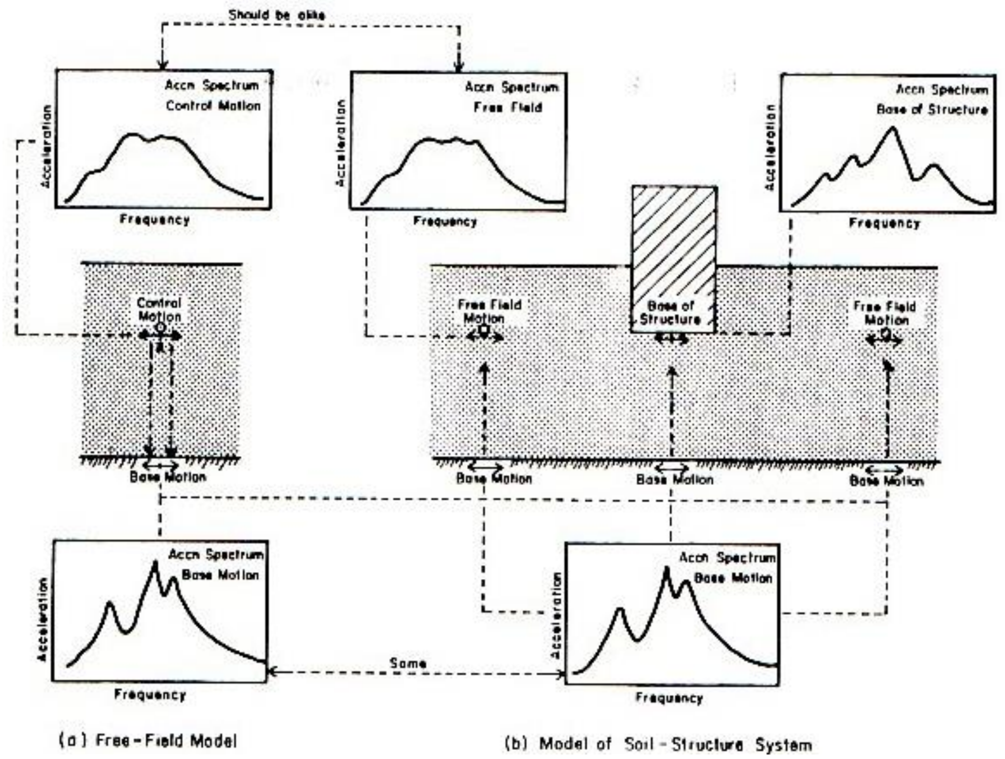


Fig.2.1: Soil- Structure Interaction Model

Regarding the type of boundary to be used, conventionally there is a tradition to use hinge or roller support at the lateral boundaries. These supports do not correctly represent the transmitting boundary. Transmitting waves are perfectly reflected back into the domain after reaching the supports and no energy is transmitted out of the domain. There is a tendency to neglect this factor by asserting that the effect may be neglected by placing the boundary far away from the structure. Adopting this strategy,

although the effect gets reduced as the domain becomes larger, this often leads to a very large mesh demanding a tremendous amount of computer speed and memory. To solve this problem Wolf proposed the superposition boundary where the total solution is decomposed into symmetric far field boundary and anti symmetric far field boundary. The soil-structure system needs to be analyzed twice for each of the two different boundary conditions. An (1996) used this approach for the analysis of the underground subway stations, the Daikai and Kamisawa station located in Kobe using the software WCOMD. To make the problem computationally efficient, An (1996) developed a strategy of overlapping of two narrow boundary zones with complementary boundary condition corresponding to the symmetric and anti symmetric far field boundary conditions. The system was installed in the software WCOMD. The boundary condition is depicted in figure 2.2.

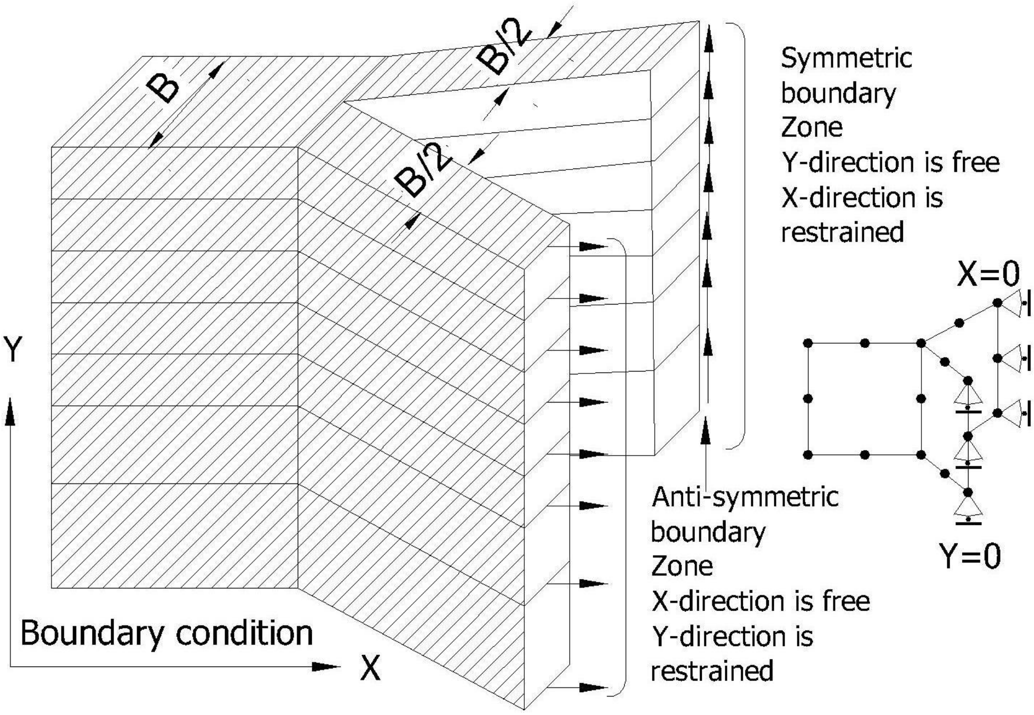


Figure 2.2: Mixed Artificial Boundary condition attached to the main mesh

This boundary condition was attached to the main mesh of the soil-structure system. Each boundary zone has got half of the total stiffness and mass. The wave that propagates from the main mesh enters the two boundary zones simultaneously. After reflection at the artificial boundary the amplitude of the waves reflected back to domain will have same magnitude but opposite sign and thus cancel each other. This result in zero energy transmitted back to the main mesh simulating the far field boundary condition roughly. The total length of the domain is checked so that the whole domain can dissipate almost all of the energy from the main mesh to the far field.

The physics of the Symmetric and anti symmetric lateral boundary condition

Consider a wave pulse advancing along the string with one end free and other end fastened at the rod (support). If the support is fixed, the wave pulse must remain at rest at the support.

The wave pulse arriving at the support then exerts a force; the result is that the reaction force kicks back on the string and sets up a reflected pulse traveling in opposite direction. The nature of the reflected pulse is shown in the figure 2.3 (a). Similarly, if the support is free to move in transverse direction (the string is tied to a light ring that slides on a smooth rod), the wave pulse arriving at the support causes the end to overshoot and sets up the reflected wave in opposite direction (Sears et al., 1985). The nature of the pulse is shown in the figure 2.3 (b).

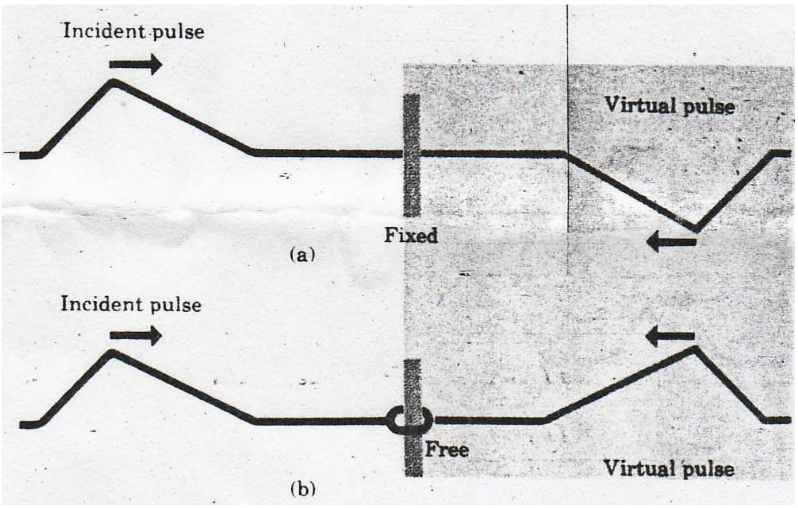


Fig.2.3: Nature of the pulse (a) at a fixed end of a string, and (b) at a free end, in terms of imaginary virtual pulse

It is evident from the figure that the nature of the reflected pulse in the two cases is exactly opposite. Thus when the two reflected waves are superposed, they cancel out each other, the result being no wave pulse traveling back along the string.

Wolf proposed this kind of strategy for evaluation of soil-structure interaction problems (Maekawa et al., 2003). This strategy has been adopted by an (1996), and employed in the software WCOMD as explained in section.

Joint Element

(Tadanobu, 2000; Maekawa et al., 2003)

The soil-structure interface can be modeled using the joint elements. Shear springs having a spring constant K_s and normal spring having a spring constant K_n are used to express the characteristics of these elements. The ideal constitutive relationship of the joint elements is shown in figure 2.4. In the ideal relationship both K_s and K_n become infinite and henceforth, the numerical computation becomes infinite. Thus, for carrying out the numerical computation, the values of these spring constants have to be set finite and as large as possible. The normal stress/strain relationship is assumed to be bilinear for opening/closure mode. The normal stress is zero in case of separation and when the soil and structure is in contact a large value of the stiffness is sufficient to avoid overlap of elements is assigned.

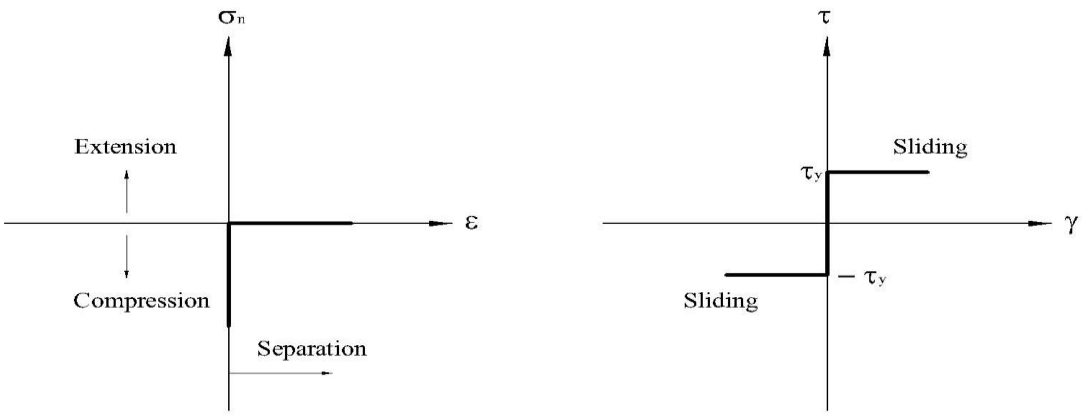


Fig. 2.4: Ideal Constitutive relationship of Joint element

Past Research on RC Bridge Pier Structures

G.M. Calvi and G.R. Kingsley

Since structural damage can be considered to be directly related to displacement demands, it can be controlled most efficiently through the imposition of displacement (or drift) limits rather than strength limits. In WCOMD even micro limits are considered using limiting strains not displacement.

Junichi Sakai, Shigeki Unjoh

To investigate the effects of multidirectional seismic excitation on the dynamic response of reinforced concrete bridge columns, an earthquake simulation test and a series of dynamic analyses are conducted and found the effect could be serious with even change in failure mode, therefore two separate cases of time history analyzes were considered.

Past Research on Finite Element Modeling

Montesinos, et.al (2006) used plane strain elements for modeling the soil-structure system. A free boundary condition was adopted at the lateral sides of the mesh. The bottom of the discretization was placed at 58 meters from the surface corresponding to the top of the gravel layer in Port Island. Fixed boundary condition was adopted at the bottom of discretization. The finite element model is shown in figure 2.5.

The lateral boundary on the both sides of the structure was placed far enough from the structure so that they have negligible effects on the response of the structure. This was confirmed by carrying out several numbers of simulations by placing the boundaries at different distances from the structure. The location was accepted when there was a region of soil between the boundary and the structure where the free field soil deformations were recovered. Free field deformations were previously obtained by running a large mesh without the structure. This resulted in the overall mesh length of 1000 meters.

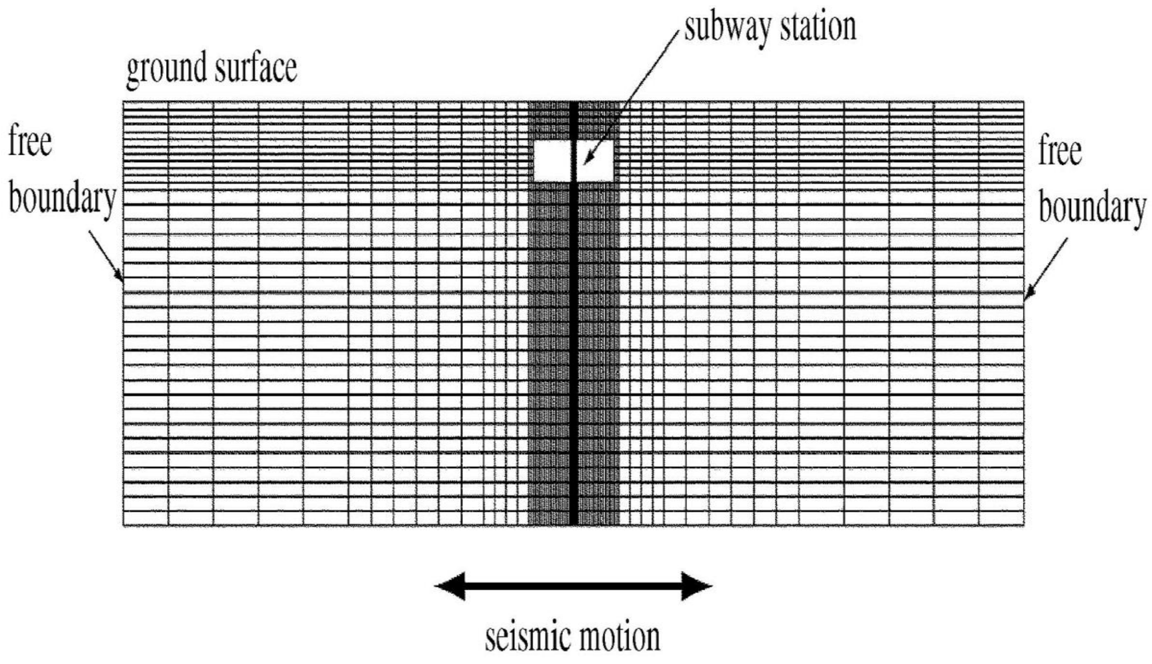


Fig. 2.5: Finite Element model used in soil-structure numerical analysis of Daikai subway station (Montesinos, et.al 2007)

Nishioka and Unjoh modeled the soil as plane strain element and structure as beam elements. The lateral boundaries adopted at the sides were free in horizontal direction and fixed in vertical direction. The finite element model is shown in figure 2.6.

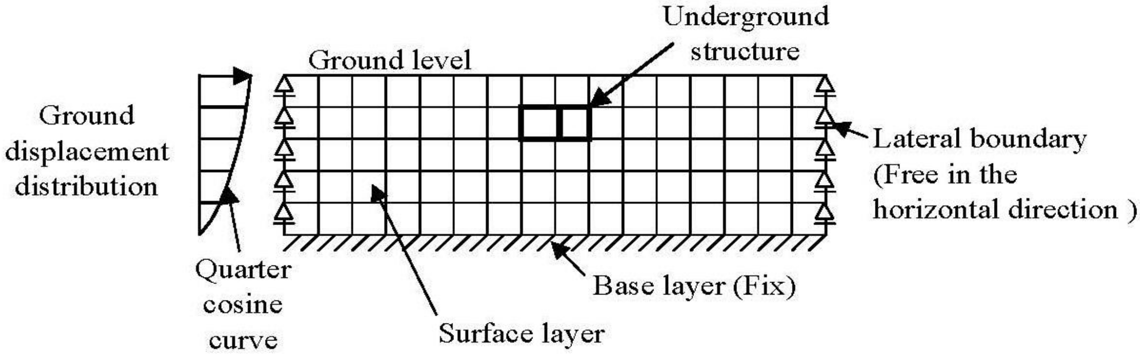


Fig.2.6: Finite element model used for analysis of strain transmitting characteristics of underground structure

Horizontal distance between the structure and the lateral boundaries was set approximately three times longer as the thickness of the surface layer on both sides of the structure. Fixed boundary condition was adopted at the bottom of the dicretization. The thickness of surface layer taken in the analysis is 25.2 m.

DYNAMIC ANALYSIS OF RC Pier – SOIL SYSTEM

The Computational Simulation Tool: WCOMD

When real behavior of the RC structure has to be accessed, engineers generally have two choices, either to conduct experiment in laboratories or to carry out computer simulation. Although, the actual result is obtained by conducting experiment in laboratories, it is limited to particular and standard cases only. On the other hand, the computer simulation has practically got no limits in its application. The computer simulation has emerged as a Virtual Testing Center with which we are able to solve structural problems of complex nature, tackle nontraditional problems and also find out optimal solutions. Other interesting applications include assessment of the remaining life of the existing structures and investigation of damage and failures of structures. The tool mostly employed for simulation of real behavior of the RC structures is the finite element based nonlinear analysis techniques (Okamura and Maekawa, 1991; Cervenka, 2002; Maekawa et al., 2003).

WCOMD is software developed by University of Tokyo, Japan (Okamura and Maekawa, 1991). It is the software developed for nonlinear analysis of RC structures for plane problems. Figures 3.1, 3.2 and 3.3 show the graphical user interface of the UC-win/ mesh and UC-win/ WCOMD. UC-win/ mesh is the program used to generate the finite element mesh. The type of materials, material plate properties, boundary conditions and mesh geometry are all defined in this module. Six kinds of materials are available in UC-win/ mesh: Concrete, Steel, Elastic, Soil, RC joint and Universal joint. For carrying out nonlinear analysis, the mesh file is then exported to UC-win/ WCOMD. WCOMD is the solver where nonlinear calculations are made. The static and dynamic loads are defined in this module. Three types of loading patterns are available in UC-win/ WCOMD: Dead weight, Static load and Dynamic load. The elements used in the WCOMD software are eight nodded quadrilateral isoparametric elements.

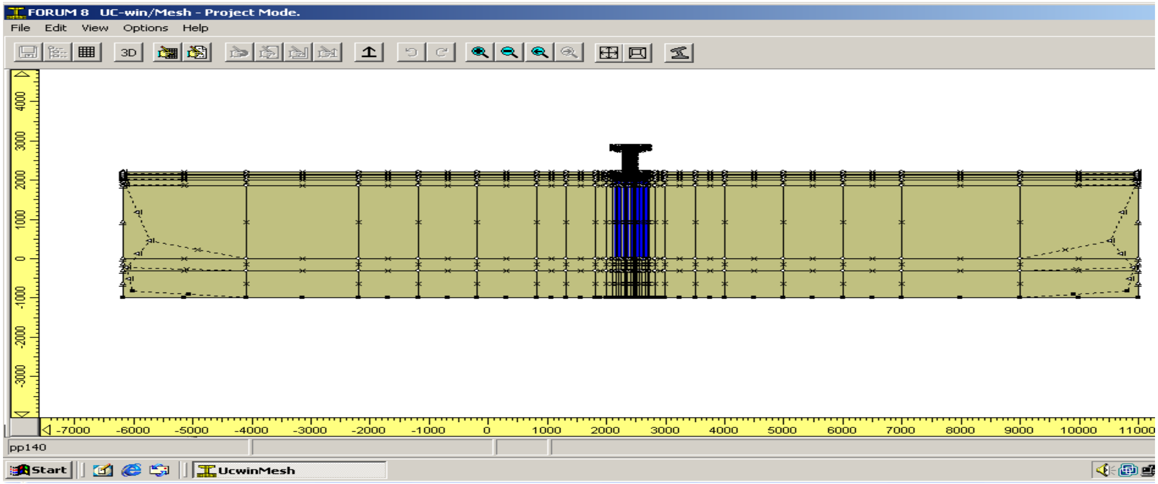


Fig.3.1: UC-win/ Mesh Window (piled model with lateral boundary conditions)

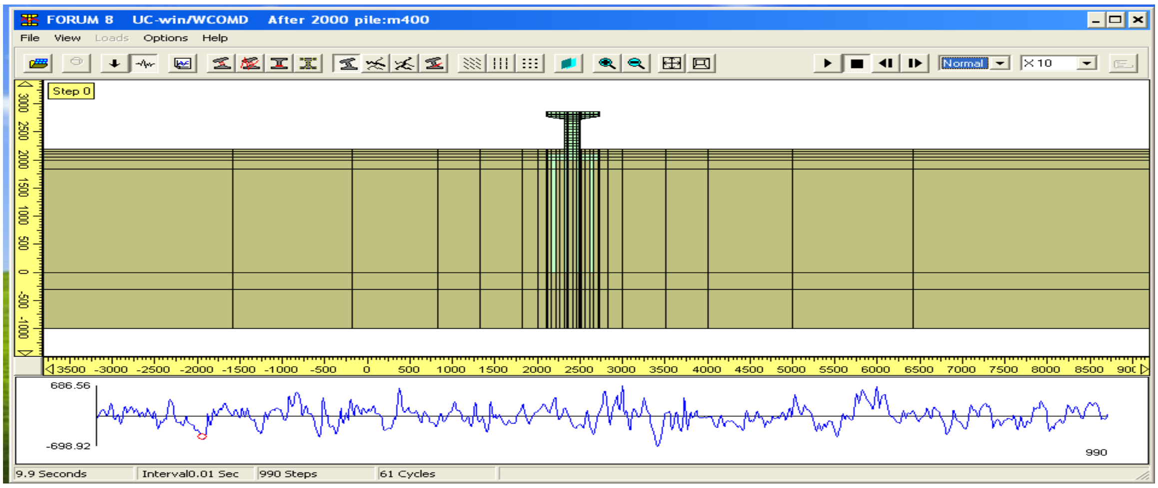


Fig.3.2: UC-win/ WCOMD Window (input ground acceleration in horizontal direction)

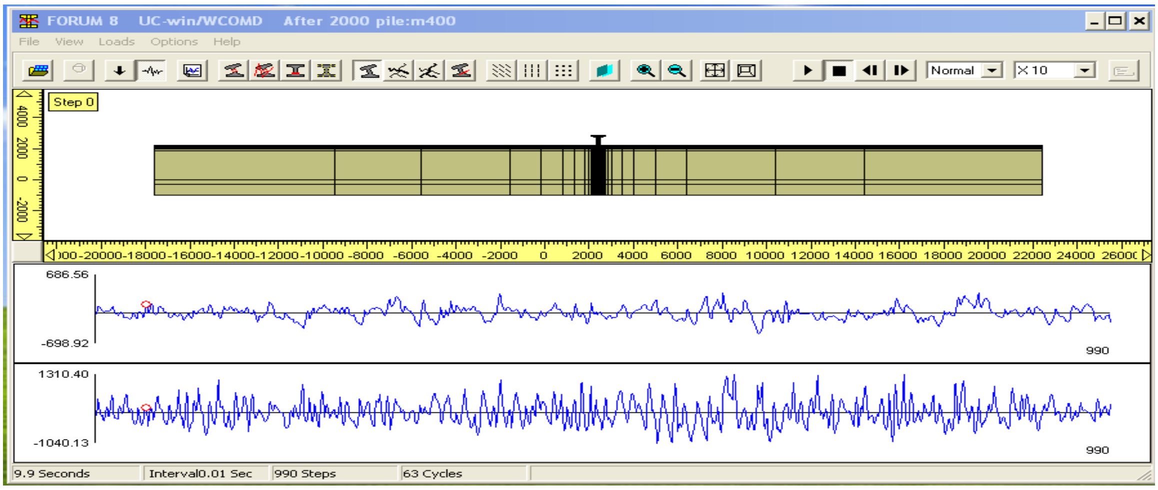


Fig.3.3: UC-win/ WCOMD Window (input ground acceleration in horizontal and vertical directions)

Special constitutive models for finite element based nonlinear analysis for RC has been incorporated in WCOMD such as Tension Stiffening, Compression Softening, and Shear Transfer etc. Constitutive model for soil material has also been incorporated in the software, suitable for the analysis of soil-structure interaction system. For modeling of interface between the RC-RC elements and RC- soil elements, RC joint element and universal joint element models have also been incorporated in the software. The material models for cracked concrete and reinforcing bars in concrete have been developed based on test results of reinforced concrete specimens subjected to uniaxial loadings. Combined material models and reinforced concrete plate element has been developed. The multidirectional smeared crack model has been employed. In this model, the material models are expressed as average stress and average strain relationships. The smeared crack model has been used to describe the overall behavior of a member. However, the smeared crack model is not suitable for regions where large discontinuities occur. For such cases, discrete crack model have been employed. The joint element has been developed for the purpose. The joint element model is shown in figure 3.4.

