EVALUATION OF SEISMIC CAPACITY OF EXISTING HIGHWAY BRIDGES IN SRI LANKA

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DECLARATION

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ABSTRACT

Sri Lanka is an island located in the Indian Ocean and it lies in the large Indo-Australian plate seemingly far away from any of the plate boundaries. Therefore, many people believe that this fortuitous scenario makes Sri Lanka safe from earthquakes.

But an intra-plate earthquake can occur anywhere at any time. Some geologists pointed out that the Indo Australian plate is being separated into two and its boundary lies 500km away from the southern coast of the country. Therefore, Sri Lanka has a moderate risk to face an earthquake.

There are over 4000 bridges on National Road Network with length varying from 3.0m to 500.0m. These bridges have varying widths about 3.0m to 25.0m and some of these have been constructed more than 50 to 100 years back. They were constructed using steel concrete composite or steel. These bridges have not been designed for seismic loads and they have not been detailed for seismic effects. Therefore, it is a must to evaluate the seismic capacity of those bridges and retrofit those if necessary.

This study was focused to develop a priority list (Bridge Rank) for the purpose of further investigation on seismic capacity. It was also focused to carry out a case study for a selected bridge from the developed priority list to find out its seismic capacity.

Bridges on the "A" class roads with the overall length of the bridge is greater than 25m were considered in this study. To develop the priority list for thesebridges, the method given in the "Seismic Retrofitting Manual for Highway Bridges" published by the Federal Highway Administration (Report No. FHWA-RD-94-052) was used. The parameters required to input to the above methodology were obtained from the previous research findings and the bridge inventory that is maintained by the Planning Division of RDA, Sri Lanka.

The bridges considered under this study have low risk to fail due to possible earthquake loadings with local conditions since the bridge rank is between 0 to 24 on the scale of 100.

Bridge No 1/1 on PeliyagodaPuttalam road (Japanese Friendship Bridge) was selected for further investigation from the developed priority list since it gives the bridge ranking 12. A response spectrum analysis was carried out to find the actions of the bridge during an earthquake. For the analysis of the bridge, a Finite Element Model was developed using SAP 2000. Codes of practices for Australian standards were used to find out the seismic capacities of the substructure and the actions of superstructure was compared with the originally designed actions.

The bridges considered under this study have low risk to fail due to possible earthquake loadings since the bridge rank is between 0 to 24 on the scale of 100. It is proposed to replace the bridge bearings of the bridge no 1/1 on PeliyagodaPuttalam road based on the results of the case study.

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Keywords: Earthquake, Bridges, Bridge rank, Retrofitting

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LIST OF ABBREVIATIONS

a	-	Acceleration coefficient
A_g	-	Gross cross sectional area of the member
A_s	-	Area of tension reinforcement
AVR	-	Abutment vulnerability
B	-	Width of the deck
b	-	Width of the section
b_{max}	-	maximum transverse column dimension
CVR	-	Column vulnerability
D	-	Dead load
d	-	Effective depth to tension reinforcement
E	-	Seismic hazard rating
e	-	Eccentricity of the prestressing force
EP	-	Earth pressure (Soil pressure + Surcharge)
EQ	-	Earthquake loading
F	-	Framing factor
F_v	-	Site factor depend on long term spectral acceleration
f_{cf}'	-	Characteristic flexural strength of the concrete
f_{cu}	-	Characteristic strength of concrete
f_y	-	Yield strength of reinforcement
F_v	-	Site factor depend on short term spectral acceleration
H	-	Average height of piers/columns supporting the bridge deck.
H_r	-	Total elastomer thickness
K_H	-	Lateral Stiffness
k_u	-	Neutral axis parameter
K_v	-	Vertical Stiffness
K_θ	-	Rotational Stiffness
L	-	Length of the bridge deck. (From seat to adjacent expansion joint)
Lc	-	effective column length

LVR - Liquefaction vulnerability

M_u - Ultimate resistance moment

M_{uo} - Ultimate strength in bending without axial forces

N - Required seat length

P - Prestressing force

PGA - Peak ground acceleration

P_s - amount of main reinforcing steel expressed as a percent of the column cross sectional area

R - Bridge rank

S - Site factor

S_1 - Long term spectral acceleration

SRC - Seismic retrofitting category

S_s - Short term spectral acceleration

V - Structural vulnerability

V_1 - Superstructure vulnerability

V_2 - Substructure vulnerability

V_{uc} - Shear strength excluding shear r/f

V_{us} - Shear strength contributed by shear r/f

W - Load of the wearing surface

Z - Section modulus of the uncracked section

z - Lever arm

α - angle of skew

ϕ - Capacity reduction factor

ε_{sc} - shear strain at edge of bonded surface due to loads normal to bearing surface

ε_{sh} - shear strain at edge of bonded surface due to force tangential to the surface or movement of the structure or both

ε_{sr} - shear strain at edge of bonded surface due to relative rotation of bearing surface to bearing surface

δ_a - maximum shear displacement tangential to the bearing surface

LIST OF APPENDICES

CHAPTER 1 INTRODUCTION

1.1 Background

Can we get there? How Quickly? How does it provide the health facilities? How heavy a load can be transported? How much it will cost to repair the damages? How long will it take? These are some of questions posed by the disaster managers, recovery planers, and structural engineers after a natural disaster.

Damages to built environment from natural disasters are unpredictable and unavoidable at most of the times. Natural disasters occur when the earth releases its concentrated energy gained from various energy sources. It can release the energy in the form of earthquakes, cyclones, tsunami, volcanic eruptions, landslides, floods, etc., and cause much damage to both human lives and built environments.

Damage due to earthquakes and tsunamis are the most vulnerable and it occurs within very short period (Within few seconds). Past records indicate that there were one massive tsunami and few medium scale earthquakes hit the Island.

Everyone in the country believes that the Island is far away from the boundary of tectonic plates. Therefore there is no any risk to face an inter plate earthquake. But the risk due to intra plate earthquake cannot be neglected.

Also it is required to concern about the new research findings regarding the plate tectonics around the country. Some geologists pointed out that the "Indo-Australian" plate is going to separate into two and its boundary lies 500km away from the Southern coast of the country [1].

There are more than 4000 bridges on National highways in the country. Those bridges are not designed to cater for seismic effects. At least earthquake resisting detailing is not applied for the bridges.

Therefore, there is a risk to damage the bridges in the country in case of an earthquake. This urges to the relevant authorities to find out the resisting capacity of the existing bridges in our road network for possible seismic loadings under local conditions and retrofit those, if necessary.

1.2 Objectives

Main objective of the research is to prepare a priority list (Bridge rank) to identify the priority of the bridges that requires further evaluation for retrofitting under loadings in Sri Lankan conditions.