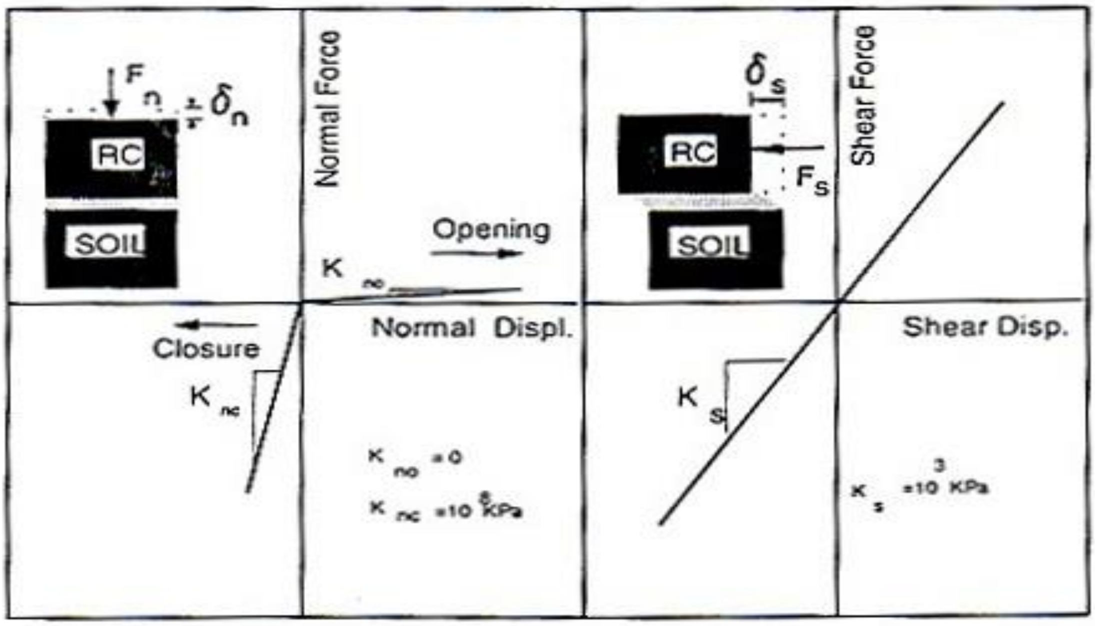


Fig. 3.4: Constitutive relationship for RC/ Soil interface model

The joint element is a one-dimensional element having zero thickness. It expresses the relationship between the normal force and normal displacement and shears force and shear displacement.

Modeling of RC-Soil System

For analysis of partially underground structures, coupled analysis of the complete RC-soil system is necessary. For this nonlinear path dependent constitutive model of both soil and structure is needed. This is mandatory for dealing with the kinematic and inertial interaction between the soil and the structure under seismic excitation.

The Ohsaki's soil model is used as a constitutive model for soil in WCOMD. The Ohsaki model describes the relationship between the shear strain and shear stress with the aid of three parameters, G_o, S_u and Vs. The relationship is as follows:

Generally, only SPT – N value is obtained from the standard penetration test during the geotechnical investigation of the soil. Hence, all the other soil properties are obtained by using the empirical equations based on SPT N value. The following empirical equations are used to obtain the different soil properties:

$G_o = 11.76\ N^{0.8}$ (N/mm^2)

$S_u = G_o / 600$ (N/mm^2) : for clay

$S_u = G_o / 850$ (N/mm^2) : for sand and clay

$S_u = G_o / 1100$ (N/mm^2) : for sand

Shear Velocity is calculated using the following formula:

$$V_s = \frac{1}{100}\sqrt{980\ (cm/sec^2)\,G_0\ (N/cm^2) \Big/ \gamma\ (kN/cm^3)} \quad (m/sec)$$

Here,

G_o = initial shear modulus

S_u = maximum shear strength

Vs= Shear Velocity

Analytical parameter

Geological Profile at Nepalese RC Bridge Pier

The RC Bridge Pier is located different region of Nepal. The deposits consist of alternating beds of sand, sand & clay and clay soil. The bedrock is not found at least up to the depth of 40 meters. It has been proposed to run the Open, Pile & Well Foundation through the sand, sand &clay and clay beds of deposits with about 20 meters deep cutting. The geotechnical investigation works has been carried out for the Bridge Pier only. Three boreholes were made along the river. The investigation was carried out for a depth of 25 meters from the ground surface. The soil profile at the Bridge Pier is broadly divided into two layers.

Location & Geometry of Selected Nepalese RC Bridge Pier for computation

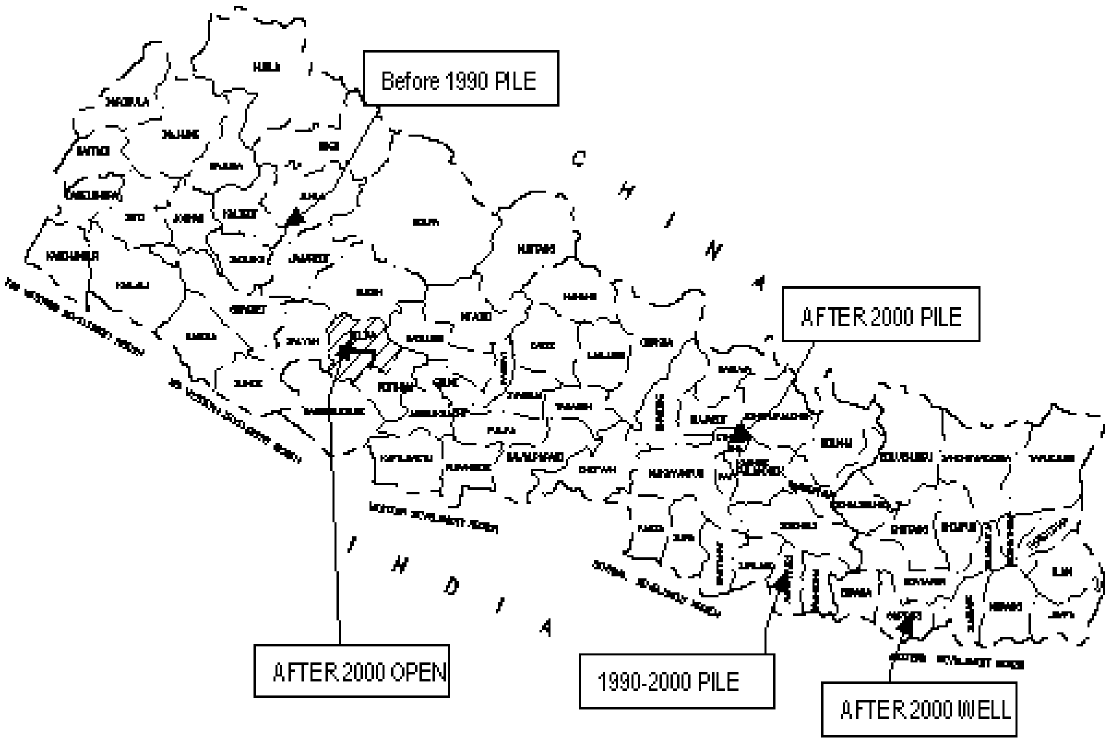
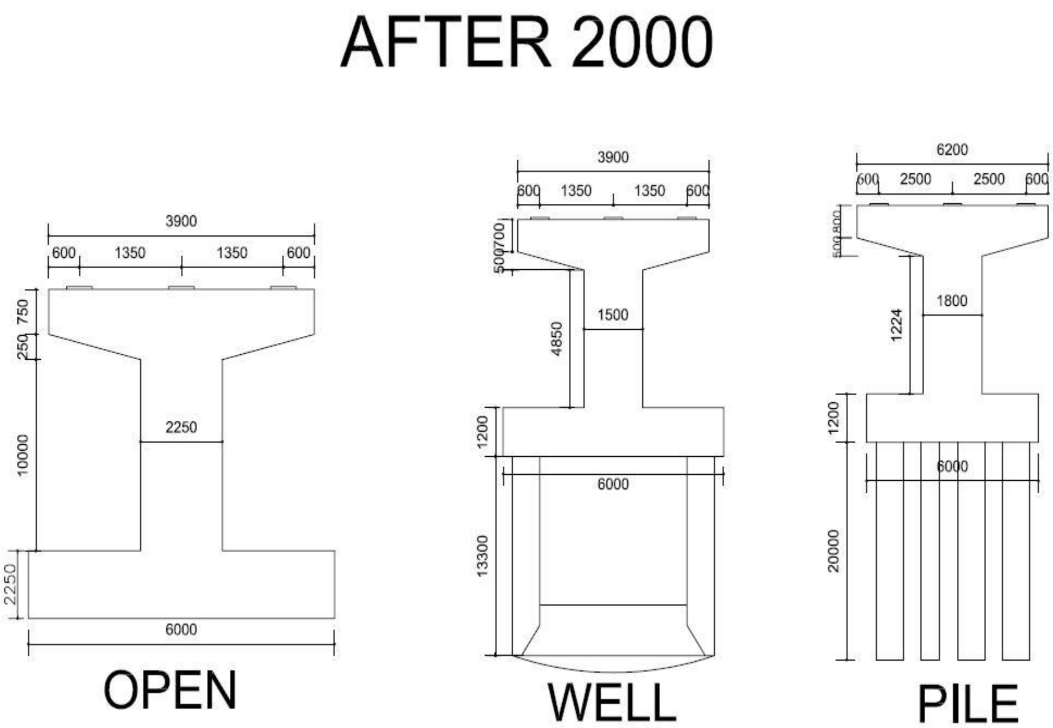


Fig.4.1 Selected RC Bridge Pier for Computation



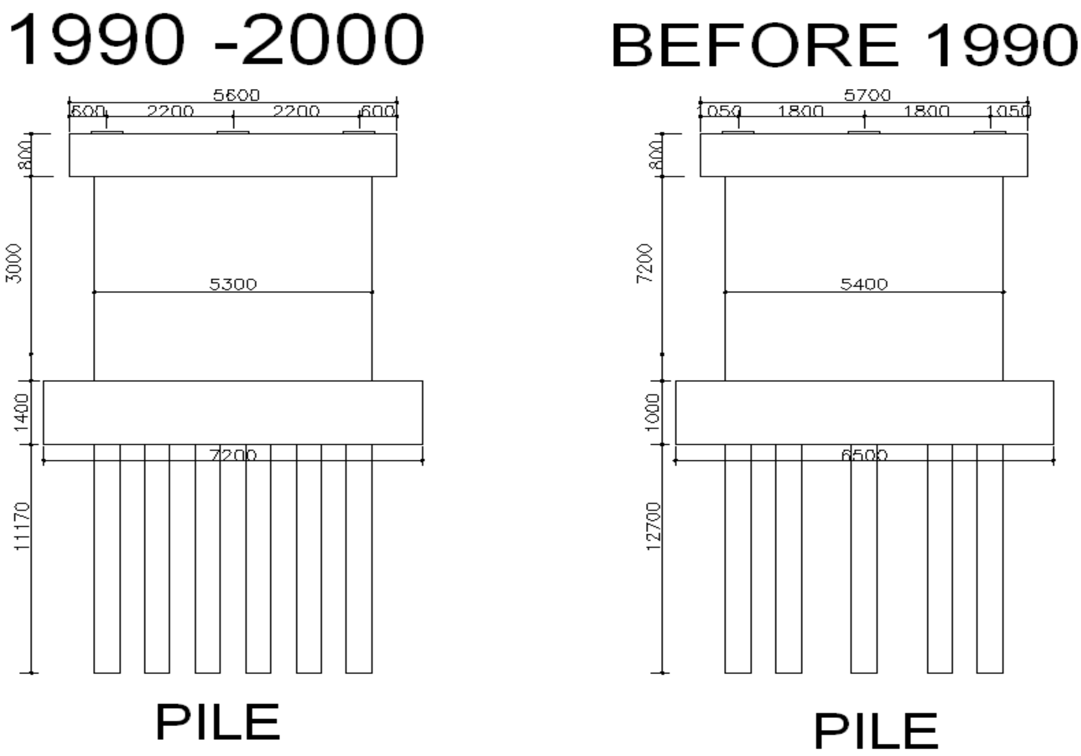


Fig.4.2: Geometry of the Bridge Pier (selected typical pier bridge piers)