1.3 Methodology

To prepare the priority list, it was adopted the method given in the Seismic Retrofitting Manual for Highway Bridges Published by the Federal Highway Administration (Report No. FHWA-HRT-06-032)[2].

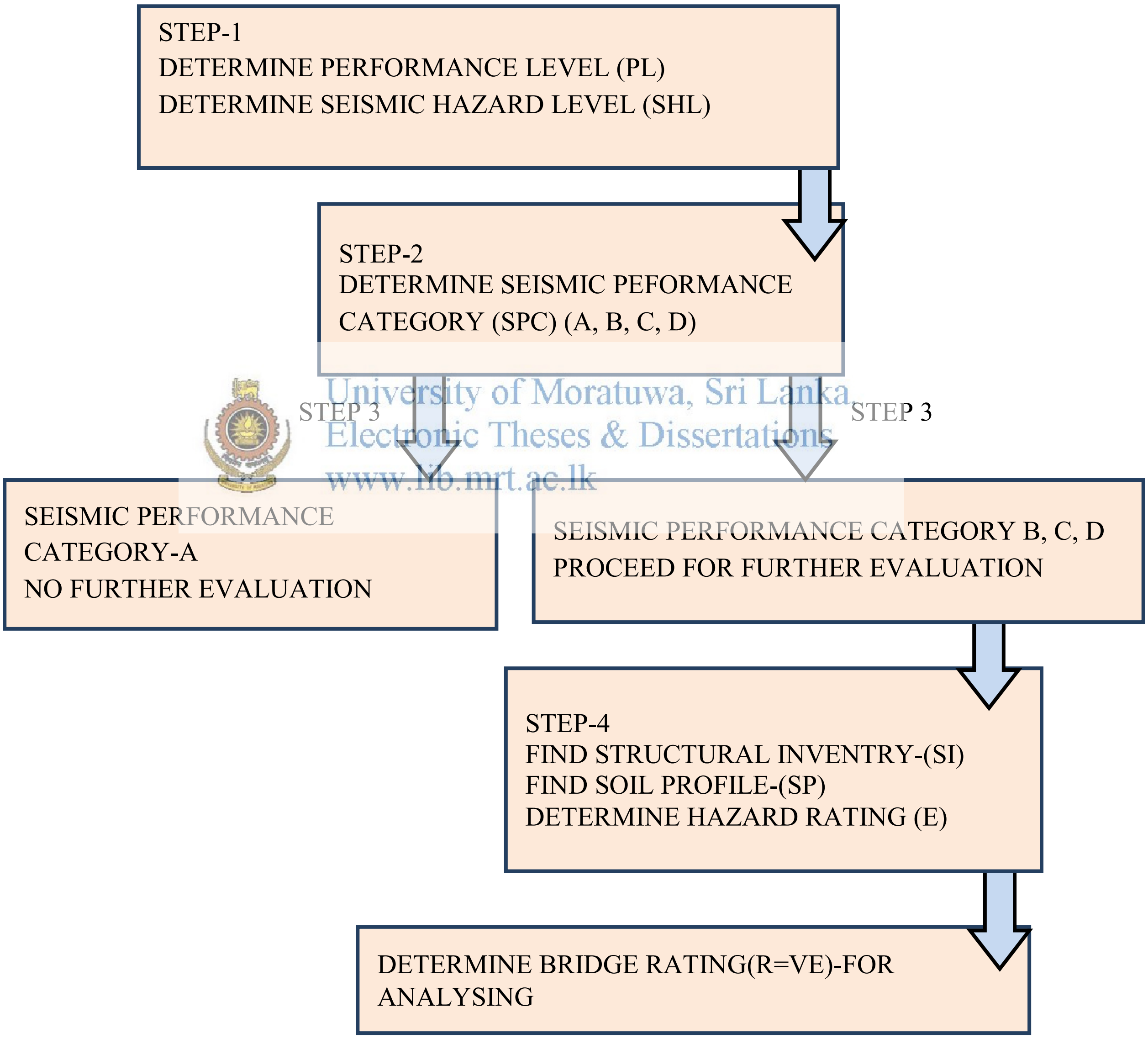

Figure 1-1 Flow chart showing the steps of bridge ranking [3]

A bridge was selected based on the bridge ranking to detailed structural analysis. The bridge was analyzed as per the Australian standards[4] [5] [6] [7] by generating a model using SAP 2000 vr.14.1.0. From this analysis, it was determined the bending moments, shear forces, torsional moments, etc. of the element of the bridge.

Structural requirements of the bridge elements were calculated according to the Australian Standards for seismic loadings. It was compared the findings with the existing details of the bridge element to find the elements that required retrofitting.

1.4 Outcomes of the Study

As per the developed bridge rank, bridges on “A” class roads are in the low risk to fail due to a possible earthquake loadings since the rank of those are in between 0 to 24 on the scale of 100.

From the above bridges, the bridge No 1/1 on PeliyagodaPuttalam road (Japanese Friendship Bridge) was analyzed for possible earthquake loadings and it was found that it is necessary to replace the bridge bearings to cater the possible seismic effects for Sri Lanka.

1.5 Outline of the Thesis

The arrangement of the report in brief is given below.

- Chapter 2 – Literature review.

  A detailed literature review was carried out in order to achieve the objectives. Literature was reviewed on earthquakes, Earthquake history in Sri Lanka, National road network and bridges in Sri Lanka, Bridge failures due to seismic effects, Peak ground acceleration, Bridge ranking and detailed seismic evaluation.

- Chapter 3 – Methodology.

  In Chapter 3 of this report, the methodology of this study is described. It is explained from the selection of the bridges for this study to calculate the bridge rank. Also it explains the design checks that were carried out for the bridge that was analyzed under the case study.

- Chapter 4 – Evaluation of Existing Bridges.

  In this chapter, it is discussed the preparation of bridge rank according to the Seismic retrofitting manual for highway bridges published by the Federal Highway Administration (Report No. FHWA-HRT-06-032).

- Chapter 5 – Case Study.

  Analysis of a bridge that was selected from the developed bridge rank in chapter 4 is included in this chapter. It also includes the capacity calculation of bridge elements according to the Australian Standards of design.
- Chapter 6 – Conclusion and Future Works.

  This chapter concludes the whole research topics carried out under this study. Also this includes the areas that need an extended study.

CHAPTER 2 LITERATURE REVIEW

2.1 Earthquakes

An earthquake (also known as a quake, tremor or tremble) is the result of a sudden release of energy in the earth's crust that creates seismic waves. The seismicity or seismic activity of an area refers to the frequency, type and size of earthquakes experienced over a period of time [8].