Layout of Reinforcements

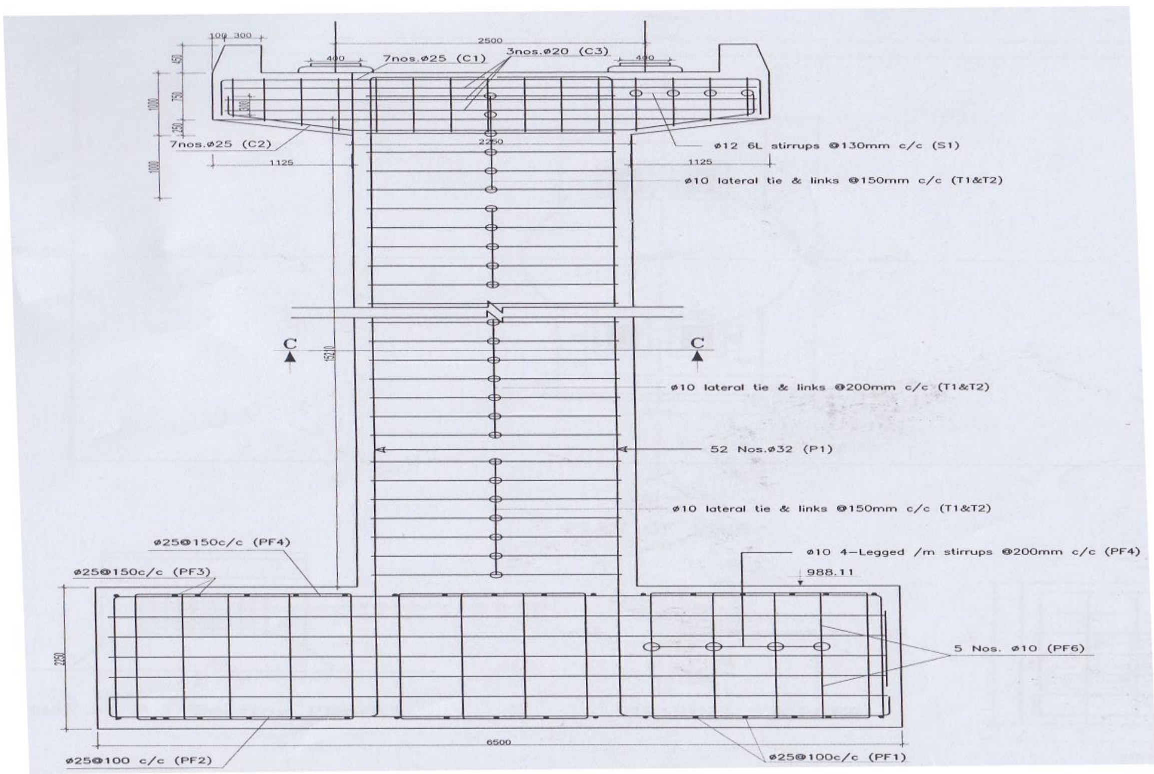


Fig. 4.3: Reinforcement details in Bridge pier After 2000 Open

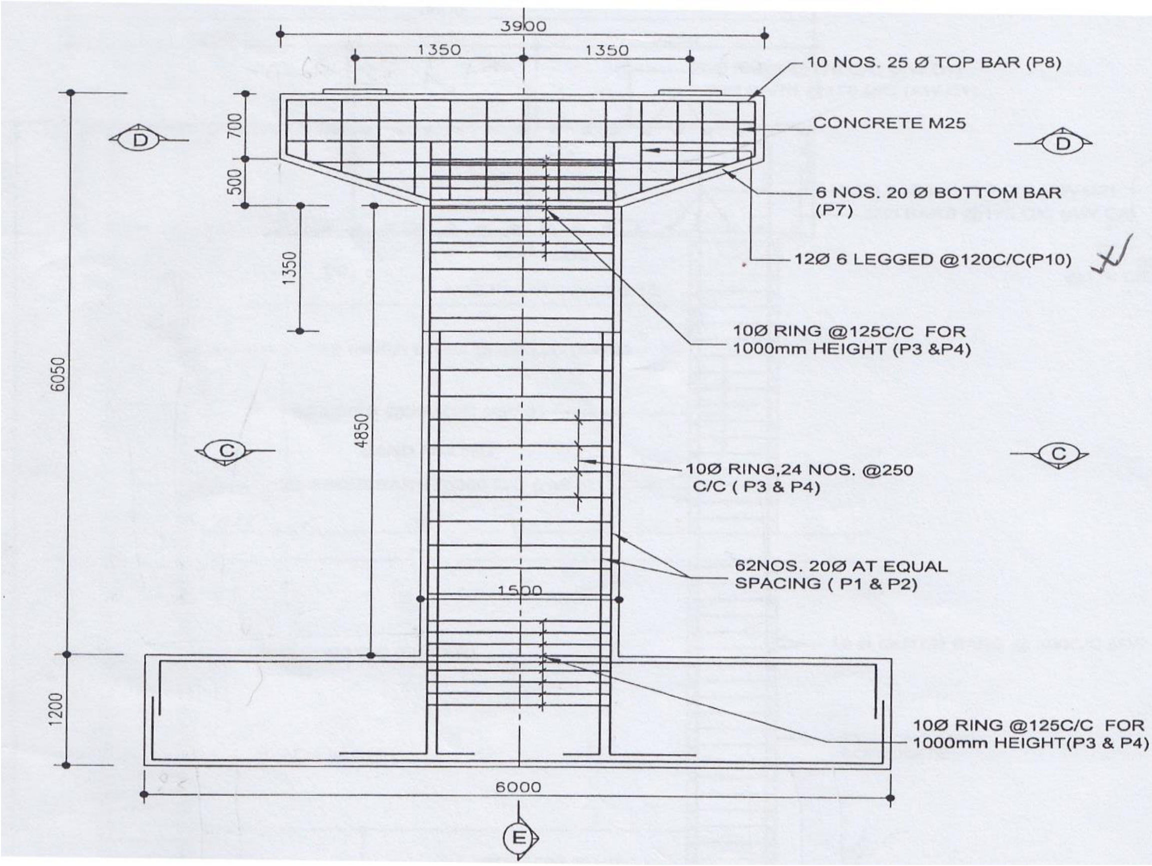


Fig. 4.4: Reinforcement details in Bridge pier After 2000 Well

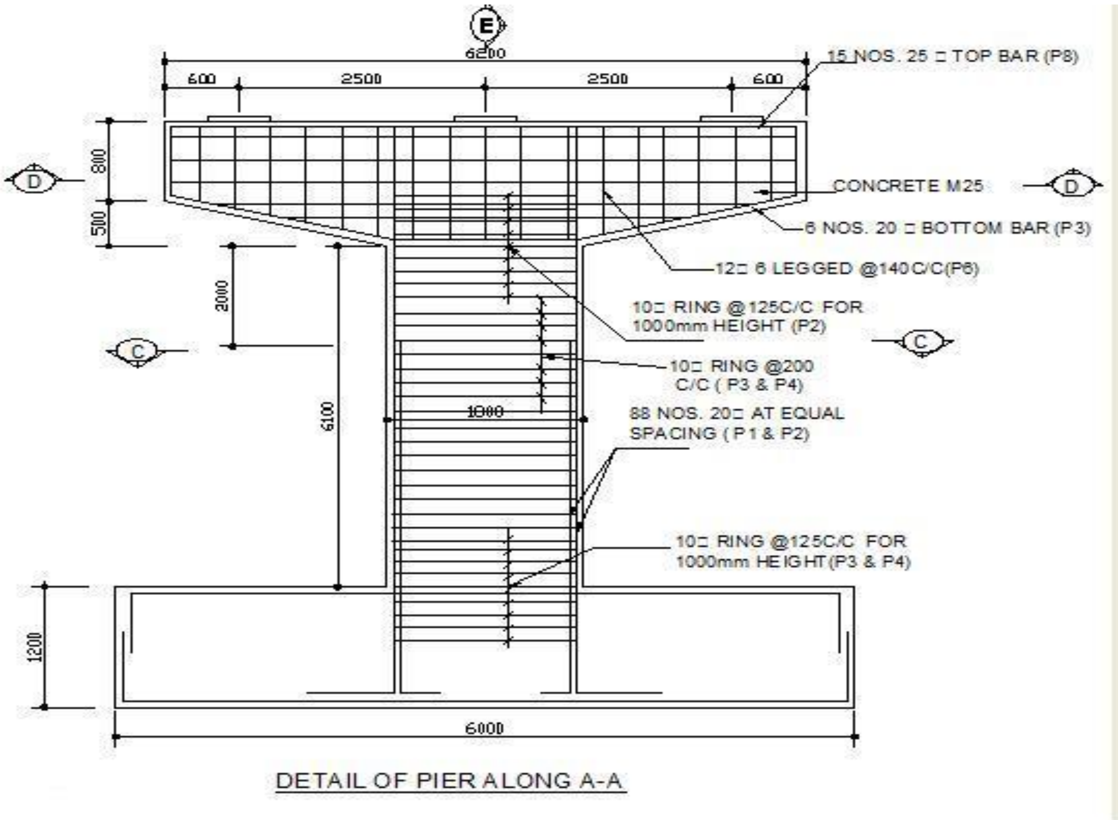


Fig.4.5: Reinforcement details in Bridge pier After 2000 Pile

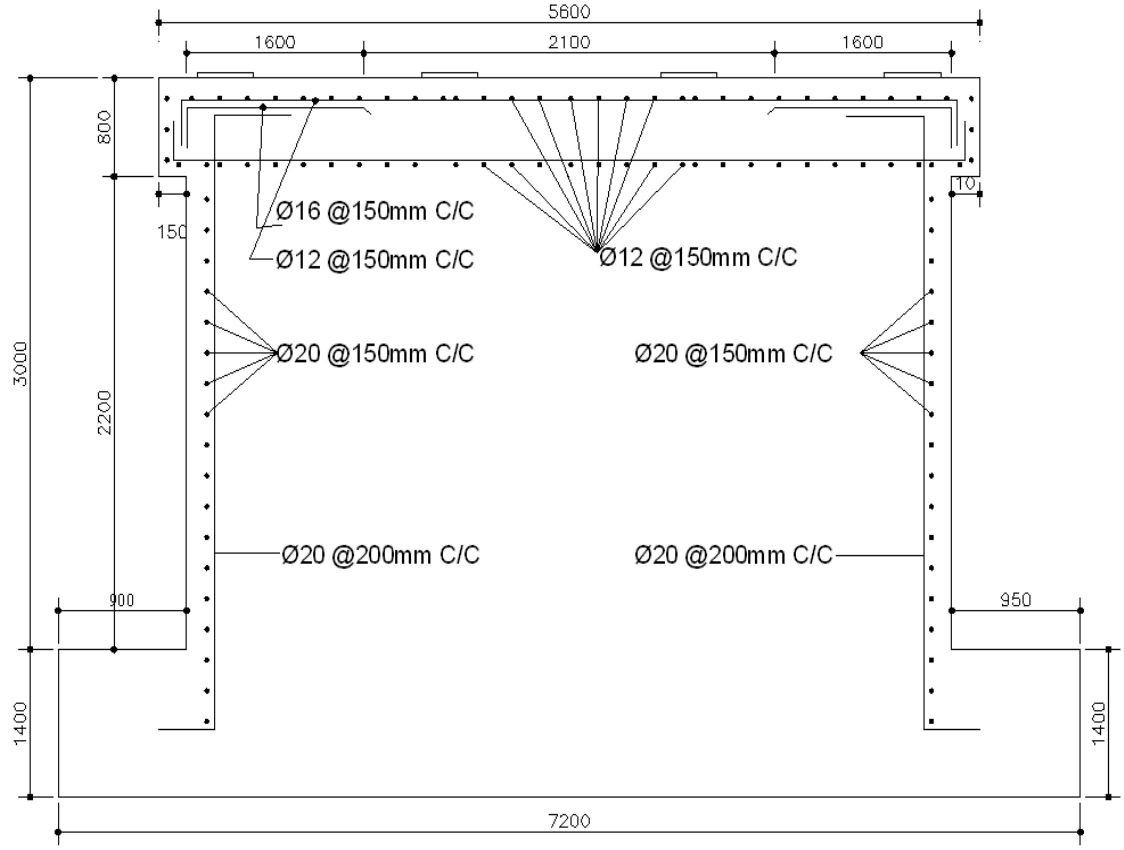


Fig. 4.6: Reinforcement details in Bridge pier 1990-2000 Pile

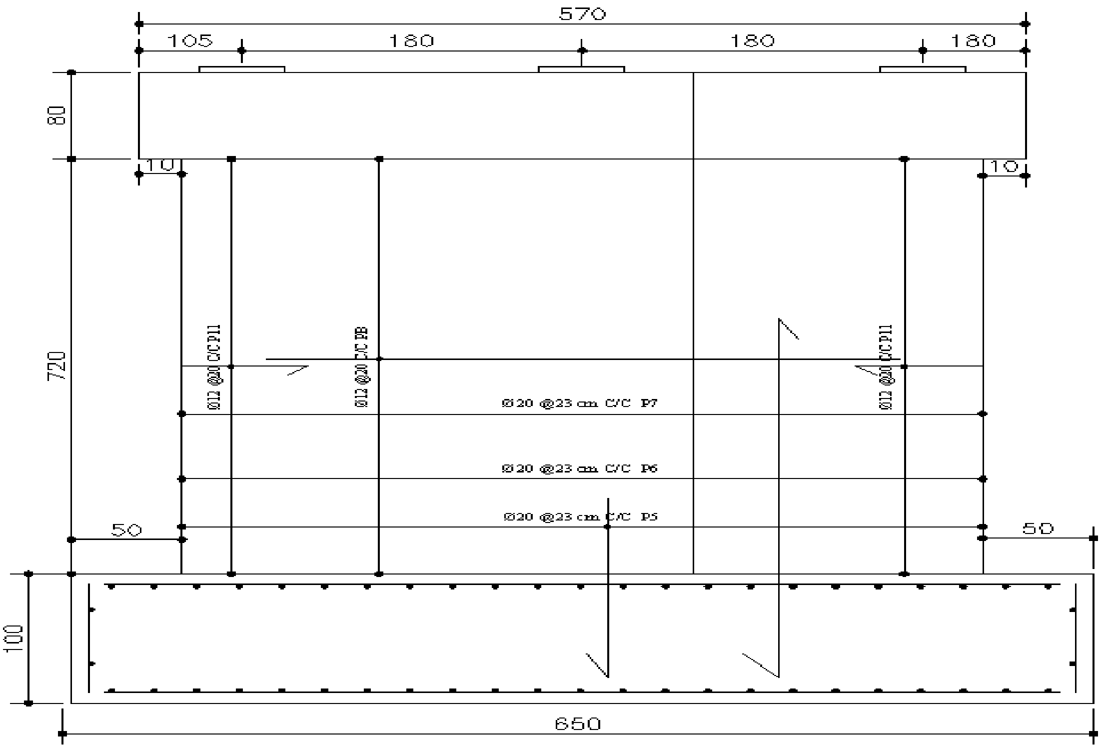


Fig.4.7: Reinforcement details in Bridge pier Before 1980 Pile

Soil Profile used in Computation

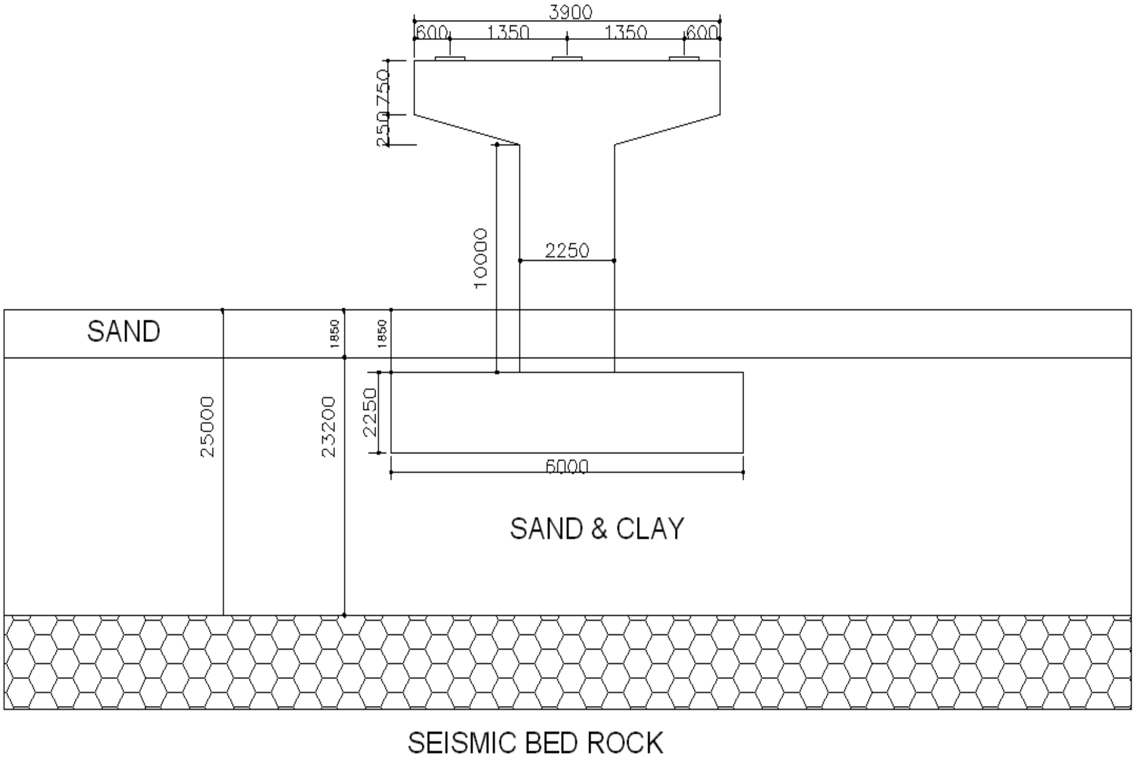


Fig. 4.8: Soil Profile considered in Computation of After 2000 Open

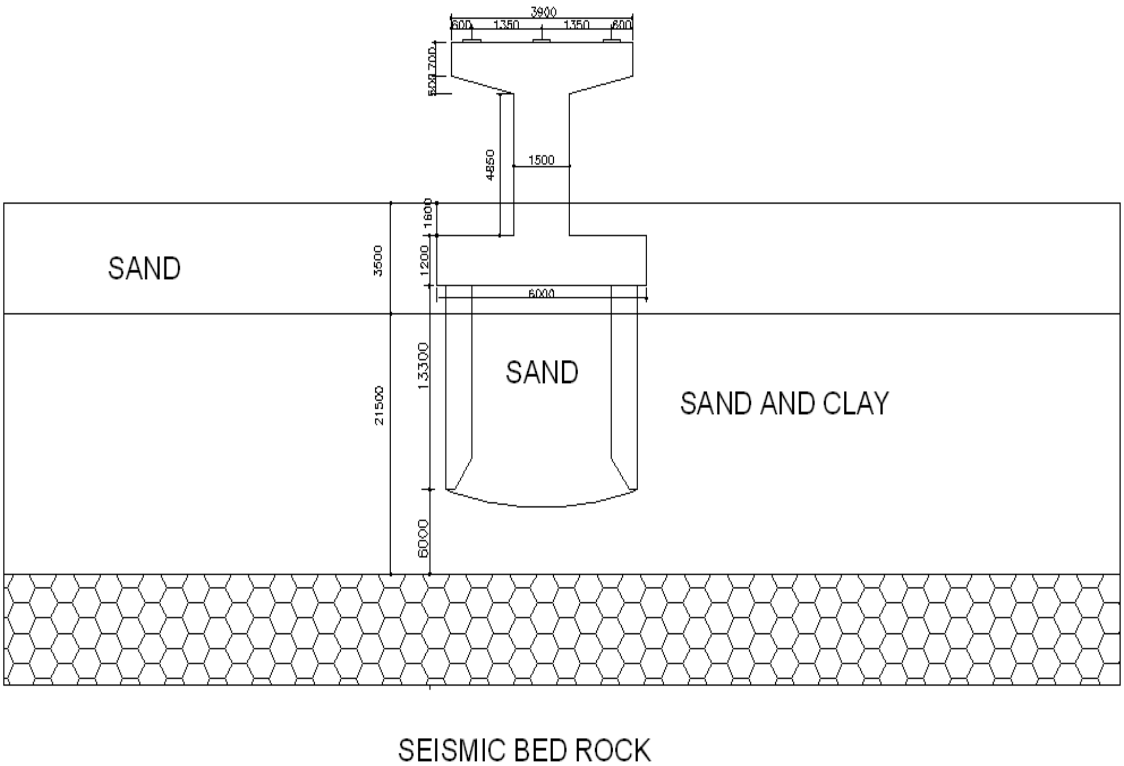


Fig.4.9: Soil Profile considered in Computation of After 2000 well

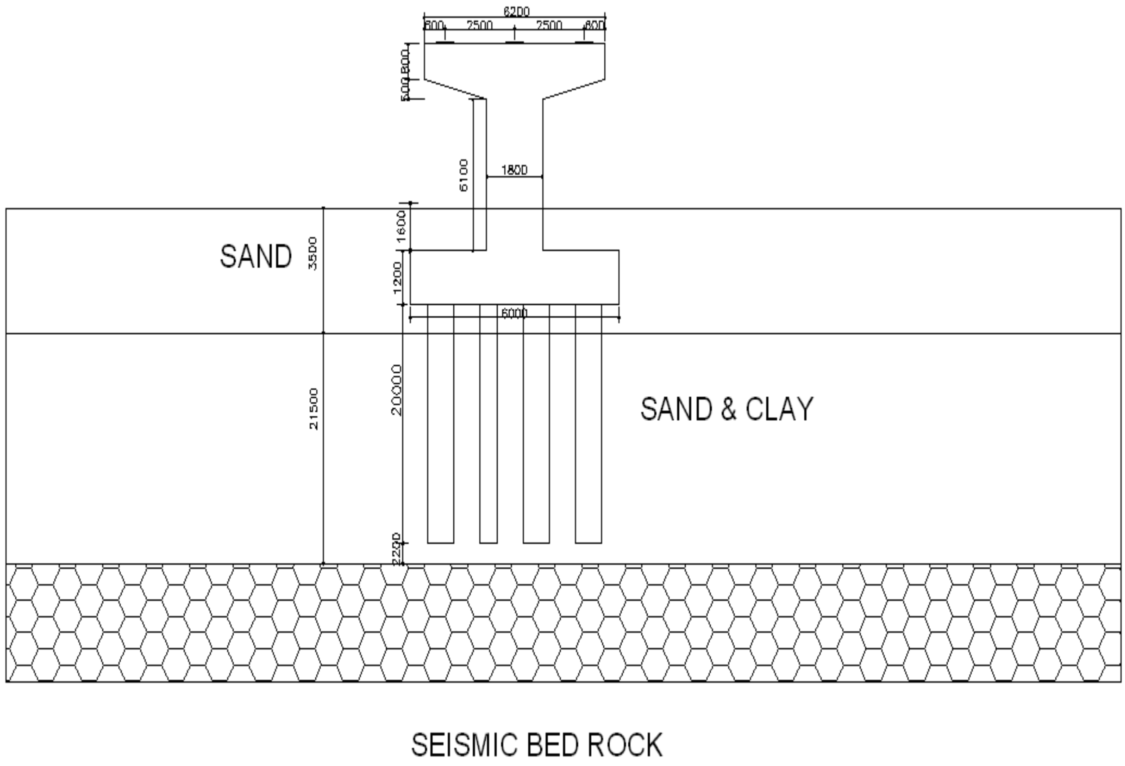


Fig.4.10: Soil Profile considered in Computation of After 2000 Pile

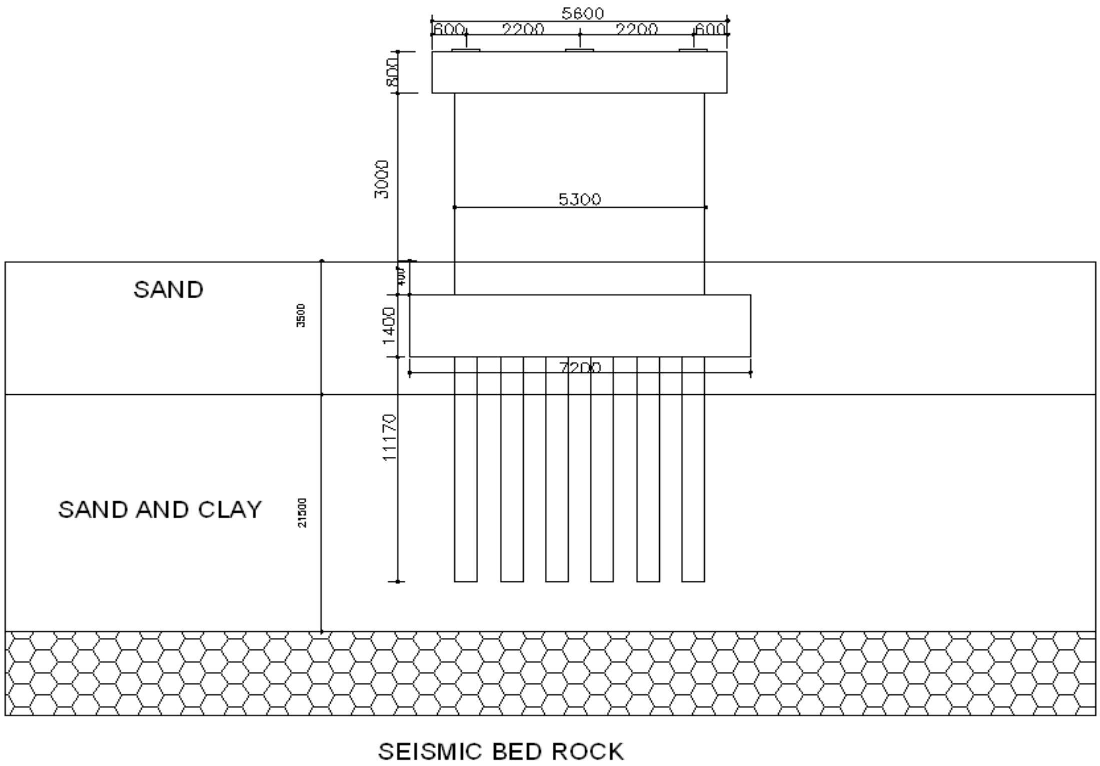


Fig. 4.11: Soil Profile considered in Computation of 1990-2000 Pile

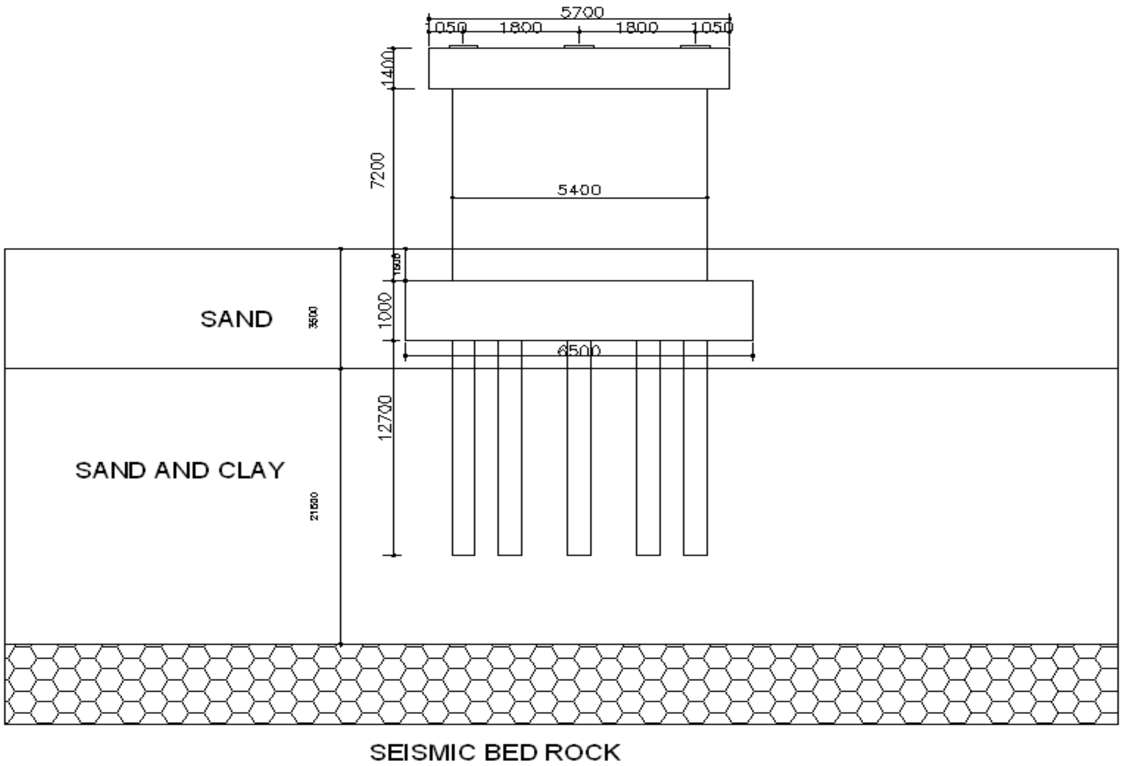


Fig.4.12: Soil Profile considered in Computation of Before 1990 Pile

Material Properties

Structural Concrete and Steel and their characteristics

The RC Bridge Structure is yet to be constructed. For this reason, the structural properties, which were used in the design, is used here as well for the analysis.

Grade of Concrete = M20 = 20 N/mm^2

Grade of Steel = Fe415 = 415 N/mm^2

Table 4.1: Structural Components and their characteristics

S.N.	Description	Percentage of Steel				Width (m)			Depth (m)			
		Pier cap	Pier	Pile cap	Pile	cap	Pier	Pile cap	Pier cap	Pier	Pile cap	Pile
1	After 2000 Open	2.33	2.03	1.13	-	3.9	2.3	2.25	1	10	2.25	-
2	After 2000 Well	0.76	2.68	1.18	1.58	3.9	1.5	1.2	1.2	4.9	1.2	13.3
3	After 2000 Pile	0.85	2.53	1.14	1.76	6.2	1.8	1.2	1.3	6.1	1.2	20
4	1990-2000 Pile	0.63	2.26	2.44	2.53	5.6	5.3	1.4	0.8	3	1.4	9.77
5	Before 1990 Pile	0.3	0.8	0.68	2.19	5.7	5.4	1	0.8	7.2	1	12.7

Soil Material Properties

The different properties of the three soil types of soil used in the analysis are tabulated in table 4.2 below.

Table: 4.2: Soil Profile and its characteristics

Parameters	Unit	After 2000 ,1990- 2000 ,Before 1990	
		Open , Well, Pile	
		Layer I	Layer II
Layer Thickness	m	3.5	21.5
Soil Type		Sand	Sand & Clay
SPT N		7	12
Unit Weight	KN/m^3	18	
Poisson Ratio		0.3	
Go	N/mm^2	56	86
Eo	N/mm^2	145	223
Su	N/mm^2	0.051	0.101
Shear Velocity	m/s	174	216

Input Statics load

Statics load depends upon the types of lane and types of vehicle loading, in this thesis all five bridges two types of lane (single lane & Double land) and IRC class 'A' loading(Fig. 4.13) is to be used as shown table 4.3

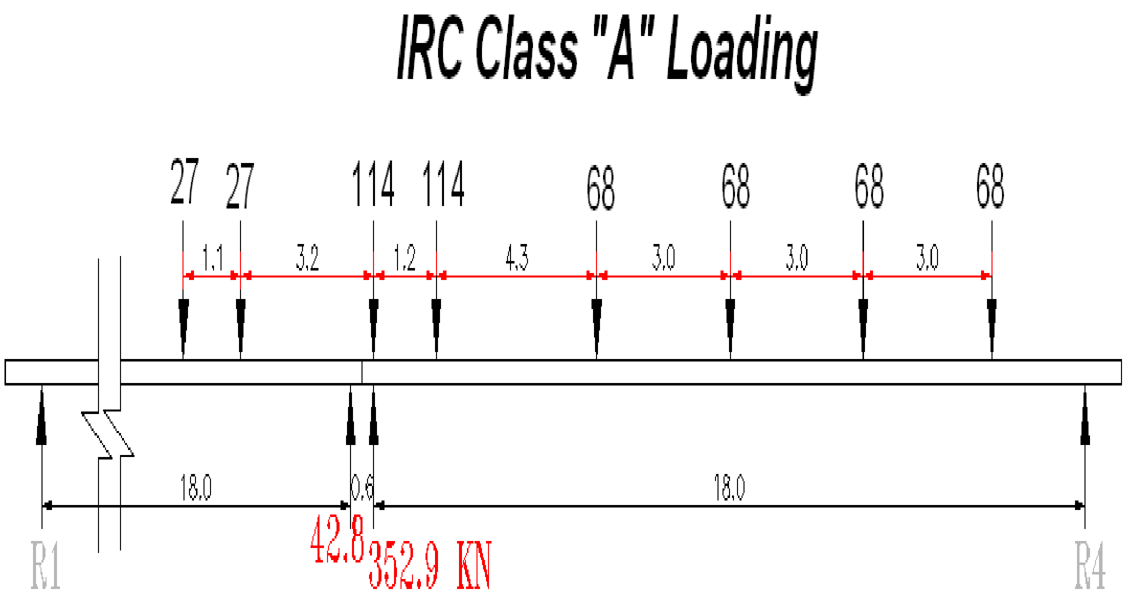


Fig.4.13: IRC Class “A” Loading

Table: 4.3: Static load on Pier from Superstructure

S.N.	Description	Types of lane	Span (m)	Super Structure Load		Total Load on pier from superstructure(KN)
				Self. Weight(KN)	Live Load (KN) including impact	
1	After 2000 Open	single	18	1362.4	601.40	1963.80
2	After 2000 Well	single	18	1362.4	601.40	1963.80
3	After 2000 Pile	double	21.7	1880.1	1134.4	3014.5
4	Int. 1990 to 2000	double	22.64	1961.54	1183.54	3145.08
5	Before 1990	double	18.27	1582.93	955.09	2538.02

Input Earthquake Wave motion

The records of strong ground motions are not available in the vicinity of the site. It was intended to study the effect of strong vertical ground motion. For this reason, the earthquake wave motion recorded at the Gazali, Uzbekistan 1976 observatory is chosen as the input wave for this research. This wave has got horizontal ground acceleration (0.71g) and vertical ground acceleration (1.34g). The earthquake wave is shown in the figure 4.14 and 4.15.

Fig. 4.14: Input Earthquake Wave (Horizontal Acceleration)

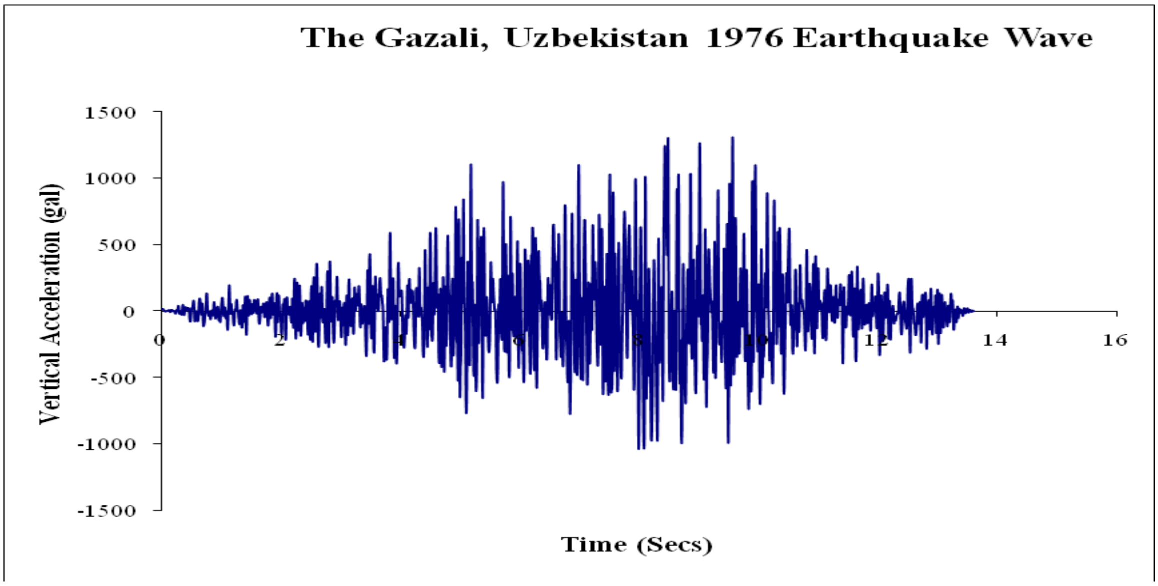


Fig4.15: Input Earthquake Wave (Vertical Acceleration)

As the time history analysis is a very time consuming task; to reduce the analysis time the duration of the time history has been truncated to 9.23 seconds for the analysis of different cases. The N-S component of horizontal acceleration & U- D component of vertical acceleration has been considered in the analysis.

Finite Element Mesh

Figure 4.16 shows the finite element mesh of the soil-structure interaction analysis model of the RC Bridge Pier.