There are two types of earthquakes called

- Inter plate earthquakes
- Intra plate earthquakes

Inter plate earthquakes occur near the tectonic plate boundaries and those are very common. But intra plate earthquakes can occur anywhere in the world and those are very rare.

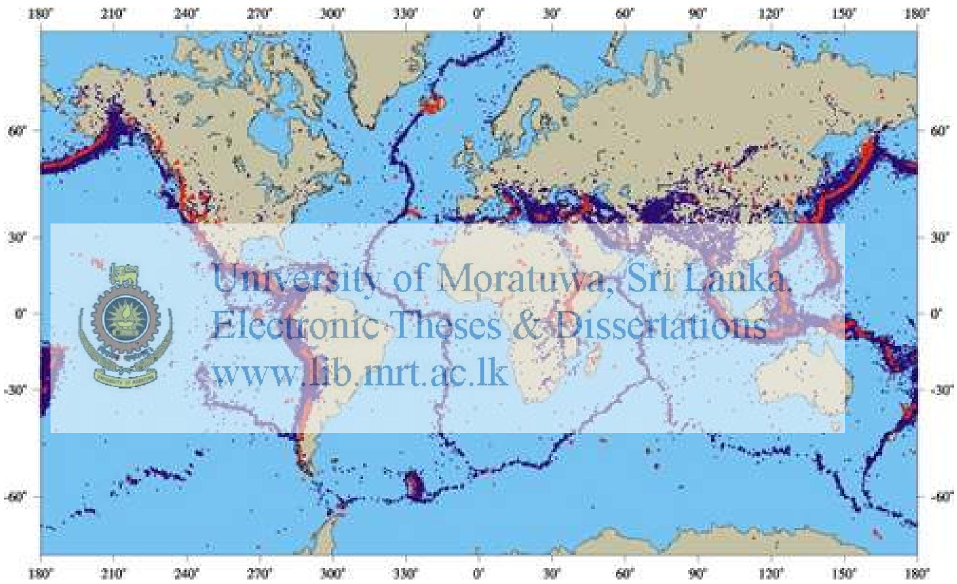

Figure 2-1Distribution of earthquakes around the world

(Source www.prophecydude.org)

2.1.1 Earthquake history of Sri Lanka

Sri Lanka is a country that is located on a low seismic area. But there are few recorded seismic events in different parts of the country. Onshore hazard are low but earthquakes in the range of **M** 5.0-6.0 have occurred in the Gulf of Mannar [9].

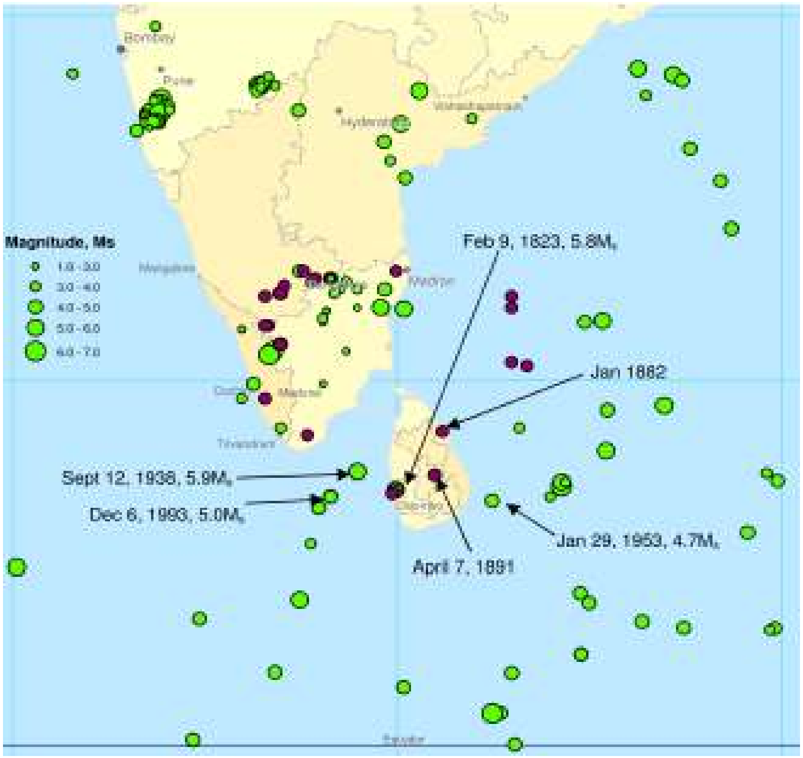

Figure 2-2Past earthquakes in and around Sri Lanka [10]

Past earthquake details are listed below.

Table 2-1Past seismic events in and around Sri Lanka[9]

Date	Location	Magnitude	Damage
14 April 1615	Colombo		2000 people were killed & 200 houses were damaged
09 Feb. 1823	Colombo	5.8M	No damages reported
18 April 1891	Mahiyanganaya		One people was killed
12 Sep. 1938	100km Northwest of Colombo – in the sea	5.9M	No damages reported
29 Jan. 1953	Southeast of Baticaloa – in the sea	4.7 M	
06 Dec. 1993	170km West of Colombo	5.0 M	No damages reported

Other than the above, it was experienced series of minor tremors in Eastern province of the country during the year 2012.

Therefore it may be worthy to check the capacity of our Civil Engineering structures to withstand seismic events in the range of M5-6[9].

2.1.2 Plate Tectonic around Sri Lanka

There are seven major tectonic plates in the world. They are African, Antarctic, Eurasian, North American, South American, Pacific, and Indo-Australian. There are dozens of smaller plates, the seven largest of which are the Arabian, Caribbean, Juan de Fuca, Cocos, Nazca, Philippine Sea and Scotia [8].

Sri Lanka is located at a place where there are no seismic events. But history has witnessed that there were few recorded seismic events in the country as mentioned in section 2.1.1. and series of minor tremors were felt in the Eastern province of the country during 2012.

New research findings show that the Indo Australian Plate is being splitting and its boundary lies 400 - 500km away from the southern coast of the country. Therefore Sri Lanka now needs to be classified as a “Moderate Earthquake Prone Area” [1] [11].

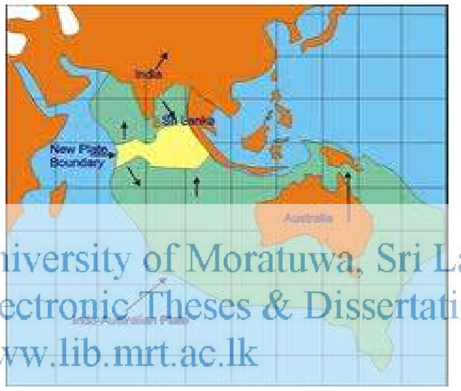

Figure 2-3New plate boundary formed near Sri Lanka[11]

2.2 National Road network and Bridges in Sri Lanka

Express ways and Road classes classified as “A” & “B” is considered as national roads. On these roads, there are about 5000 bridges as per the latest information of the Road Development Authority, Sri Lanka.

History of Sri Lankan Bridges starts with the ancient “Gal Palama” that was constructed by King Dewanampiyathissa. After that the “Bogoda Bridge” comes to the scene and it was constructed in early 16^{th}century. Part of the bridge can be seen today as well and it is good evidence to the technology and the material used in that period.

Figure 2-4Ancient "galpalama"

(Source www.galpalama.blogspot.com)

Figure 2-5Ancient "Bogoda timber bridge"

(Source www.thearchitect.lk)

During the colonial period, the bridge construction was rapidly developed. That is mainly with steel trusses and steel girders to the superstructure. Masonry substructure was very common. Apart from the above, masonry arch bridges were also constructed. Best example for the masonry arch bridges is the "Nine Arches" bridge on Badulla railway line.

Figure 2-6"Nine arches" bridge on Badulla railway line

(Source www.sunnyside.go2lk.com)

At present the most common bridge type is concrete bridges with pre stressed concrete superstructure. With the rapid development of the country, during the past ten years, bridges were constructed with the overall length over 250m. (Upparu, Manampitiya, Bridge across Bentharaganga in Southern Transport Development Project, etc.).

2.3 Bridge Failures Due to Seismic Effects

Bridges can fail in many ways due to seismic effects.

- Unseating at expansion joints

  Most areas in the world, bridges often comprise series of simple spans supported on piers. These spans are prone to be toppled from their supporting substructures either due to shaking or differential support movement associated with ground deformations. Skew bridges and curved bridges are more vulnerable for this failure [12].

Figure 2-7Superstructure dislocation at expansion joint[12]

- Bearing failure

  Bearings are the elements that transfer the loads from superstructure to substructure of the bridge. They provide restraints in one or more directions and in some cases permits movement in one or more directions. Failure of these bearings in an earthquake can cause redistribution of internal forces, which may overload superstructure or substructure or both. Collapse is also possible when bearing support is lost [12].

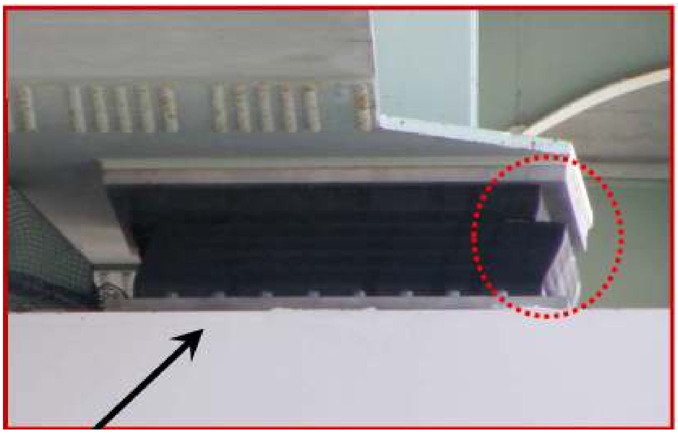

Figure 2-8Bridge bearing failure[12]

- Column failure

  Columns can be subjected to a large inelastic demand during strong earthquake. Failure in column can result in loss of vertical load carrying capacity. Column failure is often the primary cause of bridge collapse.

  Most damage to column can be attributed to inadequate detailing, which limits the ability of the column to deform inelastically. In concrete columns, the detailing inadequacies can produce flexural, shear, splice, or anchorage failure, or as is often the case, a failure that combines several mechanisms [12].

Figure 2-9Pier failure due to base shear[13]

- Abutment failure

  The type of abutment failures that can occur during an earthquake varies from one bridge to the other. Most of the time, soil liquefies during an earthquake and it will act a very important role for abutment failure. Interactions between soil and wing walls are also worsen the effects of seismic forces acting on the abutments. During an earthquake, there are large seismic forces act on stiff abutments.

  Excessive relative displacement of an abutment and the superstructure can result in abutment unseating failures. This usually happens due to a result of the soil liquefaction[12].

Figure 2-10 Abutment failure due to liquefaction[12]

- Foundation failure

  Foundation failures are very rare events due to the seismic forces. This can happen due to liquefaction of soil. But it is not clear that whether the events are rare or not reported due to the foundations remaining underground. Foundation damages associated with the soil liquefaction induced lateral spread has probably been the single greatest cause of distress and collapse of bridges [12].

2.4 Peak Ground Acceleration

Peak ground acceleration (PGA) is a measure of earthquake acceleration on the ground and an important input parameter for earthquake engineering, also known as the design basis earthquake ground motion.

Seismic hazard map is not available for Sri Lanka. Therefore, deciding the peak ground acceleration should be based on research findings and the data available for similar conditions.

NavinPeiris, [10]recommended 0.026g for 10% probability exceedance in 50 years or 475 year return period.

ChandimaKularathne, [14] “According to the available data, it was suggested for Colombo an earthquake of magnitude close to M_L=6 on Richter scale with a return period of 200 - 400 years; a design acceleration of 0.2g (196cm/s$^{2)}$ is considered as the horizontal component of the earthquake while the vertical component is neglected at this stage. Besides, the attenuation relation developed by Fukushima and Tanaka was applied for earthquake of M_L=6 on Richter scale 5km away from the epicenter. It was found that the peak ground acceleration as 186.4cm/s^2 which is quite similar to suggested acceleration of 0.2g”.

Uduweriya, Wijesundara and Dissanayake[15]proposed that the PGA at rock site for 10% of probability of exceedance in 50years or 475 years return period is 0.1g for Colombo city. Also they use the seismogenic zones related to the southern part of the India around Tamil Nadu including Sri Lanka.

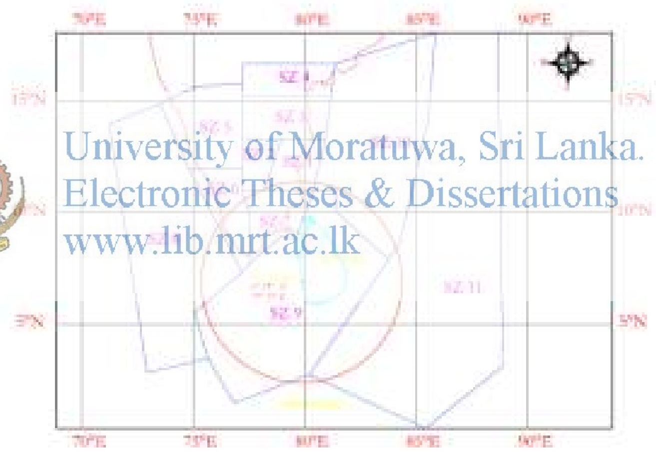

Figure 2-11Different seismic zones around Sri Lanka[15]