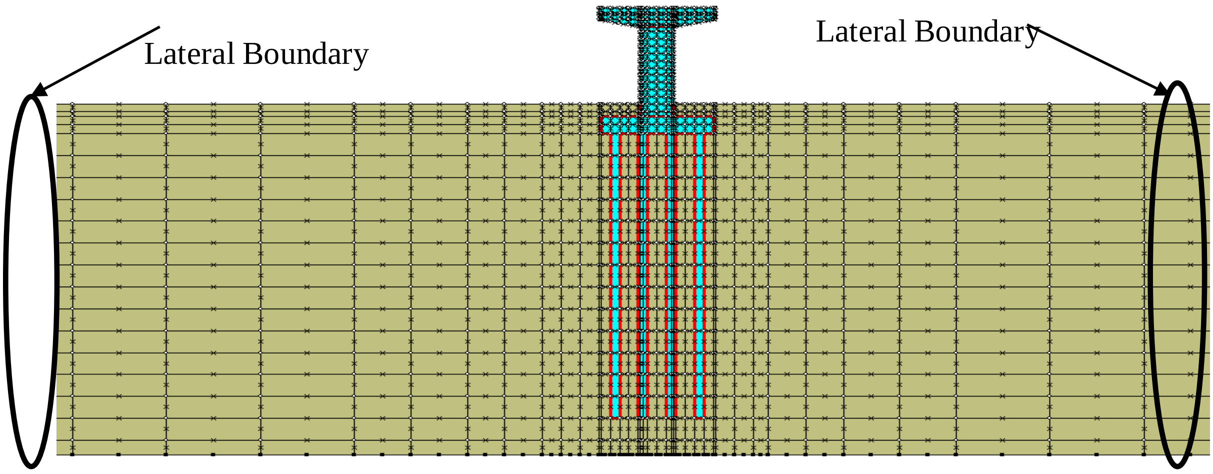


Fig 4.16: Finite Element Mesh (After 2000 Pile with Length 400m and Height 25m of Soil Mass)

Fig.4.18 shows the enlarged view of the lateral boundary condition used for simulating the far field boundary condition. The skewed dotted lines on top of the elements at the right side of the mesh in figure 4.17 are the overlapping plates. This overlapping plate represents the two different plates in depth direction. Figure 4.18 shows the enlarged view of Nepalese RC Bridge Pier. The green colored elements are RC elements; light brown colored elements are soil elements and the blue lines between the RC & Soil elements are joint elements.

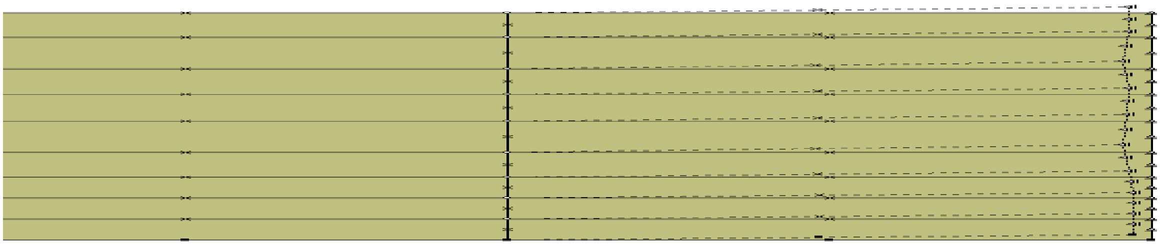


Fig.4.17: Overlapping Plates at the Lateral Boundary after 2000 Pile

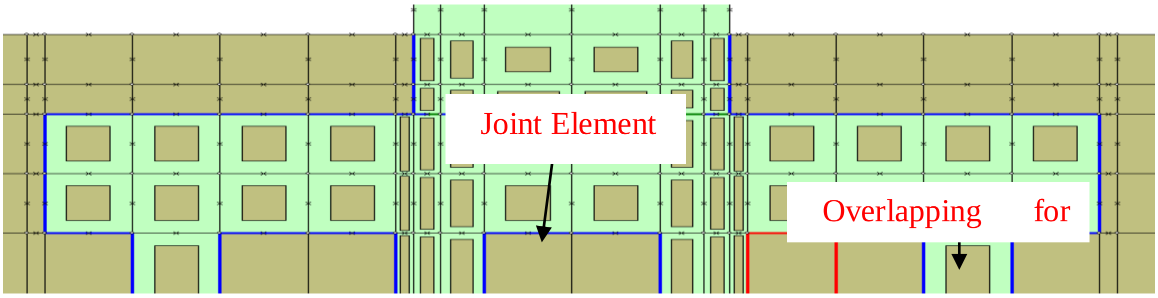


Fig. 4.18: Enlarged view of finite element mesh of the Bridge Pier after 2000 pile

Analysis Cases

Altogether 15 finite element models are constructed using the software WCOMD.

Three Condition of Nepalese RC Bridge Pier after 2000 Open

For the purpose of numerical simulation of the response of the Nepalese RC Bridge Pier, it is intended to consider three conditions of the mode (H, H+V, and H+ V with Scouring). These Three cases are given in the figure 4.19.

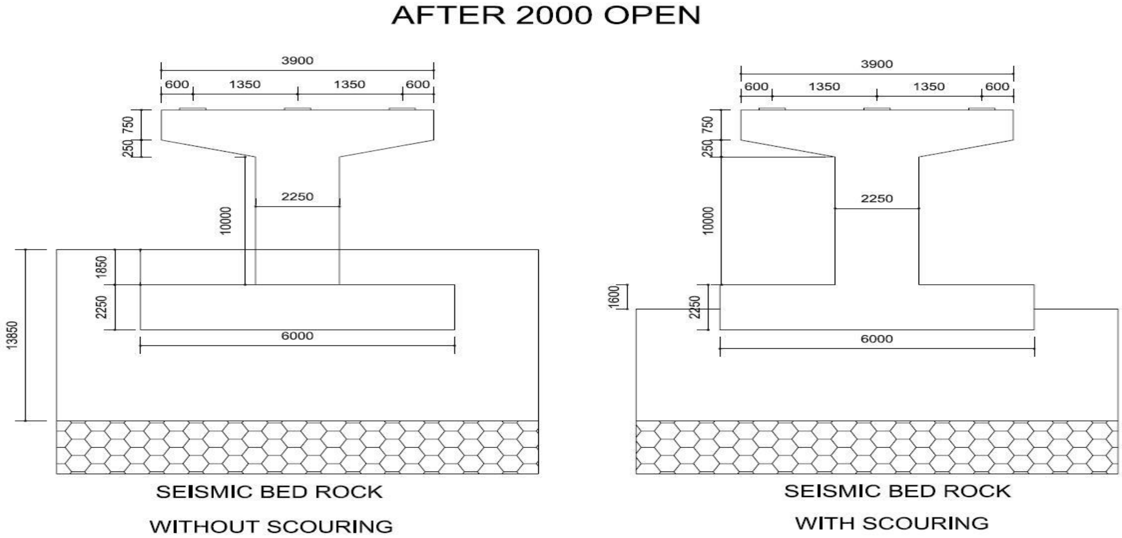


Fig. 4.19: Three Conditions of Soil Level after 2000 open (H, H+V and H+V with Scouring)

Three Condition of Nepalese RC Bridge Pier after 2000 Well

For the purpose of numerical simulation of the response of the Nepalese RC Bridge Pier, it is intended to consider three conditions of the mode (H, H+V, and H+ V with Scouring). These Three cases are given in the figure 4.20.

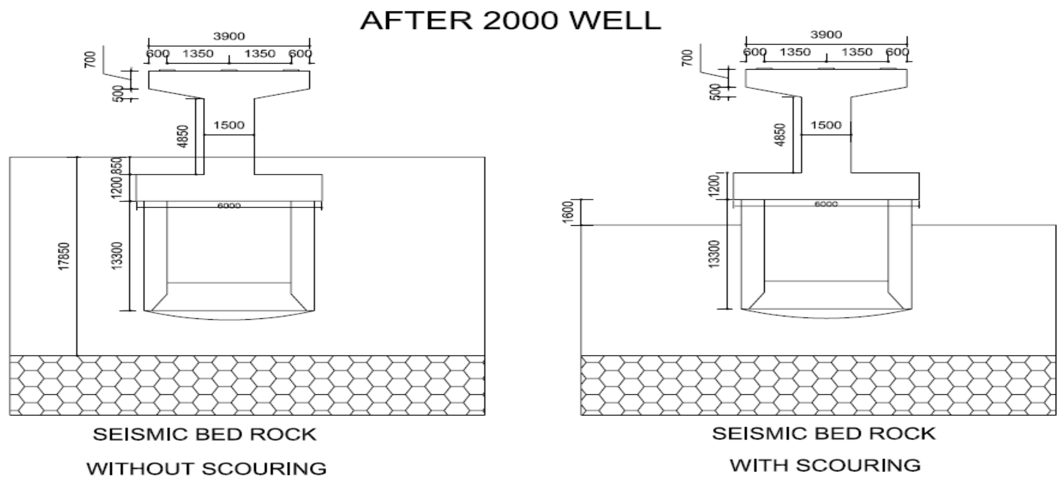


Fig.4.20: Three Conditions of Soil Level after 2000 well (H, H+V, H+V with Scouring)

Three Condition of Nepalese RC Bridge Pier after 2000 Pile

For the purpose of numerical simulation of the response of the Nepalese RC Bridge Pier, it is intended to consider three conditions of the mode (H, H+V, and H+ V with Scouring). These Three cases are given in the figure 4.21.

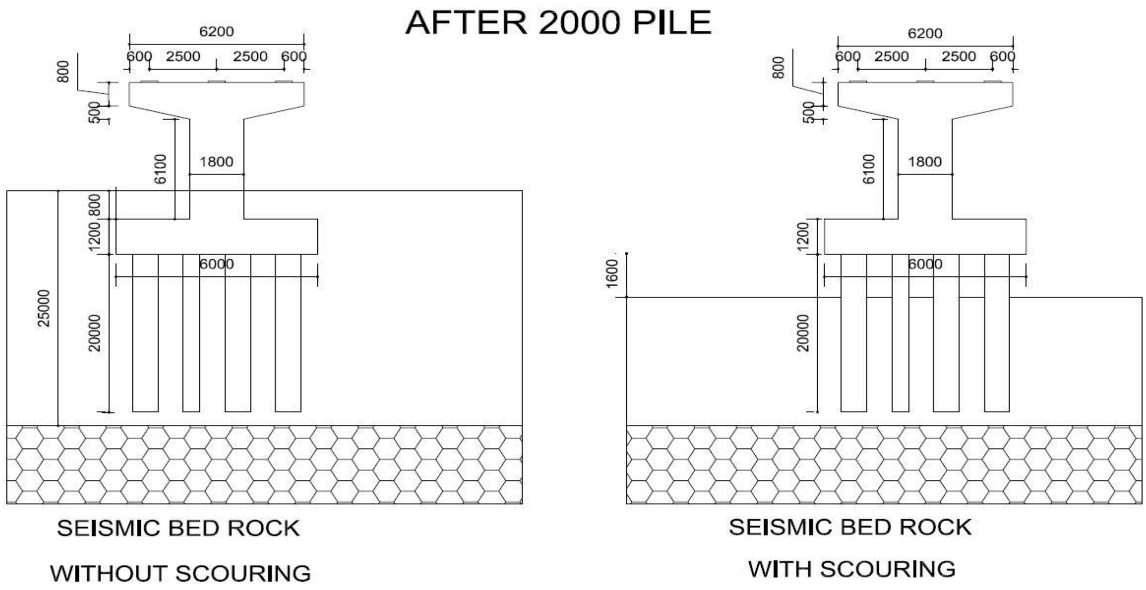


Fig. 4.21: Three Conditions of Soil Level after 2000 pile (H, H+V, and H+ V with Scouring)

Three Condition of Nepalese RC Bridge Pier 1990-2000

For the purpose of numerical simulation of the response of the Nepalese RC Bridge Pier, it is intended to consider two conditions of the soil level. These six cases are given in the figure 4.22

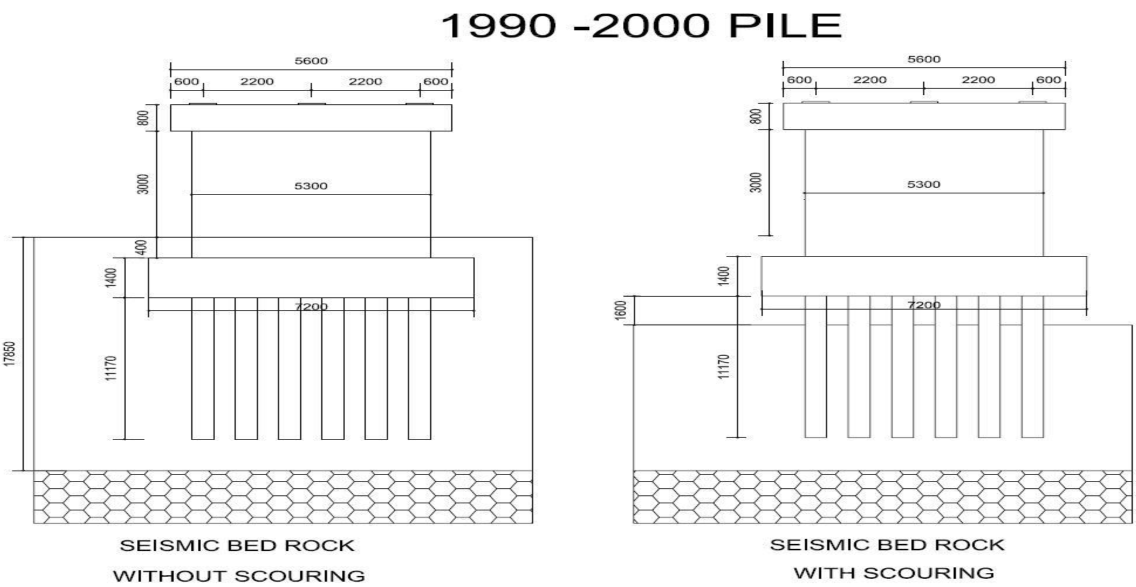


Fig. 4.22: Three Conditions of Soil Level 1990-2000 pile (H, H+V, and H+ V with Scouring)

Three Condition of Nepalese RC Bridge Pier after 2000

For the purpose of numerical simulation of the response of the Nepalese RC Bridge Pier, it is intended to consider two conditions of the soil level. These six cases are given in the figure 4.23.

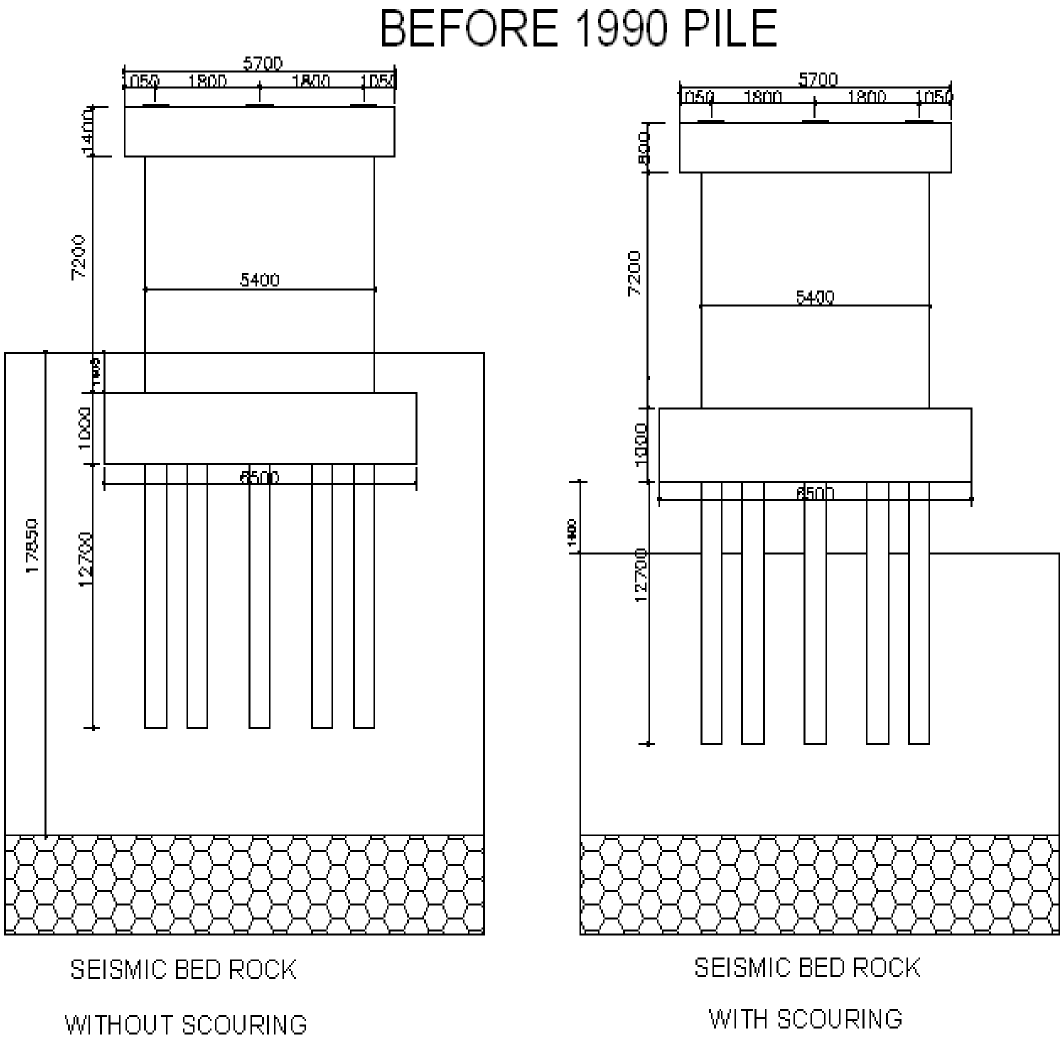


Fig.4.23: Three Conditions of Soil Level before 1990 pile (H, H+V, and H+ V with Scouring)

RESULTS AND DISCUSSION

General

All together fifteen analyses were conducted; among them three Nepalese RC Bridge pier are after 2000 open, three Nepalese RC Bridge pier are after 2000 well, three Nepalese RC Bridge pier are after 2000 pile, three Nepalese RC Bridge Pier are intermediate 1990-2000 pile and three Nepalese RC Bridge Pier are before 1990 pile. The results of these analyses are presented in this section. The performance of the Nepalese RC Bridge Pier is evaluated in terms of mean inelastic strain, the damage criteria and the deformation of the structure. For the evaluation of the results they have been grouped into three categories. First category consists of the results of horizontal strong ground motion applied. Second category consists of the results of horizontal and vertical strong ground motion applied. Third category consists of the results of scouring with horizontal & vertical strong ground motion applied.

Mean Inelastic Strain

(An, 1997; Maekawa et al., 2003)

Mean Inelastic Strain is used to represent the damage level of the whole RC structure. It is an index, which shows qualitatively the amount of damage incurred in the structure and the remaining amount of residual deformation after an earthquake. The first strain invariant (I_1) or the volumetric change of an element is closely related to the crack occurrence and expansion of the in-plane element. Thus, the first strain invariant (I_1) represents the damage level that occurs in an element. The mean inelastic strain (I) is defined as the spatial average of the first strain invariant (I_1) of all RC elements. The value is zero for elastic shear behavior under which no residual deformation exists. Hence, the mean inelastic strain is adopted here to represent the damage intensity of the whole RC structure. The first strain invariant and the mean inelastic strain are computed using the following equations:

$$I_1 = \varepsilon_{xx} + \varepsilon_{yy} = \delta V/V \qquad \text{Local}$$

$$I = \Sigma\, I_1\,(x, y)\; dx\, dy/A \qquad \text{Global}$$

Where ε_{xx} and ε_{yy} are the 2D principal strains at (x, y) and A is the total area of the RC in-plane elements.

Deviatoric Inelastic Strain

(An, 1997; Maekawa et al., 2003)

Deviatoric (Dev) Inelastic Strain is also used to represent the damage level of the whole RC structure. The strain value of deviatoric stress starts to drop due to the softening stresses created by the shear cracks propagation just before shear failure. The index is useful t o verify the shear failure mode. The Dev inelastic strain is computed using the following equations:

Strain $\gamma = \sum \sqrt{} (((\varepsilon + \varepsilon_{yy)} /2)^2) + \gamma_{xy} ^2) * dV/V$

Stress $\tau = \sum \sqrt{} (((\sigma_{xx} + \sigma_{yy)} /2)^2) + \tau_{xx} ^2) * dV/V$

Damage Criteria

Three types of damage criteria have been developed for judging the degree of damage: Failure criteria, Considerable damage criteria and Light damage criteria.

Failure: It is a state when reached, the structure cannot be repaired and the only alternative left is to demolish the structure. Failure is reached if any one of the following strain conditions is reached at any gauss point of a RC element:

Maximum tensile strain normal to crack, $\varepsilon_t = 3\%$

Maximum compressive strain parallel to crack, $\varepsilon_c = 1\%$

Maximum shear strain parallel to crack, $\varepsilon_{sh} = 2\%$

Considerable damage: It is a state of the structure where the structure can be repaired/ retrofitted. Considerable damage occurs when the peak compressive strain at any gauss point in RC element reaches the value equal to the uniaxial compressive strength of the concrete multiplied by the magnification factor, i.e.,

$$\varepsilon_c = \alpha\ \varepsilon_{peak}$$

The following relation gives the value of the ε_{peak},

$$\varepsilon_{peak} = 447.2 \sqrt{f_c'}\ (N/mm^2)\ X\ 10^{-6}$$

The value of α is taken as 1.5

Light damage: It is also a state of the structure in which the structure can be repaired/ retrofitted. Light damage occurs when the peak tensile strain normal to the crack reaches a value of 0.1%, i.e., $\varepsilon_t = 0.1\%$

Results of Nepalese RC Bridge Pier

To evaluate the performance of the Nepalese RC Bridge Pier, three cases of five Bridges model each of them are H, H+ V and H+V with Scouring were considered. The performance is expressed in terms of mean inelastic strain level, damage level and variation of stress & strain in time domain.

Mean Inelastic Strain

The mean inelastic strain in time domain is plotted for the three cases of after 2000 Open , three cases of after 2000 Well , three cases of after 2000 Pile, three cases of intermediate 1990-2000 and three cases of before 1990 shown in Appendix.

Dev Inelastic Strain

The Dev elastic strain in time domain is plotted for the three cases of after 2000 Open, three case of after 2000 Well, three case of after 2000 Pile , , three cases of intermediate 1990-2000 and three cases of before 1990 shown in Appendix.

Damage Level

The five bridges (After 2000 open, After 2000 Well , After 2000 Pile , Intermediate 1990 - 2000 Pile and Before 1990 Pile) of different three cases (H, H+V, H+V with Scouring) damage location is shown in fig.5.31.,5.32 and also tabular form in table

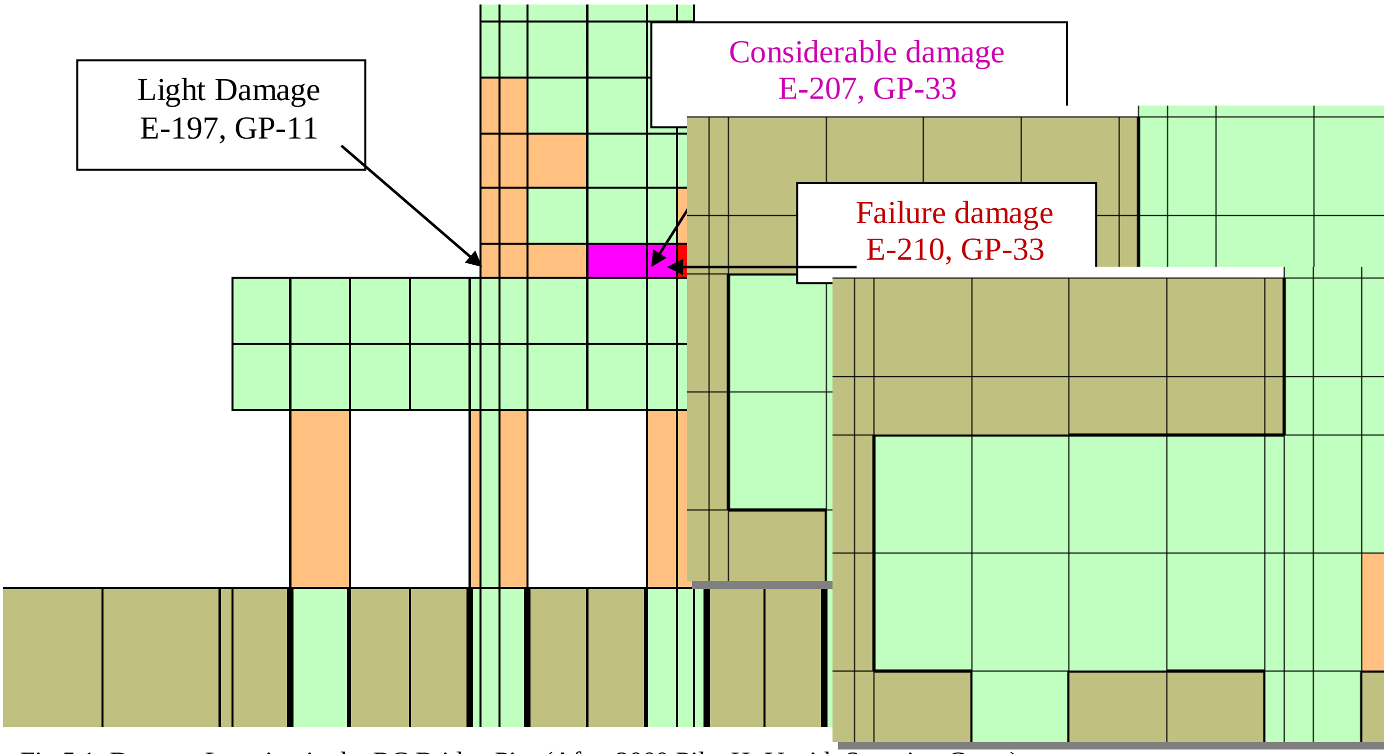


Fig.5.1: Damage Location in the RC Bridge Pier (After 2000 Pile, H+V with Scouring Cases)

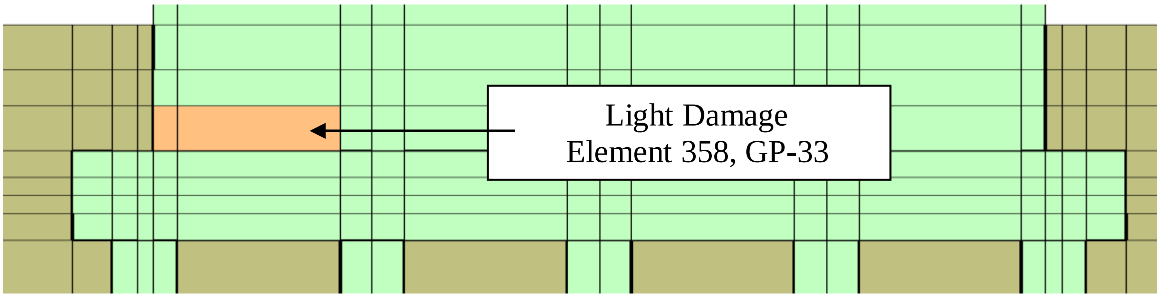


Fig.5.2: Damage Location in the RC Bridge Pier (Before 1990 Pile, H+V Cases)

From fig.5.1 & 5.2, it is clear that after scouring (After 2000 Pile), the bridge pier light damage, considerable damage and failure damage occurred and also pile foundation display light damage. It means scouring is effect the whole structure.