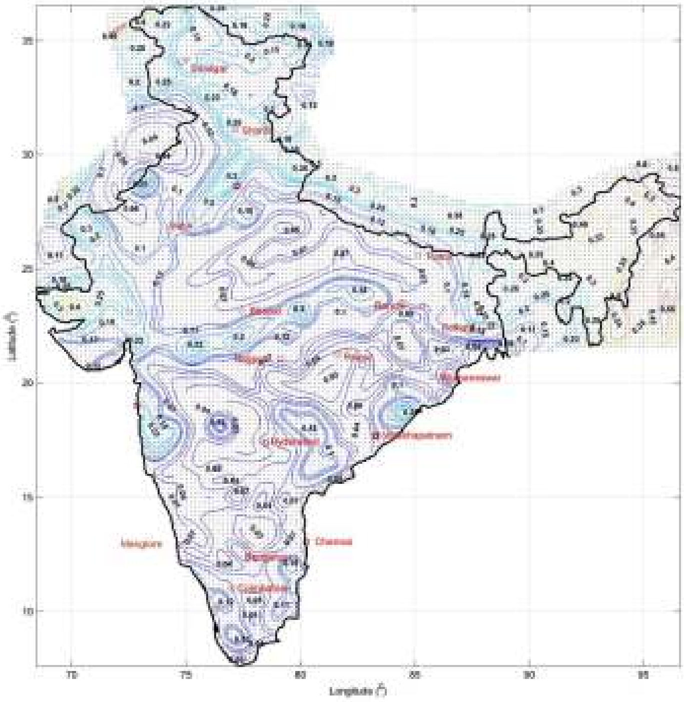

Figure 2-12 Short period spectral acceleration at T=0.2 second with return period of 500 years on A-type sites (5% damping) [16]

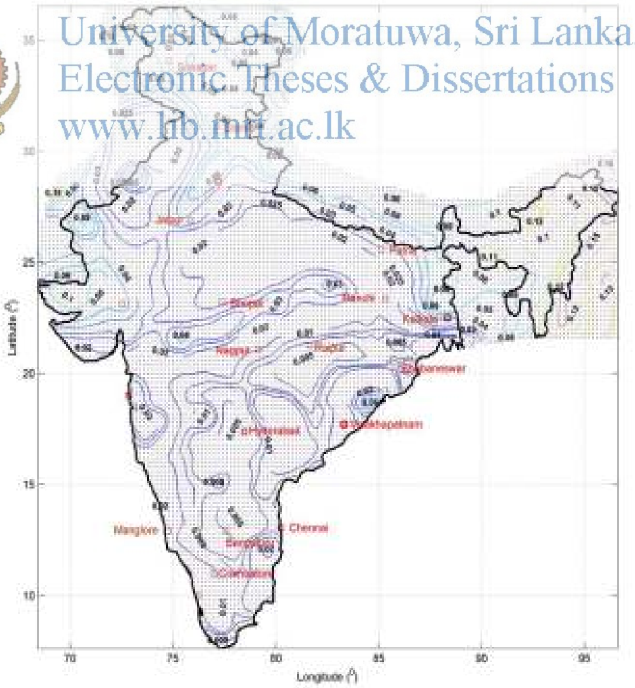

Figure 2-13 Long period spectral acceleration at T=1 second with return period of 2500 years on A-type sites (5% damping) [16]

By considering above it was decided to use 0.1g as the PGA and use the response spectrum defined in the seismic hazard map of India [15].

2.5 Bridge Ranking

In general, a seismic rating system has to be used as a basis for selecting bridges for detailed seismic evaluation.

ChingChiawChoo, Issam E. Harik, Peng Yuan, [3] proposed the method published in Seismic Retrofitting Manual for Highway Bridges Published by the Federal Administration (Report No. FHWA-HRT-06-032).

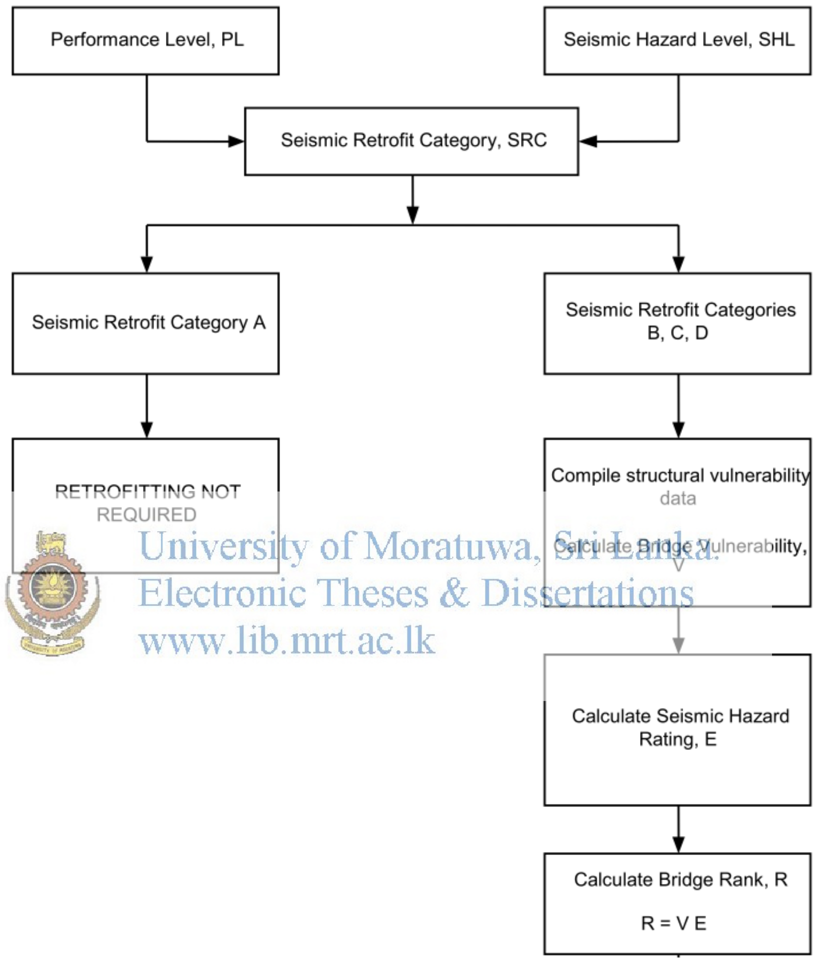

Figure 2-14 Flow chart of derivation of bridge rank [2]

Majid, Yousefi, [17]proposed a simple approach using Multi criteria decision making to rank the bridges in the inventory for retrofitting and seismic upgrading. In this method, the alternatives are analyzed based on a set of criteria including structural vulnerability (V), Seismic hazard and important classification.

In order to apply the multi criteria decision making method, it is necessary to prepare the decision matrix and needs to decide the criteria weights. Deciding of the values for criteria weights are not easy. It needs experts experience and good judgments regarding the suitability of the alternatives.

Other than the above, the structural vulnerability will be decided by visual inspections. This is also creating uncertainties.

By considering the above, the method proposed in the seismic retrofitting manual is adopted to rank the bridges under this study.

2.6 Detailed Seismic Evaluation

The Seismic Retrofitting manual for Highway Bridges, Referred as SR Manual hereafter, published by the Federal Highway Administration proposes two methods.

I. Capacity/Demand (C/D) ratio method

   In this method, the result from an elastic spectral analysis is used to calculate the force and displacement "Demand" which are then compared with the "capacities" of each of the components to resist these forces and displacements. Capacity/Demand (C/D) ratios are intended to represent the decimal fraction of the design earthquake at which a local failure of components is likely to occur. Therefore C/D ratio less than 1.0 indicates that component failure may occur during the design earthquake and retrofitting may be appropriate [2].

II. Lateral Strength Method

   SR manual also provides an alternative analysis approach. In general, the lateral strength method treats the entire bridge system, whether individual segments or frames of the bridge between expansion joints, as a single structural system. The structural system is then evaluated using an incremental collapse analysis, the load deformation characteristics of the bridge up to collapse. The fraction of the design earthquake that can be resisted without collapse is then an indicator of the need for retrofitting and the extent of strengthening required. This procedure therefore determines the strength and ductility of the critical collapse mechanism. But it can be used to identify the onset of damage when serviceability criteria may be important [2].

In the present study only the C/D (Capacity/Demand) ratio method was adopted since it can find the capacity of the each and every item of a bridge.

By reviewing the literature, it was decided to adopt the method given in the SR manual to develop the bridge rank and detailed seismic evaluation. Details of these two methods are discussed in next chapter.

CHAPTER 3 METHODOLOGY

3.1 Selection of Bridges for the Study

As discussed in the section 2.2, there are about 5,000 bridges in national highways (Except in Express ways). Using all the bridges for this analysiswould have made the researchcomplicated. Therefore bridges were selected using the following methodology.

- Consider all bridges in “A” class roads.
- Select all bridges with the overall length greater than or equal 25.0m
- Finally select the bridges with the average span are greater than or equal to15m.

Data were collected in these bridges using the bridge inventory maintained by the Planning division of the Road Development Authority and verify those by going through the as built drawing that are available at the record room of the Road Development Authority.

3.2 Ranking of the Bridges for Analysis

In general, a seismic rating system has to be used as a basis for selecting bridges for detailed seismic evaluation. The information provided is obtained from the Seismic Retrofitting Manual for Highway Bridges Published by the Federal Administration (Report No.FHWA-HRT-06-032). The flow chart of that is mentioned in the section 2.5of this report.

3.2.1 Determination of the Performance Level

Performance level of an existing bridge depends on the importance and the anticipated service life of the bridge.

Importance of all bridges can be classified as *Essential/ Standard* bridges depending on the following. Essential bridges are those that are expected to function immediately after an earthquake or which cross routes that are expected to remain open following an earthquake. All other bridges are standard.

Anticipated service life is the remaining years from the design life. There are three anticipated service life categories as shown in Table 3-1.

Table 3-1Service life categories [2]

Service Life Category	Anticipated Service Life
ASL 1	0 – 15 yrs
ASL 2	16 – 50 yrs
ASL 3	> 50 yrs

There are four performance levels for retrofitted bridges and those are selected as follows.

Table 3-2Service life categories [2]

Earthquake Ground Motion	Bridge importance & Service Life Categories					
	Standard			Essential		
	ASL 1	ASL 2	ASL 3	ASL 1	ASL 2	ASL 3
Lower level ground motion	PL 0	PL3	PL3	PL 0	PL3	PL3
Upper level ground motion	PL 0	PL1	PL1	PL 0	PL1	PL2

PL 0 – No minimum level of performance is recommended

PL 1 –Life safety PL 2 – Operational PL 3 – Fully operational

3.2.2 Determination of the Seismic Hazard Level

Seismic hazard level is to predict the ground motion during an earthquake. Selection of the hazard level depends on the site class and the peak ground acceleration. There are four hazard levels according to the retrofitting manual.

Table 3-3 Seismic hazard level[2]

Hazard level	Using $S_{D1}=F_vS_1$	Using $S_{DS}=F_aS_s$
I	$S_{D1} \leq 0.15$	$S_{DS} \leq 0.15$
II	$0.15 \leq S_{D1} \leq 0.25$	$0.15 \leq S_{DS} \leq 0.35$
III	$0.25 \leq S_{D1} \leq 0.40$	$0.35 \leq S_{DS} \leq 0.60$
IV	$0.40 \leq S_{D1}$	$0.60 \leq S_{DS}$

F_v and F_v are site factors while S_1 and S_s are long term and short term spectral accelerations. The values of those are selected using Figure 2-12 and Figure 2-13.

F_a and F_v depend on spectral acceleration and site class. There are six site classes according to the retrofitting manual from A to F. The details of those are described below.

Table 3-4Site class[2]

Site class	**Description**
A	Hard rock
B	Rock
C	Very dense soil with N > 50
D	Stiff soil with 15 < N < 50
E	Soil with N < 15
F	Peats or highly organic clays

Values of the F_a and F_v are selected using theTable 3-5 and Table 3-6.

Table 3-5 Short period spectral acceleration at T=0.2 second with return period of500 years on A-type sites (5% damping) [2]

Site Class	Spectral Acceleration at short period (0.2sec) S_s[1]				
	$S_s \leq 0.25$	$S_s = 0.25$	$S_s = 0.75$	$S_s = 1.00$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F[2]					
Notes: Use straight line interpolation for intermediate values of S_s. Site specific geotechnical investigations and dynamic site response analysis should be performed for class F soils.					

Table 3-6 Long period spectral acceleration at T=1 second with return period of 2500 years on A-type sites (5% damping) [2]

Site Class	Spectral Acceleration at long period (1.0sec) S_1[1]				
	$S_1 \leq 0.25$	$S_1 = 0.25$	$S_1 = 0.75$	$S_1 = 1.00$	$S_1 \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F[2]					
Notes: Use straight line interpolation for intermediate values of S_1. Site specific geotechnical investigations and dynamic site response analysis should be performed for class F soils.					

3.2.3 Determination of the Seismic Retrofitting Category (SRC)

Seismic retrofitting categories (SRC) are used to identify the minimum screening requirements, evaluation method and retrofitting measures of the deficient bridges. They are determined using performance level and seismic hazard level.