Deformation, Crack Pattern and Yield Locations

Figure 5.3 shows the maximum deformation profile of the base case model. In the figure we can clearly see the separation between the soil and the structure at various interface locations along the periphery of the structure. Figure 5.3 shows the crack.

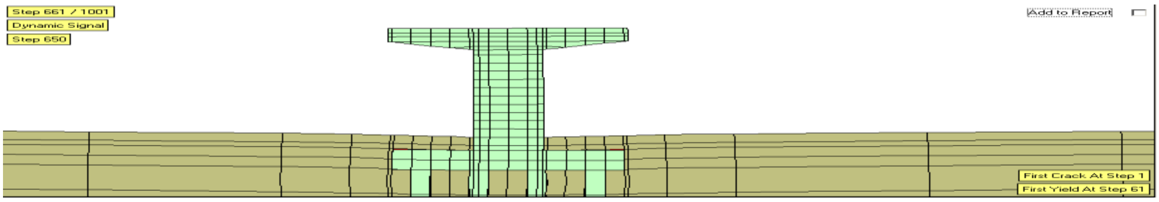


Fig.5.3: Deformation in the structure (After 2000 Pile)

Table: 5.1 First Yield Result

First Yield Result					
S.N.	**Description**	**Types of earthquake**	**Element /GP**	**Strain X**	**Strain Y**
1	After 2000 open	Scouring H+V	302/23	5.07E-04	-1.97E-03
2	After 2000 Pile	H	**309/31**	-1.86E-04	2.55E-05
		H+V	370/31	-1.75E-04	2.32E-05
		Scouring H+V	191/31	-1.74E-04	2.34E-05
3	After 2000Well	H	449/23	2.27E-04	-8.70E-06
		H+V	449/23	2.27E-04	-8.70E-06
		Scouring H+V	397/23	2.61E-04	-5.49E-06
4	Before 1990	H	378/23	5.13E-06	-1.77E-04
		H+V	378/23	5.13E-06	-1.77E-04
		Scouring H+V	253/23	5.16E-06	-1.77E-04

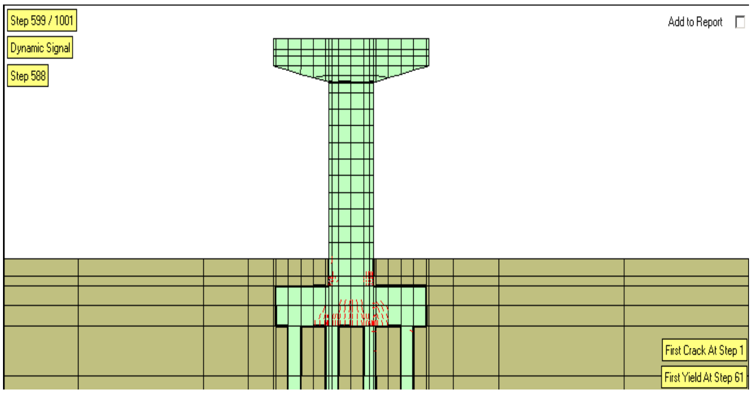


Fig.5.4: Crack in the structure (After 2000 Pile)

Table: 5.2 First Crack Result

First Crack Result								
S.N.	**Description**	**Types of earthquake**	**Element /Gauss Point**	**Mean normal strain**		**Mean shear strain**		**Crack angle (deg)**
				E_x	**E_y**	**G_{xx}**	**G_{xy}**	
1	After 2000 open	H	175/23	**2.49E-04**	-1.88E-05	-5.93E-06	-5.93E-06	90
		H+V	175/23	**2.49E-04**	-1.89E-05	-6.06E-06	-6.06E-06	90
		Scouring H+V	175/23	**2.61E-04**	-1.79E-05	-1.23E-05	-1.23E-05	90
2	After 2000 Pile	H	234/23	**6.81E-04**	-2.80E-06	-7.02E-05	-7.02E-05	95
		H+V	234/23	**5.77E-04**	-3.54E-06	-4.91E-05	-4.91E-05	95
		Scouring H+V	187/33	9.69E-05	-2.51E-05	-	-3.90E-05	101
3	After 2000Well	H	448/23	**1.53E-04**	-4.93E-06	-	-1.32E-05	91
		H+V	401/23	3.35E-04	-3.81E-06	**3.14E-06**	**3.14E-06**	90
		Scouring H+V	396/23	**1.50E-04**	-3.89E-06	-	-1.01E-05	91
4	Inter 1990 to 2000	H	708/23	**1.11E-04**	-4.84E-06	-	-1.08E-05	109
		H+V	712/23	**1.13E-04**	-4.23E-06	-	1.15E-05	70
		Scouring H+V	452/31	**1.08E-04**	-6.30E-06	-	-8.81E-07	98
5	Before 2000	H	378/23	**2.85E-05**	-7.78E-06	-	-1.13E-05	16
		H+V	418/23	-8.92E-06	**2.85E-06**	-	9.28E-06	4
		Scouring	169/33	**1.07E-**	-2.26E-	-	2.85E-06	84

		H+V		**04**	05			

Structural Result

In order to investigate the variation of the stress and strain parameters in time domain, an element, which experienced light damage, has been chosen.

Result taken of different bridges of different cases (Node, Element & Gauss Point)

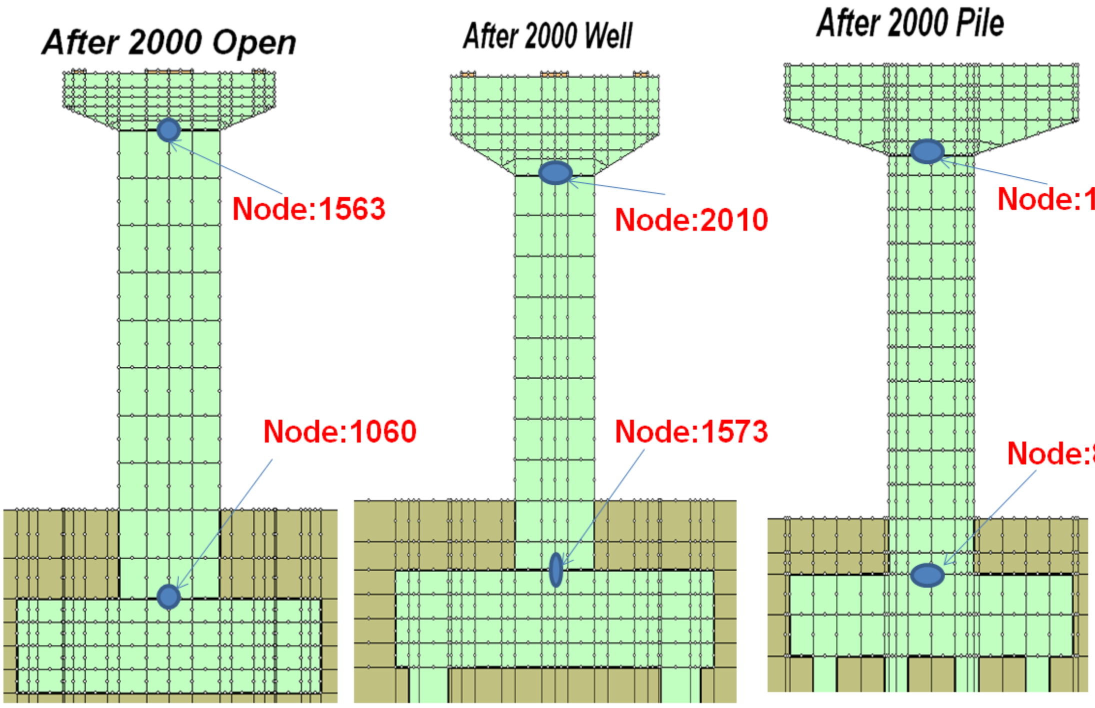


Fig.5.5: Location of Node Number After 2000 Open, Well and Pile

Result taken of different bridges of different cases (Node, Element & Gauss Point)

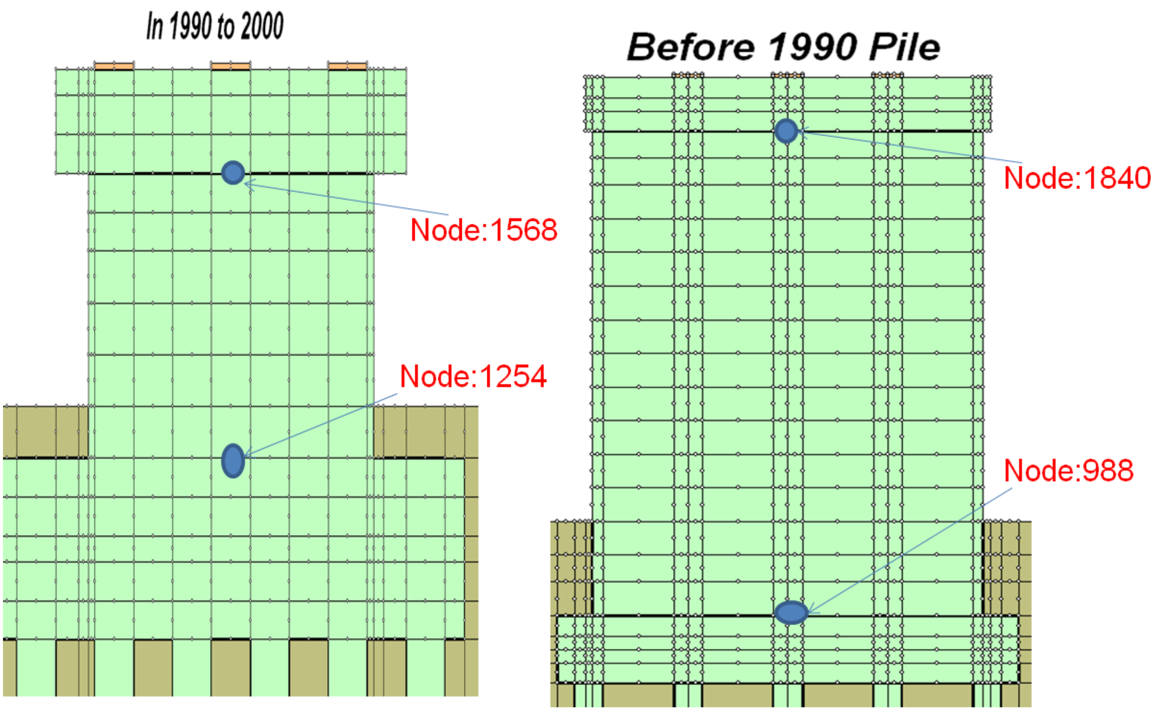


Fig.5.6: Location of Node Number in 1990-2000 Pile and Before 1990 Pile

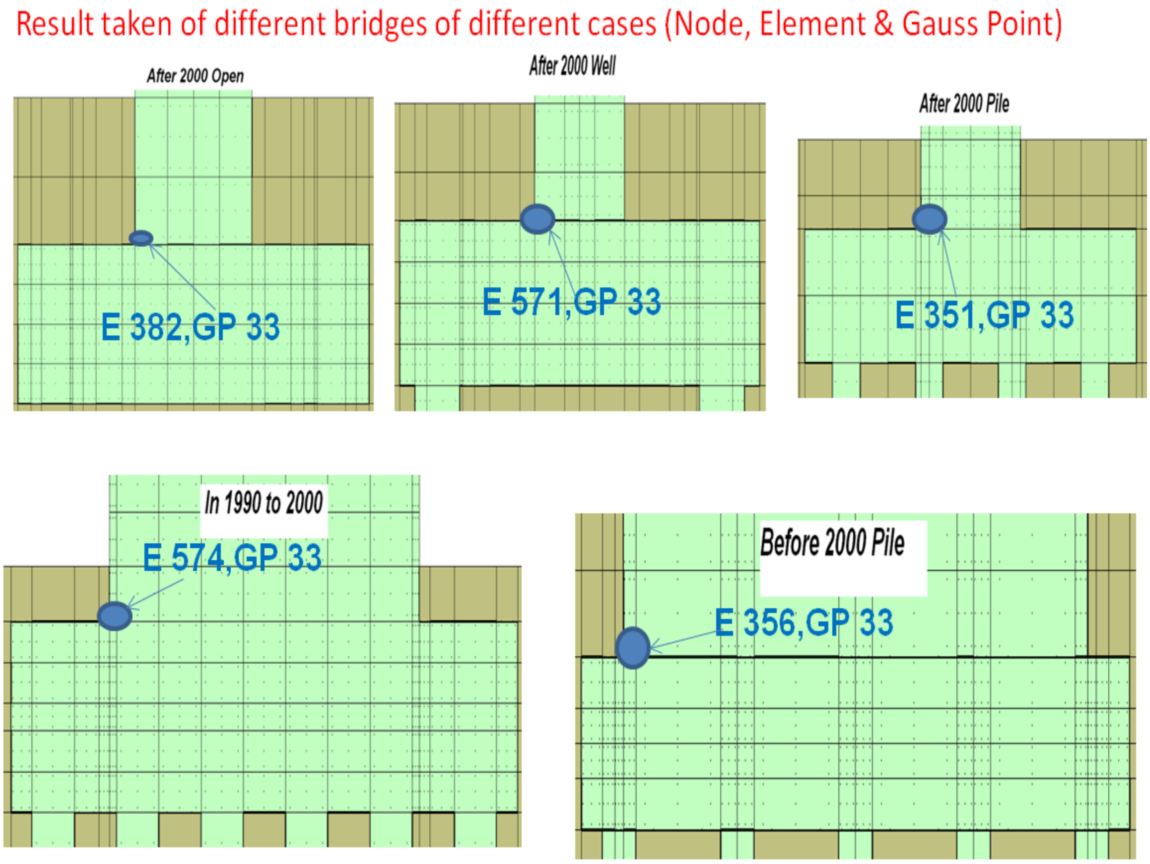


Fig.5.7: Location of Element & Gauss Point (Five Nepalese RC Bridge Pier)

Table: 5.3 Length of Soil (Five Nepalese RC Bridge Pier)

After 2000 Open				400 m AO	500 m AW	400m AP	500 m IP	400m BP
Description	300	350	400	%dif.	% dif.	% dif.	%dif.	% dif.
Horizontal (Top) (cm)	6.72	6.68	6.659	0.15	0.02	0.08	0.06	0.1
Vertical(Bottom)(cm)	1.62	1.51	1.5	0.33	0.1	0.46	0.31	0.16
Peak HA(cm/sec2)	1690	1680	1675	0.15	0.05	0.01	0.02	0.25
Peak VA(cm/sec2)	79	76	75	0.34	0.25	0.23	0.2	0.4
Strian XX	0	4E-05	4E-05	0	0	0	0	0
Strain YY	0	3E-04	3E-04	0	0	0	0	0
Strian XY	0	7E-05	7E-05	0	0	0	0	0
Mean Strain	0	7E-06	7E-06	0	0	0	0	0
Deviation Strain	0	3E-05	3E-05	0	0	0	0	0

Table: 5.4 Structure Result after 2000 Open

S.N.	Description		H	HV	HVS
1	Displacement	Horizontal (Top) N1563(cm)	6.659	7.288	13.47
		Vertical(Bottom) N1060(cm)	1.5	2.702	2.37
2	Relative Acceleration	Horizontal acceleration	1052	1096	1224
	N1563 ,N 1060(cm/sec2)	Vertical accleration	378	2012	3272
		Strian XX	3.64E-05	-0.00011	0.000059
3	Gauss Point Result	Strain YY	-0.000341	0.000618	-0.001224
	E382, GP33	Strian XY	-6.97E-05	0.00069	-0.000319
4	Global Stain Result	Mean Strain	-7.25E-06	-2.2E-05	-2.32E-05
		Deviation Strain	2.69E-05	5.29E-05	6.89E-05
	Structural Result	Peak shear strain			
	Peak Stain damage	Peak Tensile Strain			**1.01E-03**
5	Light damage	Peak Compressive Strain			**-4.98E-03**
	Considerable damage	Tensile strain normal to crak			
	Failure damage	Compressibe strain parallel to Crack			
		Shear Strain paralled to Crack			

Table: 5.5 Structure Result after 2000 Well

S.N.	Description		H	HV	HVS
1	Displacement	Horizontal (Top) N2010(cm)	**28.76**	**25.91**	**29.28**
		Vertical(Bottom) N1573(cm)	**1.41**	**6.04**	**5.49**
2	Relative Acceleration	Horizontal acceleration	893.1	1163	1370
	N2010 ,N1573(cm/sec2)	Vertical accleration	533	1695	2726
		Strian XX	1.61 x1.0e-5	-12.04 x1.0e-5	5.01 x1.0e-5
3	Gauss Point Result	Strain YY	-12.70 x1.0e-5	-63.57 x1.0e-5	-48.69 x1.0e-5
	E571,GP33	Strian XY	-26.14 x1.0e-6	-22.98 x1.0e-5	-16.78 x1.0e-5
4	Global Stain Result	Mean Strain	-23.03 x1.0e-6	15.07 x1.0e-5	4.45 x1.0e-5
		Deviation Strain	67.59 x1.0e-6	18.75 x1.0e-5	10.50 x1.0e-5
	Structural Result	Peak shear strain			
	Peak Stain damage	Peak Tensile Strain	**1.34E-03**	3.69E-03	2.56E-03
5	Light damage	Peak Compressive Strain			**-5.08E-04**
	Considerable damage	Tensile strain normal to crak			**1.30E-03**
	Failure damage	Compressibe strain parallel to Crack			
		Shear Strain paralled to Crack			

Table: 5.6 Structure Result after 2000 Pile

S.N.	Description		H	HV	HVS
1	Displacement	Horizontal (Top) N1347(cm)	25.88	13.67	23.35
		Vertical(Bottom) N850(cm)	5.39	9.17	9.46
2	Relative Acceleration	Horizontal acceleration	1052	1096	1224
	N1347,N850(cm/sec2)	Vertical accleration	986	2312	3272
		Strian XX	1.87 x1.0e-4	4.14 x1.0e-4	1.86 x1.0e-4
3	Gauss Point Result	Strain YY	-12.06 x1.0e-4	-11.36 x1.0e-4	21.14 x1.0e-4
	E351,GP33	Strian XY	-28.52 x1.0e-5	42.75 x1.0e-5	-16.14 x1.0e-4
4	Global Stain Result	Mean Strain	-41.24 x1.0e-6	-72.45 x1.0e-6	-6.92 x1.0e-5
		Deviation Strain	65.90 x1.0e-6	81.61 x1.0e-6	11.56 x1.0e-5
	Structural Result	Peak shear strain			
	Peak Stain damage	Peak Tensile Strain	1.06E-03	1.22E-03	1.41E-03
5	Light damage	Peak Compressive Strain			-9.74E-03
	Considerable damage	Tensile strain normal to crak			-1.62E-02
	Failure damage	Compressibe strain parallel to Crack			-8.46E-03
		Shear Strain paralled to Crack			-2.29E-02

Table: 5.7 Structure Result in 1990-2000 Pile

S.N.	Description		H	HV	HVS
1	Displacement	Horizontal (Top) N1568(cm)	18.8	18.72	19.023
		Vertical(Bottom) N1254(cm)	0.71	2.71	3.666
2	Relative Acceleration	Horizontal acceleration	967.5	1325	1953
	N1568,N1254 (cm/sec2)	Vertical accleration	44.3	1438	2811
		Strian XX	-0.39 x1.0e-5	5.98 x1.0e-5	1.84 x1.0e-5
3	Gauss Point Result	Strain YY	10.81 x1.0e-5	83.16 x1.0e-5	-24.24 x1.0e-5
	E574,GP33	Strian XY	51.14 x1.0e-6	65.96 x1.0e-5	-11.67 x1.0e-5
4	Global Stain Result	Mean Strain	-21.67 x1.0e-6	-46.82 x1.0e-6	-39.24 x1.0e-6
		Deviation Strain	61.94 x1.0e-6	70.14 x1.0e-6	50.36 x1.0e-6
	Structural Result	Peak shear strain			
	Peak Stain damage	Peak Tensile Strain		No damage	
5	Light damage	Peak Compressive Strain			
	Considerable damage	Tensile strain normal to crak			
	Failure damage	Compressibe strain parallel to Crack		No failure	
		Shear Strain paralled to Crack			

Table: 5.8 Structure Result before 1990 Pile

S.N.	Description		H	HV	HVS
1	Displacement	Horizontal (Top) N1840(cm)	23.91	24.01	30.16
		Vertical(Bottom) N988(cm)	1.07	2.75	3.23
2	Relative Acceleration	Horizontal acceleration	967.5	1325	1453
	N1840,N988 (cm/sec2)	Vertical accleration	44.3	1438	1811
		Strian XX	8.83 x1.0e-6	-1.74 x1.0e-4	3.37 x1.0e-5
3	Gauss Point Result	Strain YY	-49.36 x1.0e-6	13.01 x1.0e-4	-21.93 x1.0e-5
	E356 ,G33	Strian XY	41.76 x1.0e-7	33.14 x1.0e-5	-48.16 x1.0e-6
4	Global Stain Result	Mean Strain	-21.67 x1.0e-6	-46.82 x1.0e-6	-39.24 x1.0e-6
		Deviation Strain	61.94 x1.0e-6	70.14 x1.0e-6	50.36 x1.0e-6
	Structural Result	Peak shear strain			
	Peak Stain damage	Peak Tensile Strain		1.22E-03	3.69E-03
5	Light damage	Peak Compressive Strain			
	Considerable damage	Tensile strain normal to crak			
	Failure damage	Compressibe strain parallel to Crack			
		Shear Strain paralled to Crack			

Peak acceleration

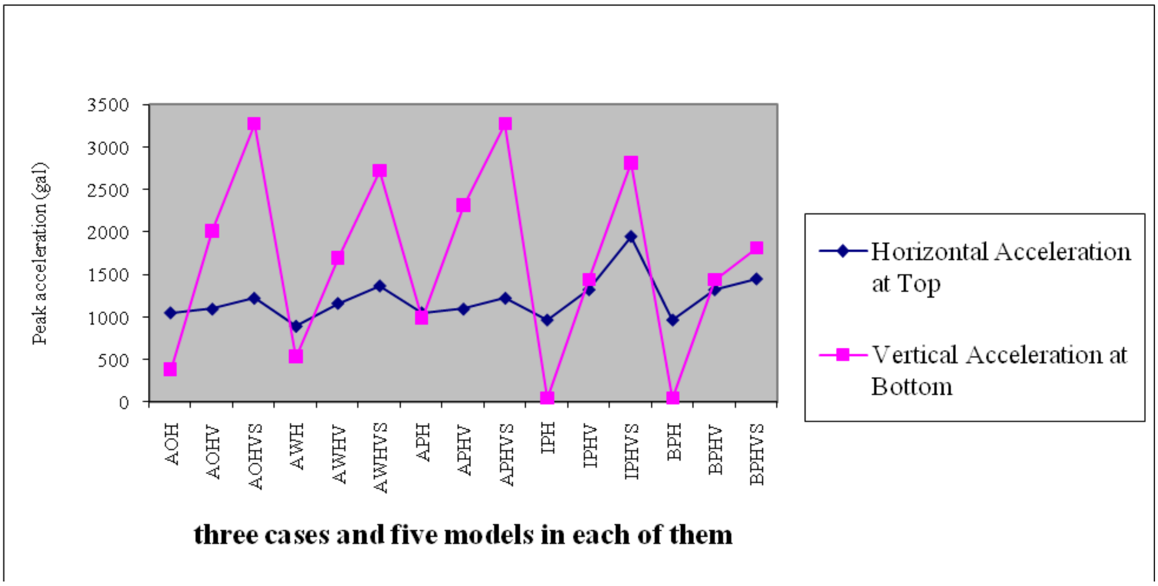


Fig.5.8: Peak Acceleration (cm/sec2)

The largest acceleration horizontal at top (IPHVS Case) and vertical at bottom (APHVS Case)

Absolute displacement

Figure 5.8 shows the relative displacement between the top and bottom of the structure of the after 2000 Open, Well & Pile, intermediate 1990-2000 Pile and before 1990 Pile.