Table 3-7 Seismic Retrofitting categories[2]

Hazard Level	**Performance Level**				
	During upper level earthquakes			During lower level earthquakes	
	PL0 No minimum level	PL 1 Life safety	PL 2 Operational	PL0No minimum level	PL3 fully operational
I	A	A	B	A	C
II	A	B	B	A	C
III	A	B	C	A	C
IV	A	C	D	A	D

Depends on the seismic retrofitting category, retrofitting manual suggests the analysis method and the required checks have to be carried out. As per that, SRC "A" does not require to retrofit while other three categories need further evaluation.

3.2.4 Determination of the Bridge Vulnerability

Bridge vulnerability consists of the superstructure vulnerability and substructure vulnerability. Superstructure vulnerability and substructure vulnerability are calculated separately and the maximum of those is selected.

Vulnerability rating may range from 0 to 10. A rating 0 means a very low vulnerability to unacceptable damage; a value of 5 indicates a moderate vulnerability to collapse or a high vulnerability to loss of access, and a value of 10 means a high vulnerability to collapse. But the vulnerability rating values are not exactly the one of the above values.

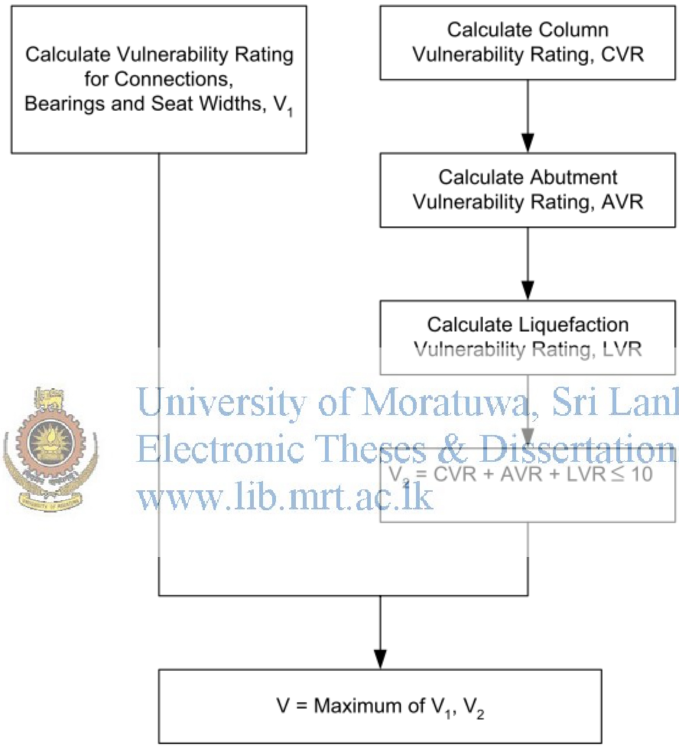

Figure 3-1 Flow chart to determine the bridge vulnerability [2]

To determine the bridge vulnerability, it requires considerable engineering judgment. Procedure for it will be described below.

For bridges classified in Seismic Retrofit Category (SRC) “B”, the vulnerability rating for bearings, transverse restraints, and support length need to be calculated along with a rating for liquefaction effects for bridges on liquefiable soils.

For bridges classified as SRC “C” or “D”, vulnerability ratings for the columns, Abutments, and foundations are also required.

3.2.4.1 Determination of Vulnerability for Connections, Bearings and Seat widths

A suggested step by step method for determining the vulnerability rating for connections, bearings, and seat widths (V_1) is as follows.

Step-I Determine whether the bridge has satisfactory bearing details. Such bridges include:

Continuous structures with integral abutments.

Continuous structures with seat type abutments where all of the following conditions are met:

Either (a) the skew is less than 20^0 (0.35rad), or (b) the skew is greater than 20^0 (0.35rad) but less than 40^0 (0.70rad) and the length to width ratio of the bridge deck is greater than 1.5.

Rocker bearings are not used.

The abutment's bearing seat under the end diaphragm is continuous in the transverse direction and the bridge has more than three beams.

The support length is equal to, or greater than, the minimum required length (N) given in equation 4-3.

If the bearing details are determined to be satisfactory, a vulnerability rating, V1, of 0 may be assigned and the remaining steps for bearings omitted.

Step-II Determine the vulnerability whether the structure collapse or loss of access to the bridge due to transverse movement, V_T.

Before significant transverse movement can occur, the transverse restraint must fail. In the absence of calculations showing otherwise, assume that the bearing keeper bars and/or the anchor bolts in bridges in SRC C and D will fail. Also assume that nominally reinforced, non-ductile concrete shear keys will fail in bridges in SRC D.

When the transverse restraint is subjected to failure, beams are vulnerable to loss of support if either of the following conditions exists:

Individual beams are supported on individual pedestals or columns.

The exterior beams in a 2- or 3- beam bridge is supported near the edge of the bearing seat regardless of whether the bearings are on individual pedestals or not.

In either of these cases, the vulnerability rating, V_T, should be 10.

Steel rocker bearings have been known to overturn transversely, resulting in a permanent superstructure displacement. All bridges in SRC D are vulnerable to this type of failure. Bridges in SRC C are vulnerable only when the skew is greater than 40^0 (0.70 rad). When bearings are vulnerable to a toppling failure but structure collapse is unlikely, the vulnerability rating should be 5. Otherwise V_T=0.

Step-III Determine the vulnerability of the structure to collapse or loss of access due to excessive longitudinal movement, V_L.

V_L is determined according to the available support length (L) measured in a direction perpendicular to the centerline of the support. This is done by comparing L with the minimum required length (N), as follows:

$$N = \left\lfloor 100 + 1.7L + 7.0H + 50\sqrt{1 + (2B/L)^2} \right\rfloor \frac{(1+1.25F_vS_1)}{cos\alpha}$$

Where, N – required seat length

L –Length of the bridge deck.(From seat to adjacent expansion joint)

H – Average height of piers/columns supporting the bridge deck.

B – Width of the deck

α – angle of skew

If L ≥ N then V_L = 0 regardless of bearing type.

If N> L ≥ 0.5N and rocker bearings are not used, then V_L = 5.

If N> L ≥ 0.5N and rocker bearings are used, then V_L = 10.

If 0.5N > L then V_L = 10 regardless of bearing type.

Step-IV Calculate vulnerability rating for connections, V_1, from values V_T and V_L, with V_1= greater of V_T and V_L.

3.2.4.2 Determination of Vulnerability for Columns, Abutments and Liquefaction potential (V_2)

The vulnerability rating for the other components in the bridge that are susceptible to failure, V2, is calculated from the individual component rating as follows.

$$V_2 = CVR + AVR + LVR \leq 10$$

Where CVR is the column vulnerability rating, AVR is abutment vulnerability rating and LVR is liquefaction vulnerability rating.