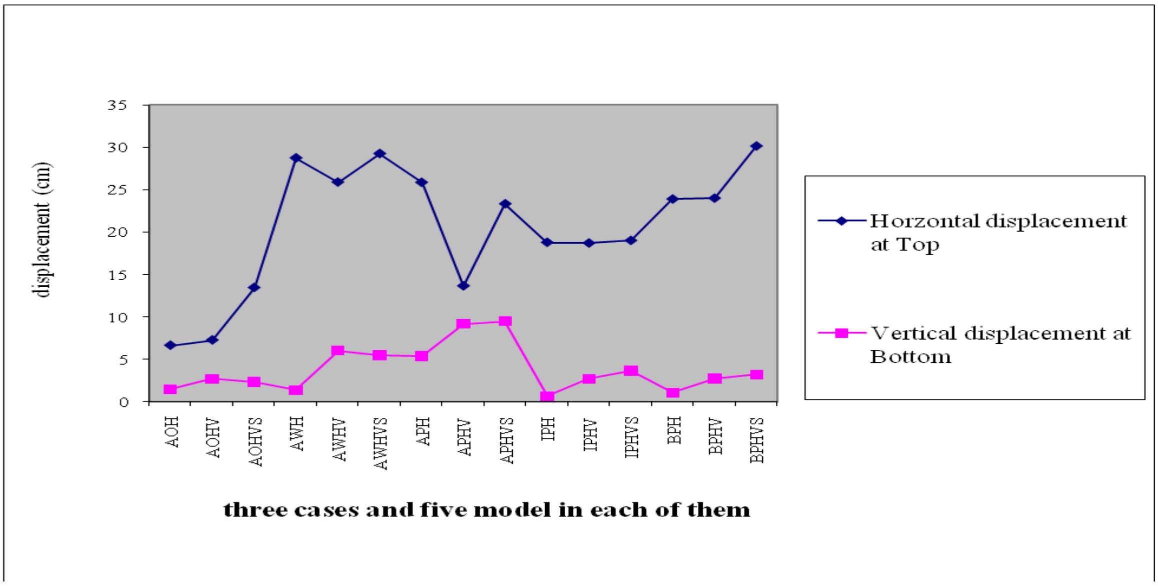


Fig.5.8: Peak displacement (cm)

The absolute displacement is first largest for the case AWHVS and second largest for the case BPHVS.

CONCLUSIONS

This research work has been carried out to access the seismic performance of the typical Nepalese RC Bridge pier structures. The severe damage and possibility of collapse of the RC Bridge piers structures with very strong coupled vertical ground motion Gazali, Uzbekistan 1976.The conclusions drawn from this research is as follows.

1. The lateral extent of soil mass for each of the five different bridges were fixed based on trial computations until the percentage difference in consecutive response quantities were found within the limit of 0.4%. The lateral extent for the Bridge 1 (After 2000 Open) is fixed as 400 m ,Bridge 2 (After 2000 Well) is fixed as 500 m, Bridge 3 (After 2000 Pile) is fixed as 400 m , Bridge 4 (Inter. 1990-2000 Pile) is fixed as 500 m and Bridge 5 (Before 1990 Pile) is fixed as 400 m.

2. The light damage, considerable damage and also failure damage of the Bridge pier at different bridges for different cases (After 2000 Open, After 2000 Pile, After 2000 Well and Before 1990).
3. The introduction of strong vertical ground motion increase the vertical displacement by 48 % all bridges. It means horizontal ground acceleration (0.71g) and vertical ground acceleration (1.37g) both applied horizontal displacement increases as well as vertical displacement increases.
4. First crack all bridge for different cases
5. First Yield Crack Bridge (After 2000 Open, After 2000 Well, After 2000 Pile and Before 1990 Pile).
6. The Nepalese RC Bridge Pier (In 1990-2000) is over safe.
7. The Nepalese RC Bridge Pier (After 2000 & Before 1990) is designed under safe.

RECOMMENDATIONS FOR FUTURE WORKS

1. The research has been carried out on only five bridge of country; Study can be conducted on more bridges of country.

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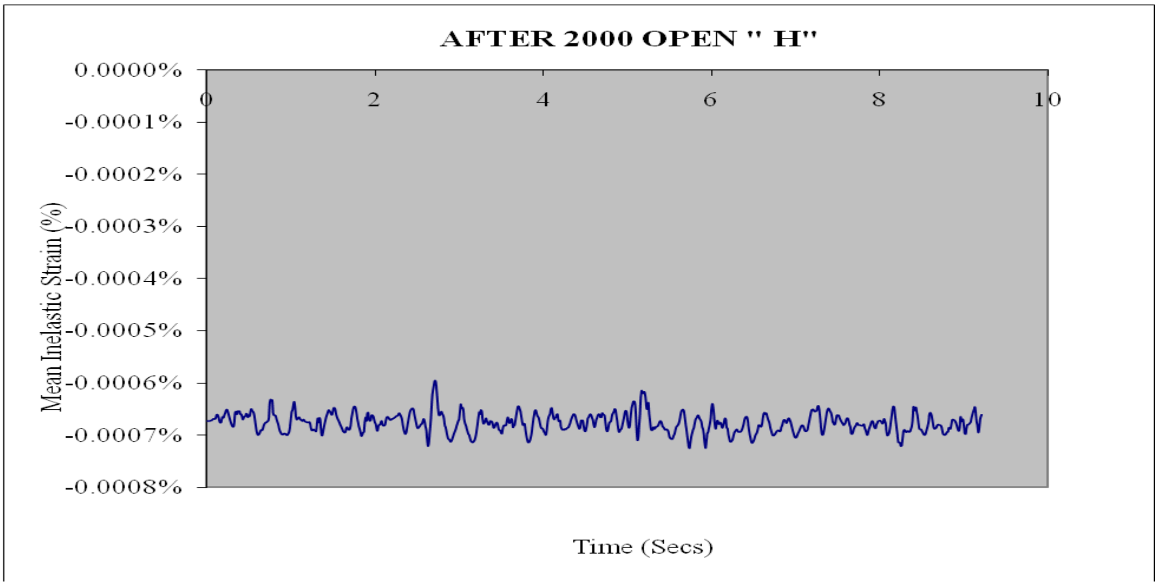
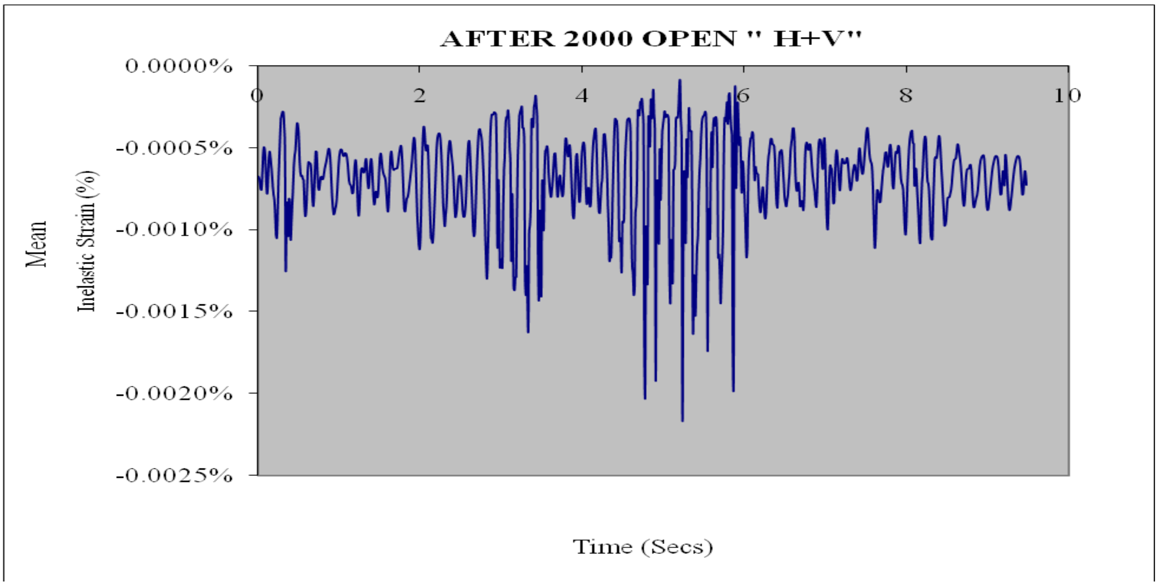
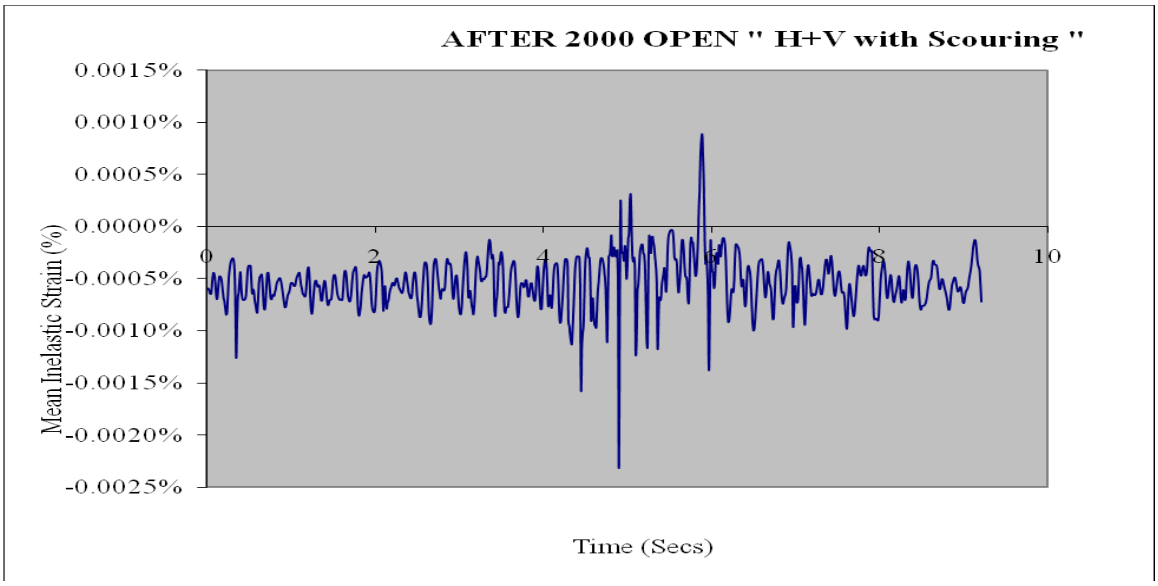
APPENDIX

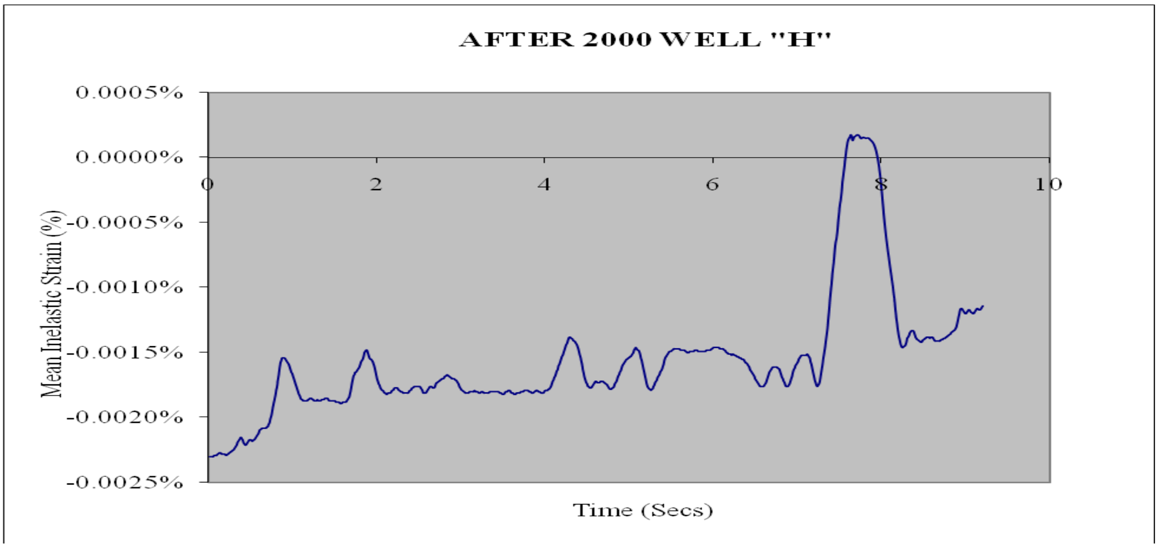


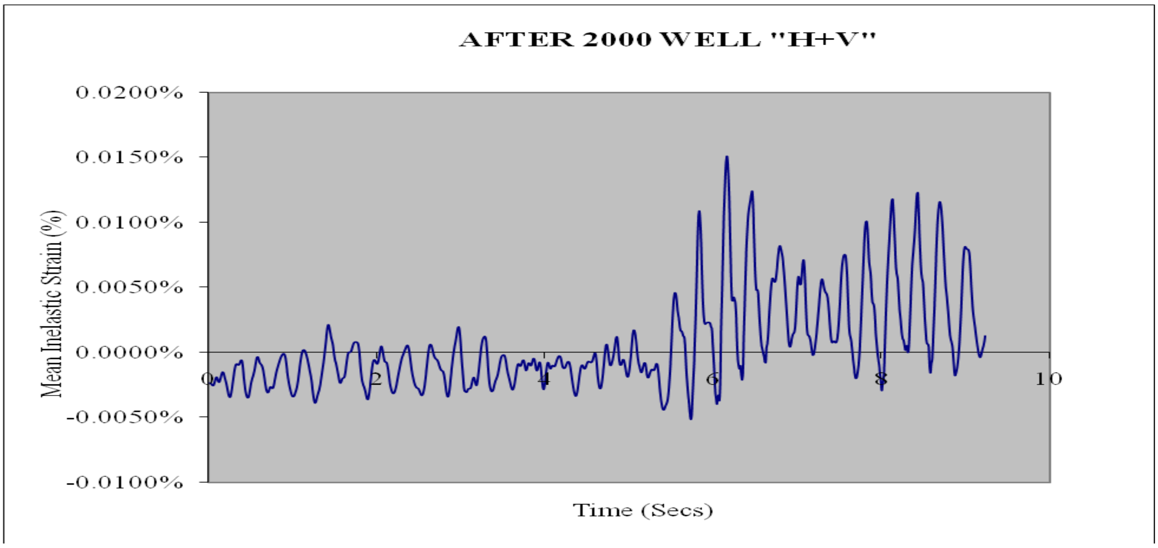
AFTER 2000 OPEN " H"
0.0000%
-0.0001%
-0.0002%
-0.0003%
-0.0004%
-0.0005%
-0.0006%
-0.0007%
-0.0008%
0
2
4
6
8
10
Mean Inelastic Strain (%)
Time (Secs)

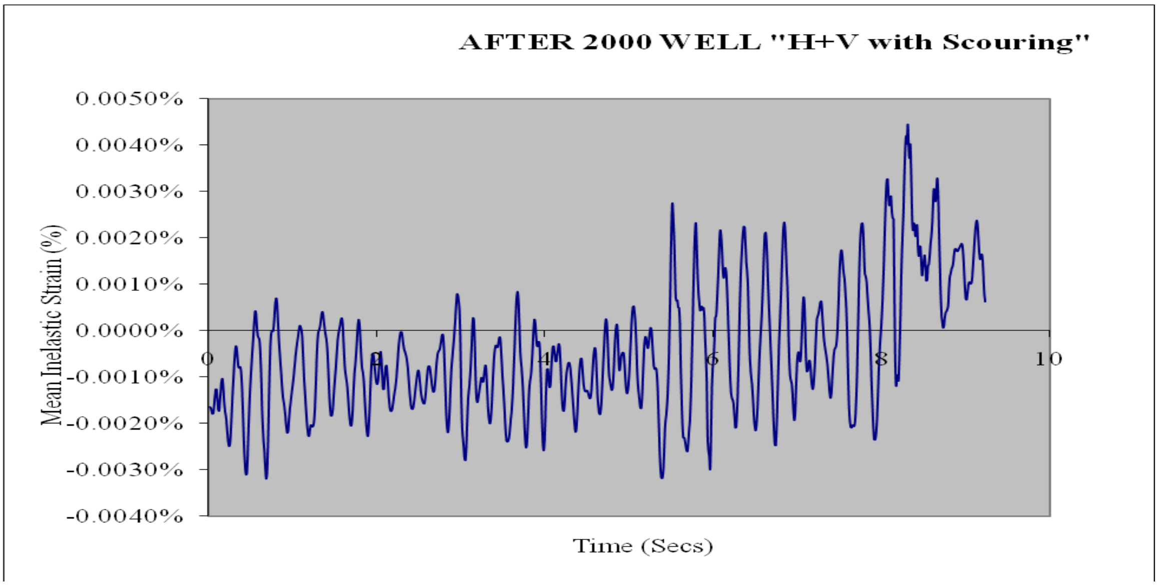


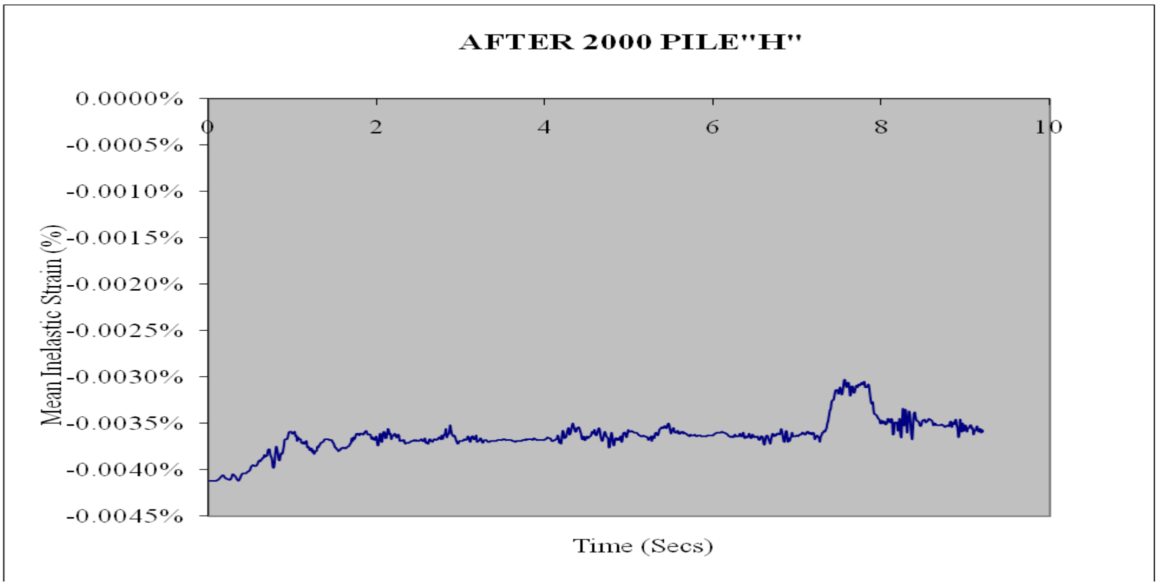
AFTER 2000 OPEN " H+V"
0.0000%
-0.0005%
-0.0010%
-0.0015%
-0.0020%
-0.0025%
0
2
4
6
8
10
Mean
Inelastic Strain (%)
Time (Secs)

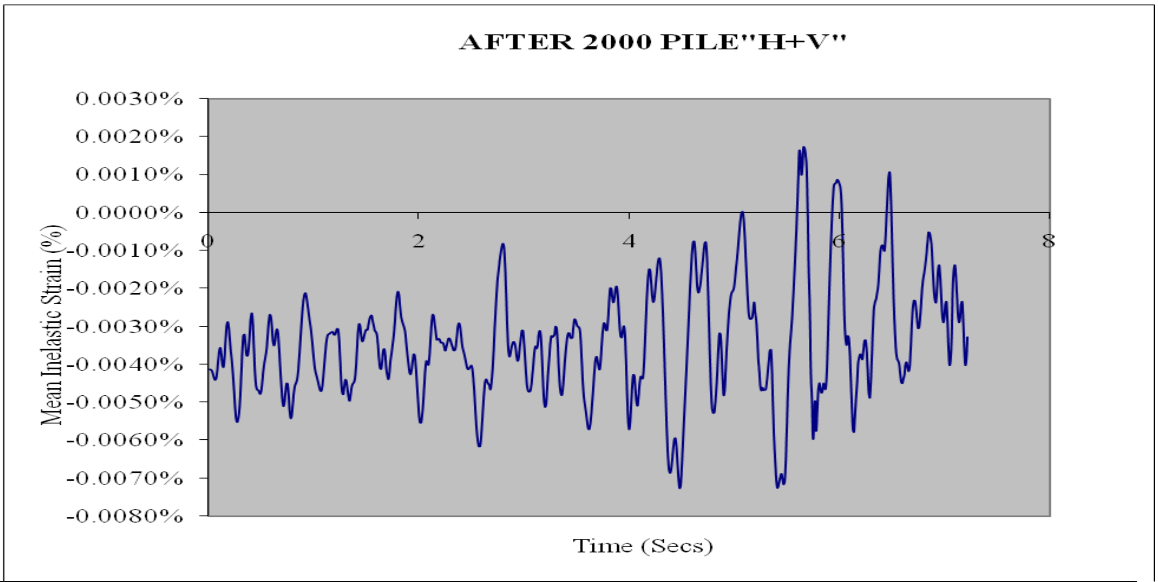

AFTER 2000 OPEN " H+V with Scouring "
0.0015%
0.0010%
0.0005%
0.0000%
-0.0005%
-0.0010%
-0.0015%
-0.0020%
-0.0025%
0
2
4
6
8
10
Mean Inelastic Strain (%)
Time (Secs)

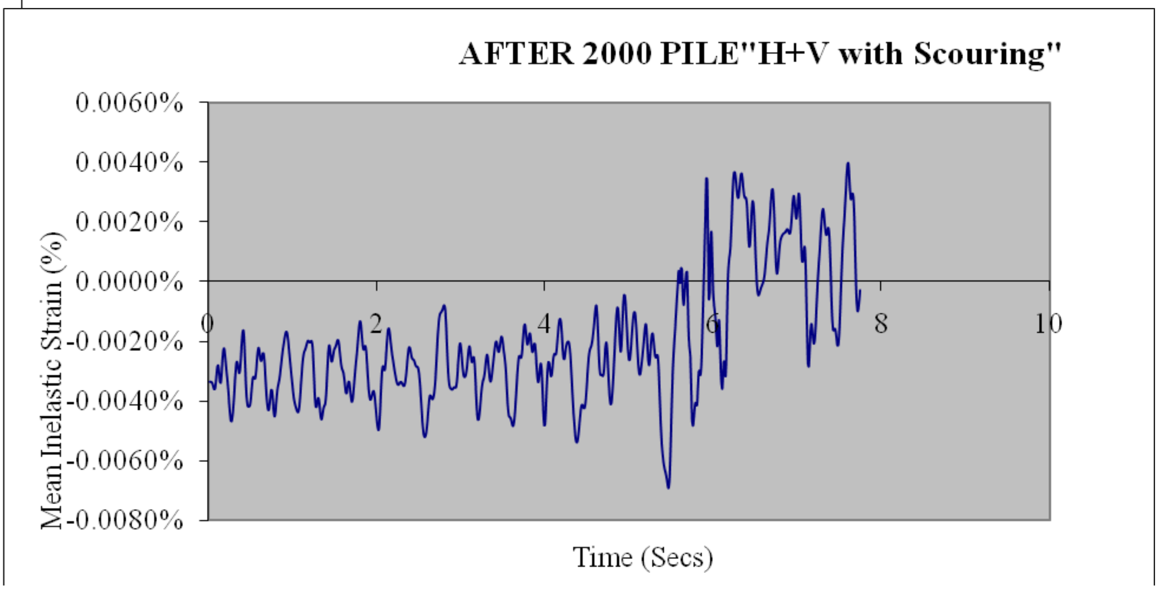

AFTER 2000 WELL "H"
0.0005%
0.0000%
-0.0005%
-0.0010%
-0.0015%
-0.0020%
-0.0025%
0
2
4
6
8
10
Mean Inelastic Strain (%)
Time (Secs)

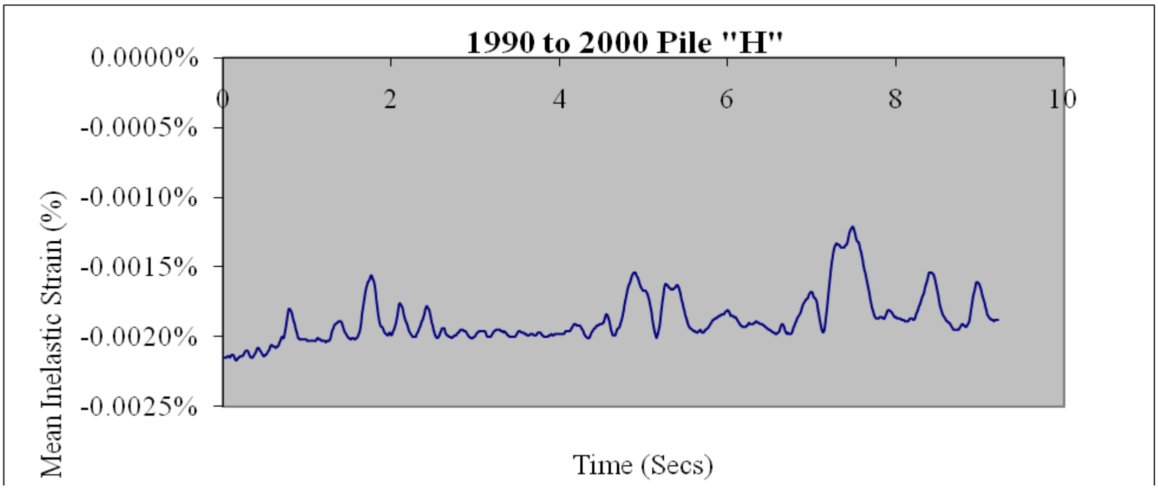

AFTER 2000 WELL "H+V"
0.0200%
0.0150%
0.0100%
0.0050%
0.0000%
-0.0050%
-0.0100%
0
2
4
6
8
10
Mean Inelastic Strain (%)
Time (Secs)

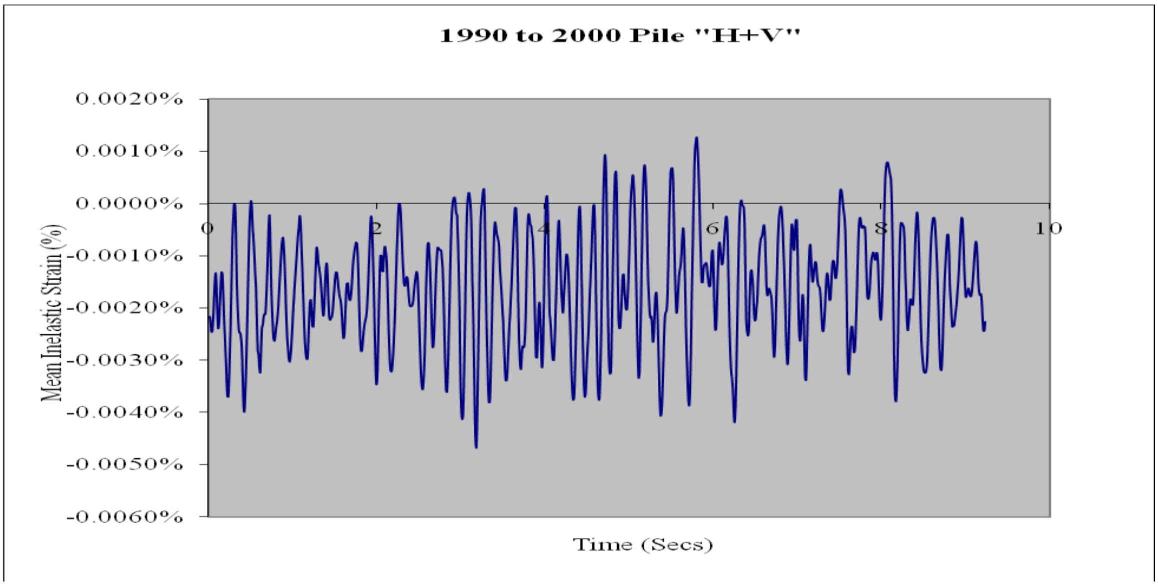

AFTER 2000 WELL "H+V with Scouring"
Mean Inelastic Strain (%)
0.0050%
0.0040%
0.0030%
0.0020%
0.0010%
0.0000%
-0.0010%
-0.0020%
-0.0030%
-0.0040%
0
2
4
6
8
10
Time (Secs)

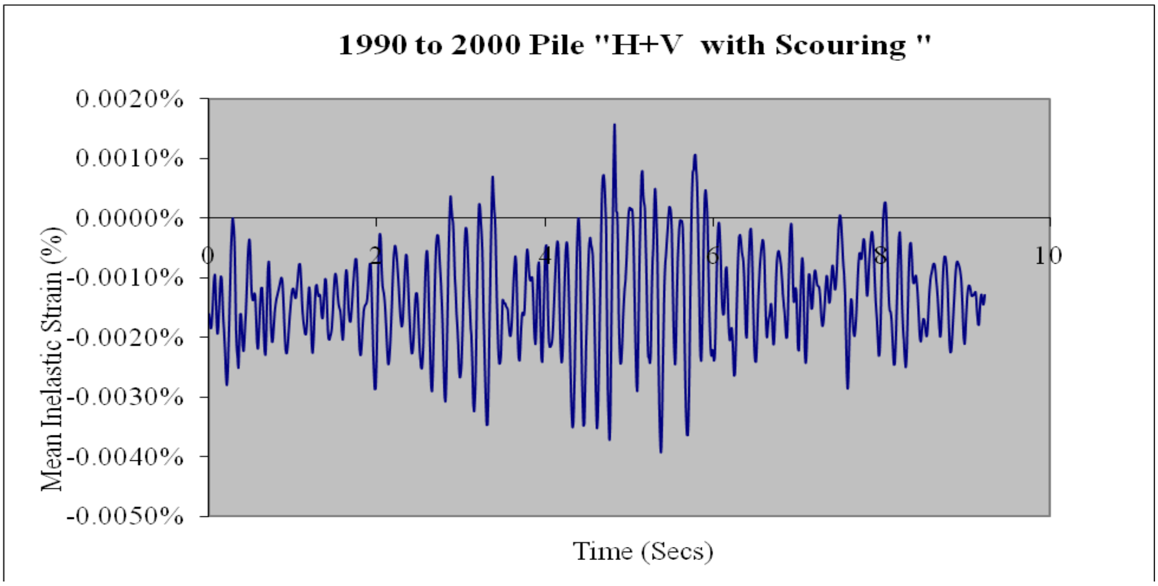

AFTER 2000 PILE"H"
0.0000%
-0.0005%
-0.0010%
-0.0015%
-0.0020%
-0.0025%
-0.0030%
-0.0035%
-0.0040%
-0.0045%
0
2
4
6
8
10
Mean Inelastic Strain (%)
Time (Secs)

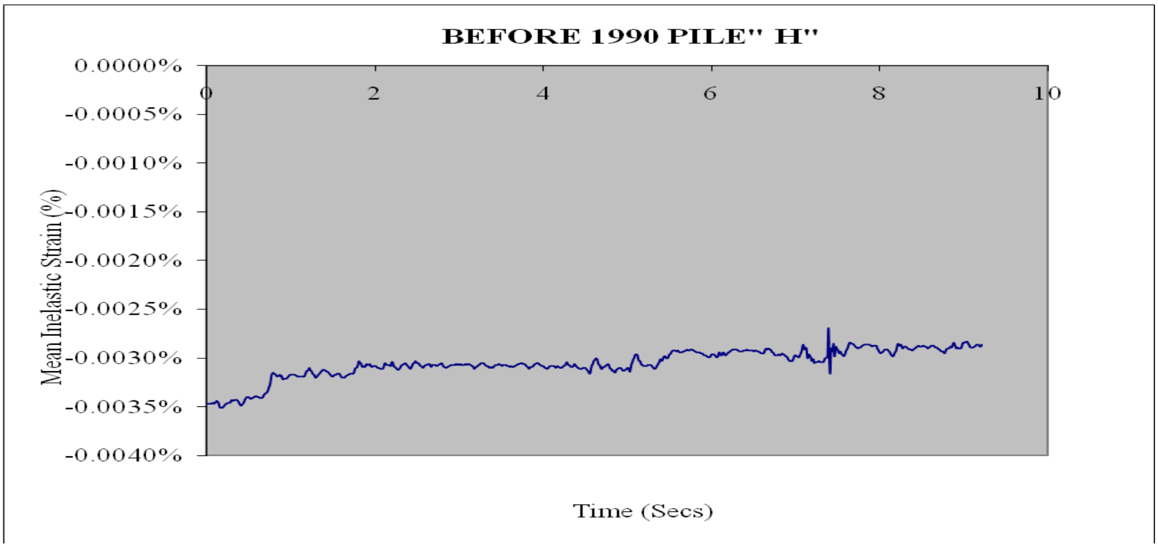

AFTER 2000 PILE"H+V"
0.0030%
0.0020%
0.0010%
0.0000%
-0.0010%
-0.0020%
-0.0030%
-0.0040%
-0.0050%
-0.0060%
-0.0070%
-0.0080%
0
2
4
6
8
Mean Inelastic Strain (%)
Time (Secs)

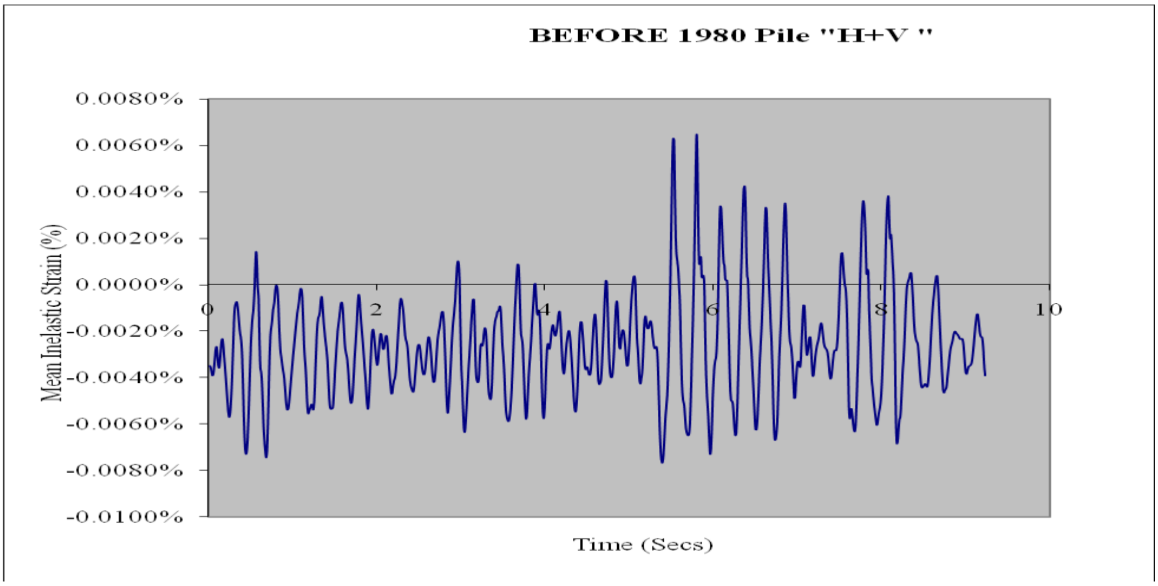

AFTER 2000 PILE"H+V with Scouring"
0.0060%
0.0040%
0.0020%
0.0000%
-0.0020%
-0.0040%
-0.0060%
-0.0080%
0
2
4
6
8
10
Mean Inelastic Strain (%)
Time (Secs)

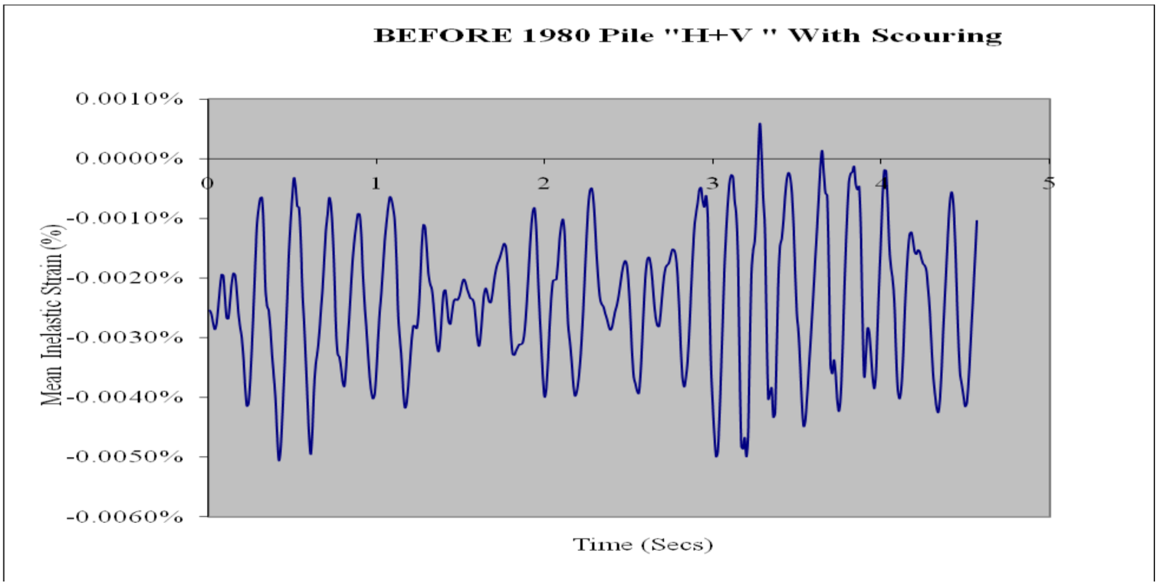

1990 to 2000 Pile "H"
Mean Inelastic Strain (%)
0.0000%
-0.0005%
-0.0010%
-0.0015%
-0.0020%
-0.0025%
0
2
4
6
8
10
Time (Secs)

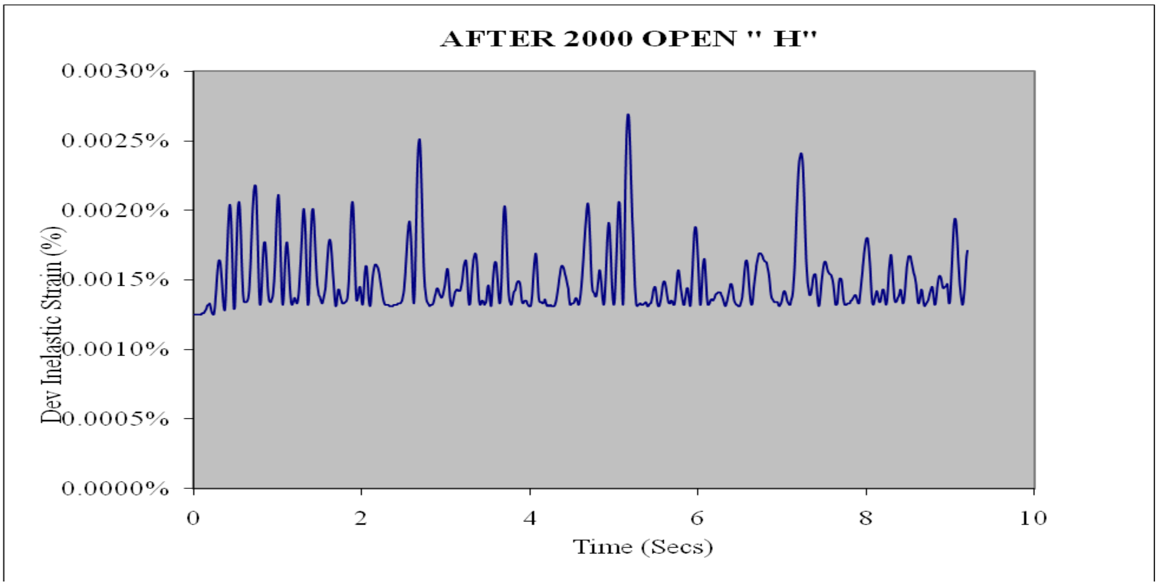

1990 to 2000 Pile "H+V"
Mean Inelastic Strain (%)
0.0020%
0.0010%
0.0000%
-0.0010%
-0.0020%
-0.0030%
-0.0040%
-0.0050%
-0.0060%
0
2
4
6
8
10
Time (Secs)

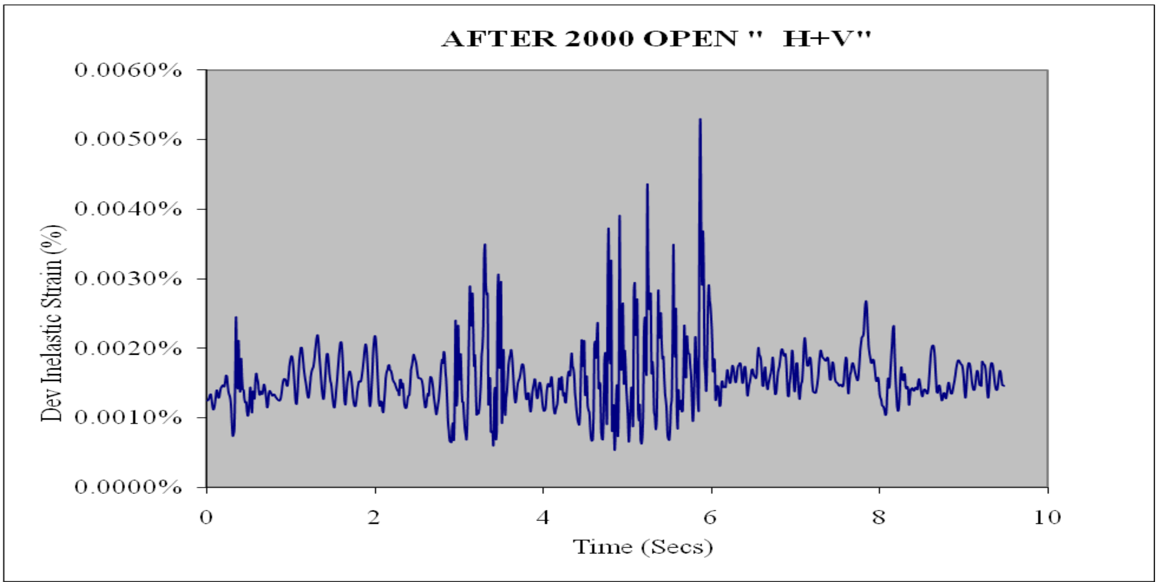

1990 to 2000 Pile "H+V with Scouring "
Mean Inelastic Strain (%)
0.0020%
0.0010%
0.0000%
-0.0010%
-0.0020%
-0.0030%
-0.0040%
-0.0050%
0
2
4
6
8
10
Time (Secs)

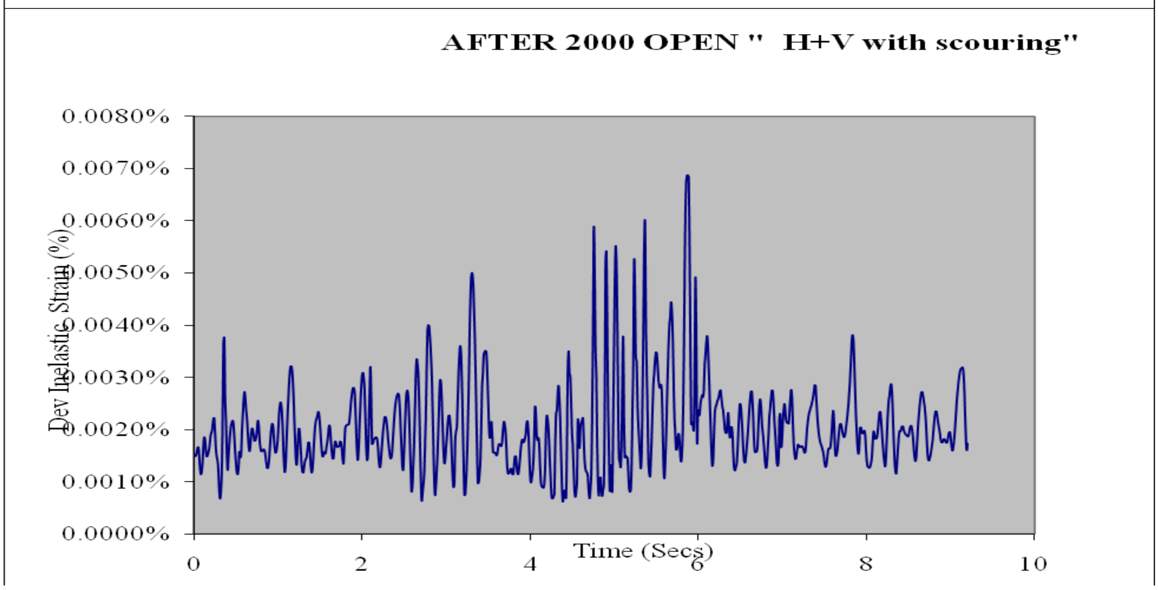

BEFORE 1990 PILE" H"
0.0000%
-0.0005%
-0.0010%
-0.0015%
-0.0020%
-0.0025%
-0.0030%
-0.0035%
-0.0040%
0
2
4
6
8
10
Mean Inelastic Strain (%)
Time (Secs)

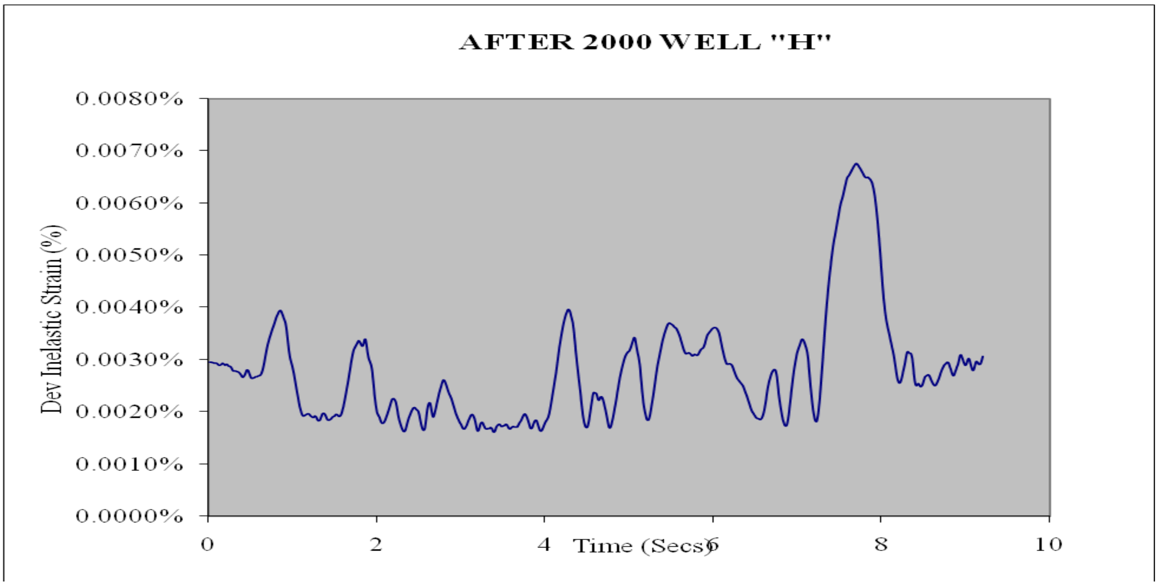

BEFORE 1980 Pile "H+V "
0.0080%
0.0060%
0.0040%
0.0020%
0.0000%
-0.0020%
-0.0040%
-0.0060%
-0.0080%
-0.0100%
0
2
4
6
8
10
Mean Inelastic Strain (%)
Time (Secs)

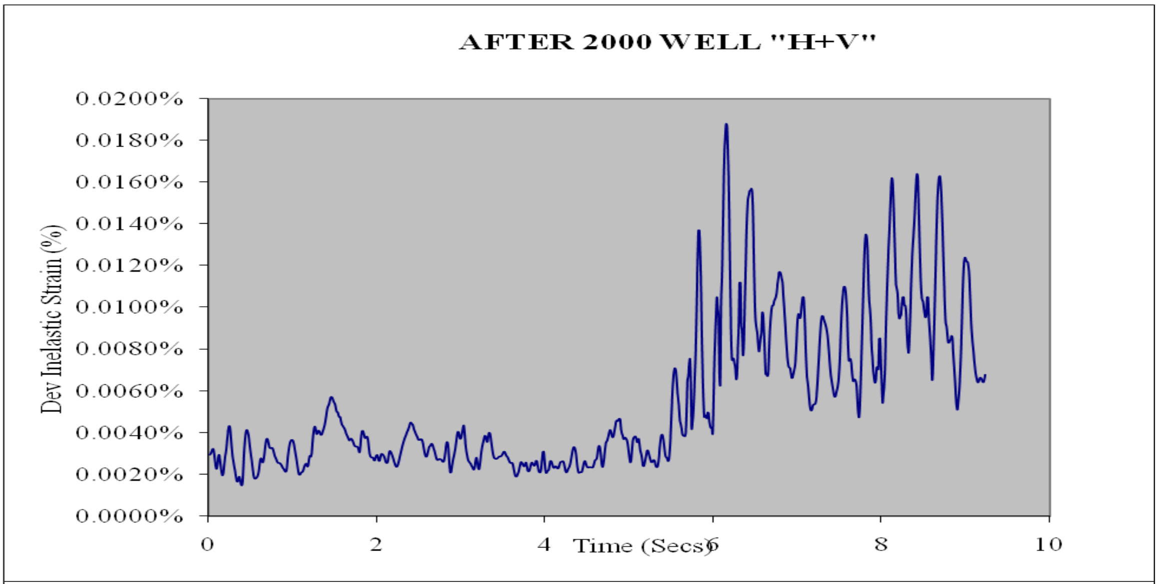

BEFORE 1980 Pile "H+V " With Scouring
0.0010%
0.0000%
-0.0010%
-0.0020%
-0.0030%
-0.0040%
-0.0050%
-0.0060%
0
1
2
3
4
5
Mean Inelastic Strain (%)
Time (Secs)

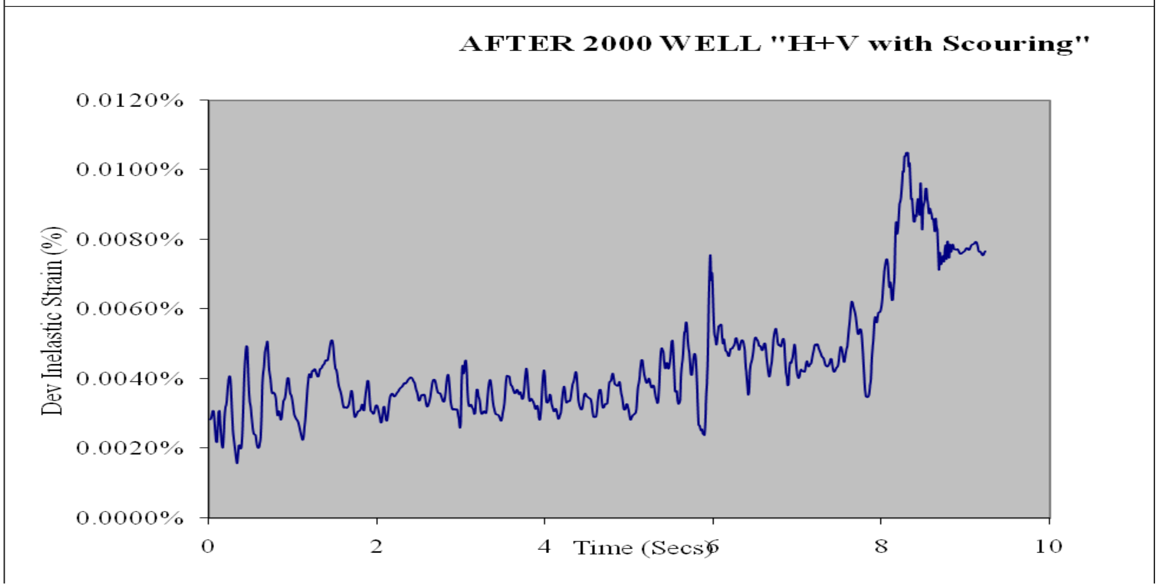

AFTER 2000 OPEN " H"
Dev Inelastic Strain (%)
0.0030%
0.0025%
0.0020%
0.0015%
0.0010%
0.0005%
0.0000%
0
2
4
6
8
10
Time (Secs)

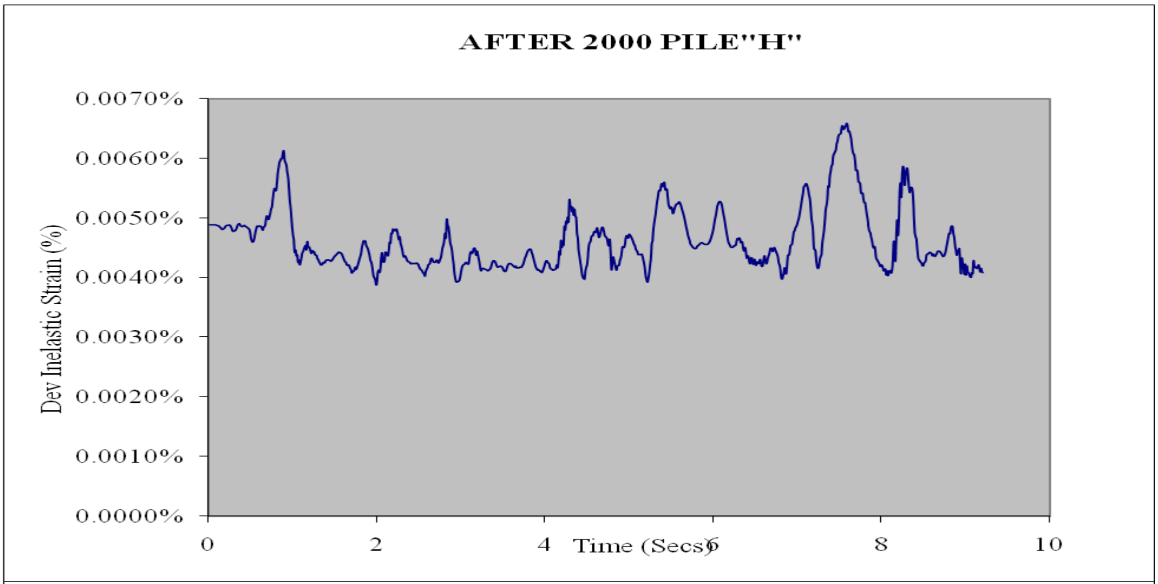

AFTER 2000 OPEN " H+V"
Dev Inelastic Strain (%)
0.0060%
0.0050%
0.0040%
0.0030%
0.0020%
0.0010%
0.0000%
0
2
4
6
8
10
Time (Secs)

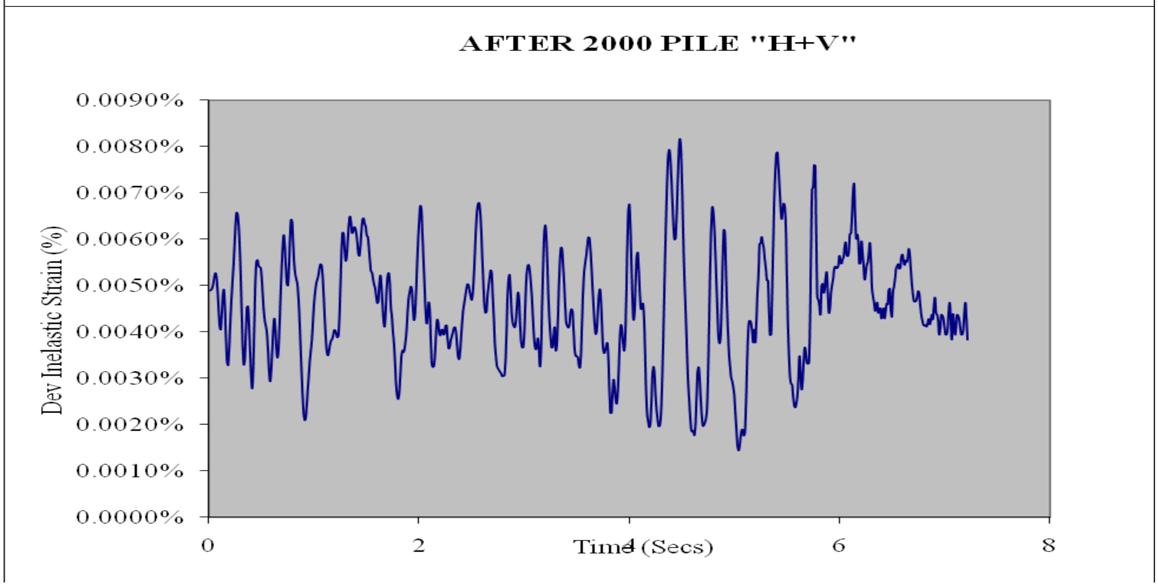

AFTER 2000 OPEN " H+V with scouring"
Dev Inelastic Strain (%)
0.0080%
0.0070%
0.0060%
0.0050%
0.0040%
0.0030%
0.0020%
0.0010%
0.0000%
0
2
4
6
8
10
Time (Secs)

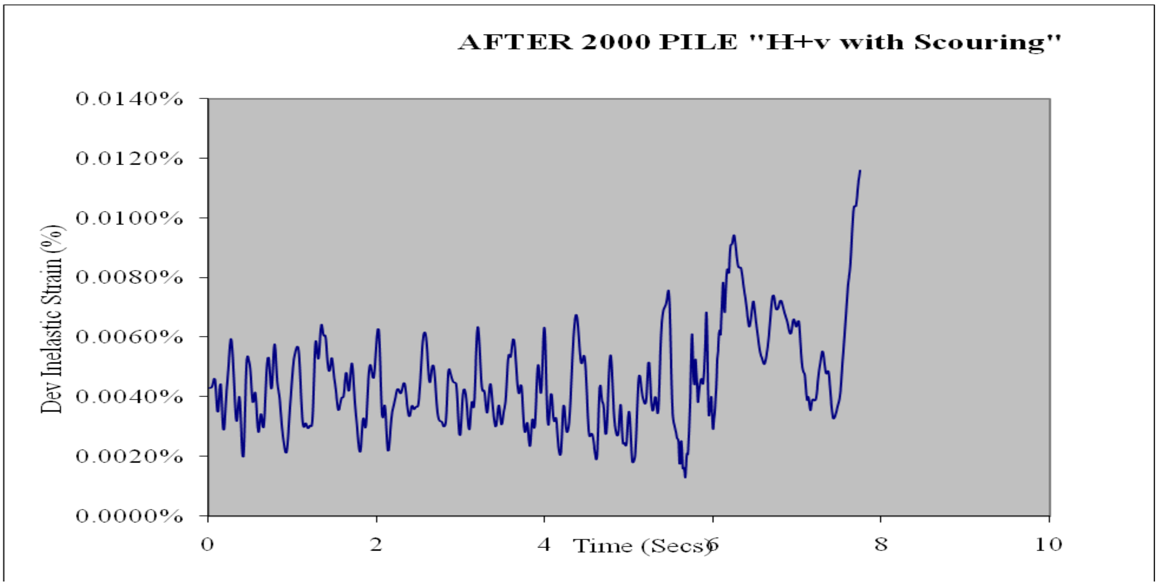

AFTER 2000 WELL "H"
Dev Inelastic Strain (%)
0.0080%
0.0070%
0.0060%
0.0050%
0.0040%
0.0030%
0.0020%
0.0010%
0.0000%
0
2
4
Time (Secs)
6
8
10

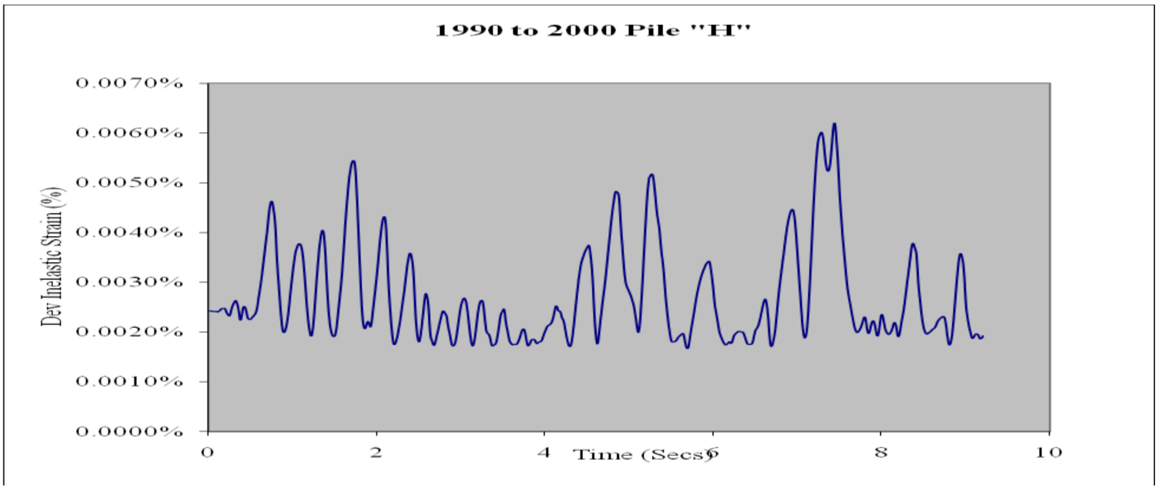

AFTER 2000 WELL "H+V"
Dev Inelastic Strain (%)
0.0200%
0.0180%
0.0160%
0.0140%
0.0120%
0.0100%
0.0080%
0.0060%
0.0040%
0.0020%
0.0000%
0
2
4
Time (Secs)
6
8
10

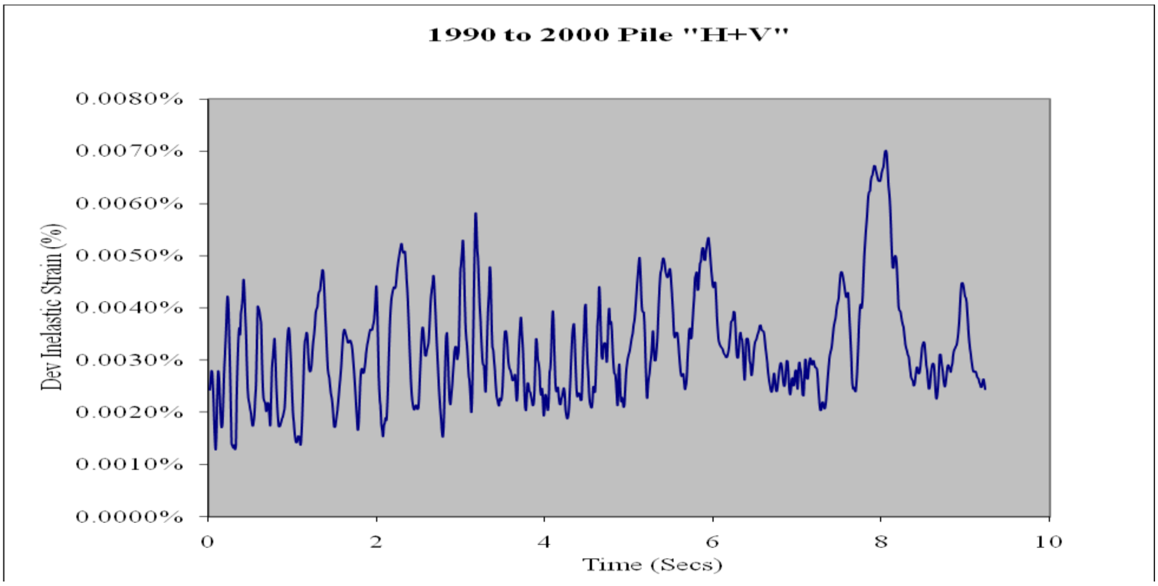

AFTER 2000 WELL "H+V with Scouring"
Dev Inelastic Strain (%)
0.0120%
0.0100%
0.0080%
0.0060%
0.0040%
0.0020%
0.0000%
0
2
4
Time (Secs)
6
8
10

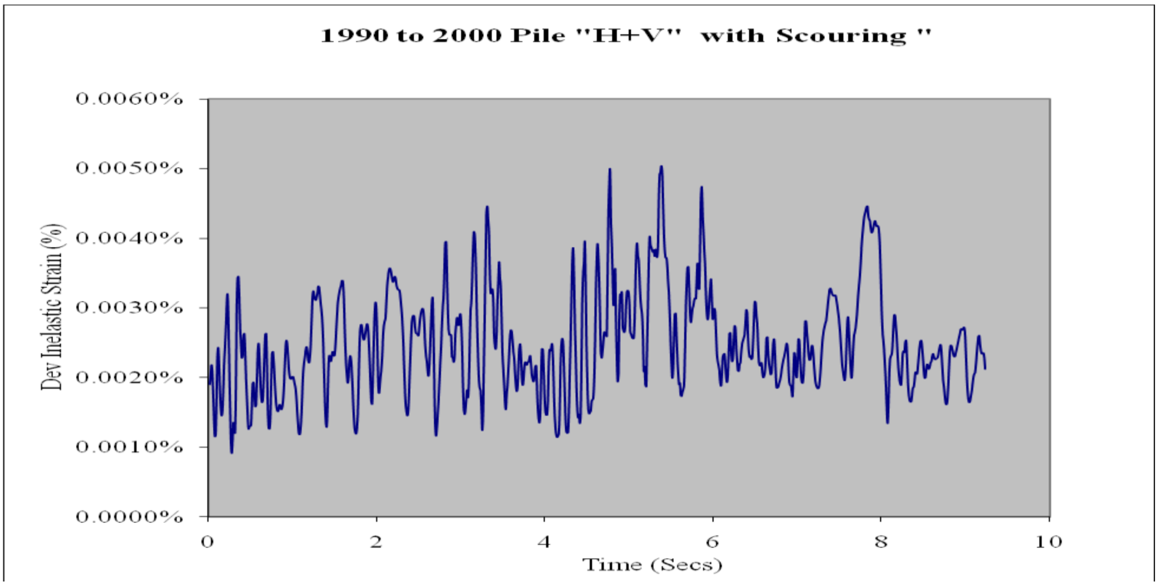

AFTER 2000 PILE"H"
0.0070%
0.0060%
0.0050%
0.0040%
0.0030%
0.0020%
0.0010%
0.0000%
Dev Inelastic Strain (%)
0
2
4
6
8
10
Time (Secs)

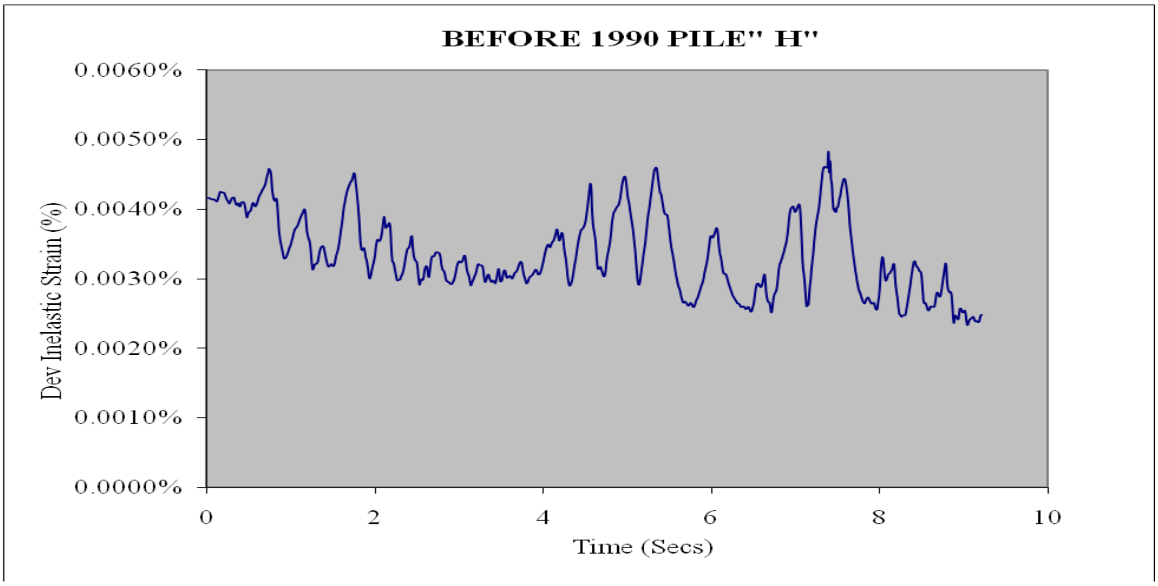

AFTER 2000 PILE "H+V"
0.0090%
0.0080%
0.0070%
0.0060%
0.0050%
0.0040%
0.0030%
0.0020%
0.0010%
0.0000%
Dev Inelastic Strain (%)
0
2
4
6
8
Time (Secs)

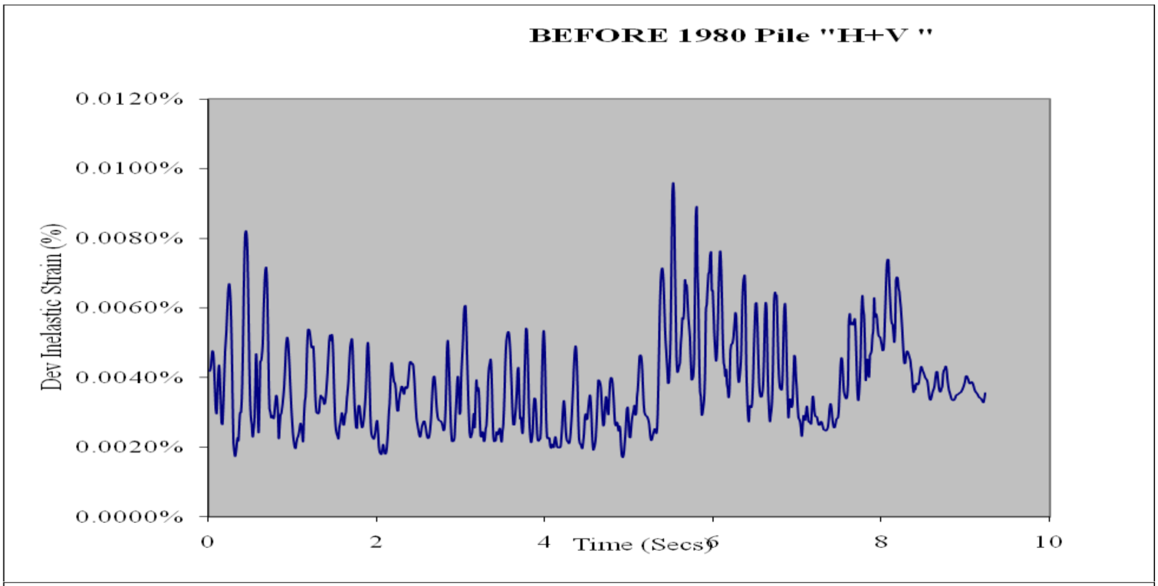

AFTER 2000 PILE "H+v with Scouring"
0.0140%
0.0120%
0.0100%
0.0080%
0.0060%
0.0040%
0.0020%
0.0000%
Dev Inelastic Strain (%)
0
2
4
6
8
10
Time (Secs)

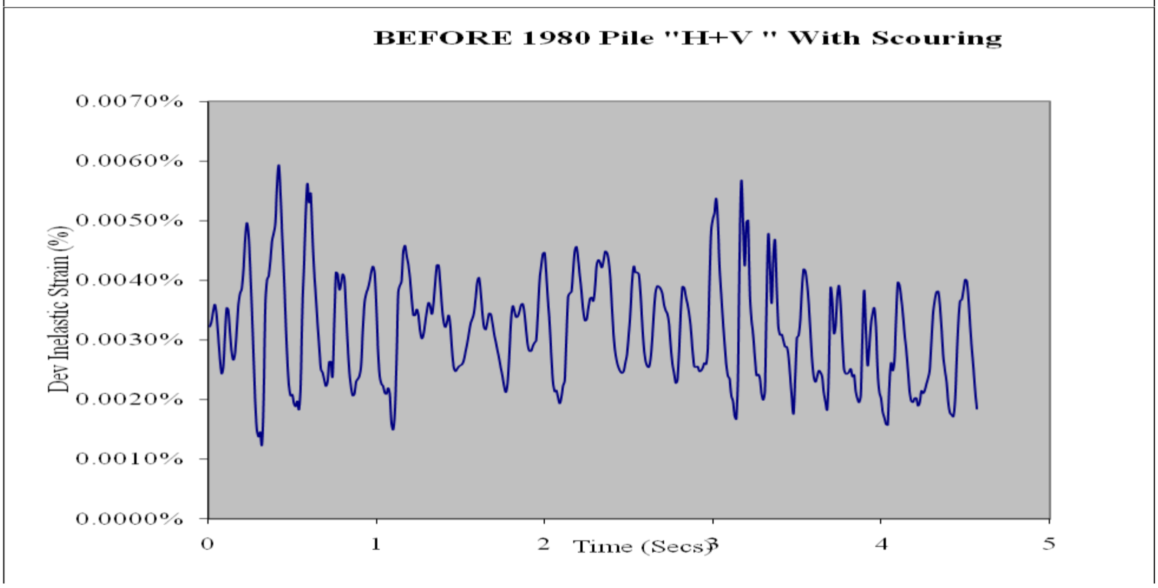

1990 to 2000 Pile "H"
Dev Inelastic Strain (%)
0.0070%
0.0060%
0.0050%
0.0040%
0.0030%
0.0020%
0.0010%
0.0000%
0
2
4
6
8
10
Time (Secs)


1990 to 2000 Pile "H+V"
Dev Inelastic Strain (%)
0.0080%
0.0070%
0.0060%
0.0050%
0.0040%
0.0030%
0.0020%
0.0010%
0.0000%
0
2
4
6
8
10
Time (Secs)


1990 to 2000 Pile "H+V" with Scouring "
Dev Inelastic Strain (%)
0.0060%
0.0050%
0.0040%
0.0030%
0.0020%
0.0010%
0.0000%
0
2
4
6
8
10
Time (Secs)


BEFORE 1990 PILE" H"
Dev Inelastic Strain (%)
0.0060%
0.0050%
0.0040%
0.0030%
0.0020%
0.0010%
0.0000%
0
2
4
6
8
10
Time (Secs)


BEFORE 1980 Pile "H+V "
Dev Inelastic Strain (%)
0.0120%
0.0100%
0.0080%
0.0060%
0.0040%
0.0020%
0.0000%
0
2
4
6
8
10
Time (Secs)


BEFORE 1980 Pile "H+V " With Scouring
Dev Inelastic Strain (%)
0.0070%
0.0060%
0.0050%
0.0040%
0.0030%
0.0020%
0.0010%
0.0000%
0
1
2
3
4
5
Time (Secs)