Column/Pier Vulnerability Rating (CVR).

Columns/Piers have failed in past earthquakes due to lack of adequate transverse reinforcement and poor structural detailing. In past earthquakes, columns/Piers have failed in shear, resulting their disintegration and substantial vertical settlements.

The following procedure may be used to determine the vulnerability of columns and piers.

Step-I. Assign column vulnerability, CVR, of 0 to bridges classified as SRC B.

Step-II Assign column vulnerability, CVR, of 0 if keeper bars of anchor bolts can be relied upon to fail, thereby prevents the transfer of load to the columns or piers.

Step-III If columns/piers have adequate transverse steel as required; assign a CVR, of 0.

Step-IV If none of the above applies, check the column/pier for shear, splice details and foundation deficiencies, and give CVR the highest value calculatedfrom the following steps:

Step 4a. Column vulnerability due to shear failure

$$CVR = Q - P_R$$

Where $$Q = 13 - 6\left(\frac{L_C}{P_S F b_{max}}\right)$$

L_c – effective column length

P_s – amount of main reinforcing steel expressed as a percent of the column cross sectional area

F – Framing factor

2.0 for multi-column piers fixed top and bottom

1.0 for multi-column piers fixed at one end

1.5 for box girder superstructure with a single column pier, fixed at top and bottom

1.25 for superstructures other than box girders with single column pier, fixed at top and bottom

b_{max} – maximum transverse column dimension

P_R – the total number of points to be deducted from Q for factors known to reduce susceptibility to shear failure, as listed below.

Table 3-8Points to be deducted from Q [2]

Factor	**P_R**
Seismic coefficient SD1 < 0.5	3
Skew < 20^0 (0.35rad)	2
Continuous superstructure, integral abutments of stiffness and length to width ratio < 4	1
Grade 40 (or below) reinforcement	1

Values of CVR less than 0 and greater than 10 should be assigned as 0 and 10 respectively.

Step 4b. Column vulnerability due to flexural failure

To account for flexural failure when the column longitudinal reinforcement is spliced in a plastic hinge location, the following CVR should be used for column supporting superstructure longer than 90m, or for superstructure with expansion joints.

$CVR = 7$ for $S_{D1} < 0.5$

$CVR = 10$ for $S_{D1} \geq 0.5$

Step 4c. Column vulnerability due to foundation deficiencies.

The following CVR should be used for columns supported on pile footings that are not reinforced for uplift, or for poorly confined foundation shafts.

$CVR = 5$ for $0.5 < S_{D1} \leq 0.6$

$CVR = 10$ for $S_{D1} > 0.6$

Setup the column vulnerability rating, CVR, to the highest value calculated from the above steps.

Abutment Vulnerability Rating (AVR).

Abutment failures during an earthquake do not usually cause total collapse of a bridge. Therefore the abutment vulnerability should be based on the damages that would temporarily prevent the access to the bridge.

Following procedure to determine the abutment vulnerability rating is based on the engineering judgment and the performance of abutments in past earthquakes.

Step-I. Assign abutment vulnerability, AVR, of 0 to bridges classified as SRC B.

Step-II. Determine the vulnerability of the structure to abutment fill settlement. The fill settlement in normally compacted approach fills may be estimated as follows.

a. One percent of the fill height when $0.24 < S_{D1} \leq 0.39$

b. Two percent of the fill height when $0.39 < S_{D1} \leq 0.49$

c. Three percent of the fill height when $S_{D1} > 0.49$

The above settlements should be doubled if the bridge is for a river crossing. If the calculated fill settlements are greater than 150mm, assign the AVR as 5. Otherwise assign AVR as 0.

Step-III. If the calculated fill settlements are greater than 150mm, assign the AVR, of 5 as in step II. Also assign AVR of 5 regardless of fill settlement if the abutment is cantilever, skew angle > 40^0 and the abutment height > 3m. Otherwise assign AVR as 0.

Liquefaction Vulnerability Rating (LVR).

There are possible types of ground failures that can results in bridge damage during an earthquake, instability resulting from liquefaction is the most significant. Therefore vulnerability rating for foundation depends on the liquefaction susceptibility and the magnitude of the acceleration coefficient.

Determination of the liquefaction vulnerability rating is based on following procedure.

Step-I. Determine the susceptibility of foundation soils to liquefaction.

High susceptibility is associated with the soils that are laterally supported to piles or vertical supports to footings, consists of saturated loose sand, silty sands or none plastic silts and those could lead to abutment slope failure.

Moderate susceptibility is associated with foundation soils that are generally medium dense soils.

Low susceptibility is associated with foundation soils that are generally dense soils.

Step-II. Determine the potential for liquefaction related damage

Table 3-9Potential for liquefaction related damage[2]

Soil susceptibility to liquefaction	Seismic coefficient S_{D1}				
	$S_{D1} \leq 0.14$	$0.14 < S_{D1} \leq 0.24$	$0.24 < S_{D1} \leq 0.39$	$0.39 < S_{D1} \leq 0.49$	$S_{D1} > 0.49$
Low	Low	Low	Low	Low	Low
Moderate	Low	Low	Moderate	Major	Sever
High	Low	Moderate	Major	Sever	Sever

Step-III. For severe bridges assign LVR of 10. This can be reduced to 5 for single span bridges with skew angle less than 20^0 and for rigid box culverts.

Step-IV. For major bridges assign LVR of 10. This can be reduced to between 5 and 9 for single span bridges with skew angle less than 40^0 and for rigid box culverts and continuous bridges with skew angle less than 20^0.

Step-V. For moderate bridges assign LVR of 5. This rating should be increased to between 6 and 10 if the vulnerability rating for bearings, V1, is greater than or equal to 5.

Step-VI. For low bridges assign LVR of 0.

3.2.5 Determination of Seismic Hazard Rating (E)

Seismic hazard includes both seismicity of the site and the geotechnical conditions. Seismic hazard rating varies from 0 to 10 and it is calculated using seismic coefficient (S_{D1}).

$$E = 10\ S_{D1} \leq 10$$

3.2.6 Determination of Bridge Rank (R)

Bridge rank is defined as the multiplication of the bridge vulnerability and the seismic hazard rating. $R = VE$

V and E are in the range of 0 to 10. Therefore R is in the range of 0 to 100. The R gives an idea about the quality of the bridge. Higher the R, the greater the need for detailed seismic evaluation and potential for retrofitting needs.

3.3 Design Checks for the Selected Bridge

The selected bridge was analysed according to the Australian Standards of design. Response spectrum analysis was used to analyze the structure. The Indian response spectrum was assigned to the model using short period and long period spectral accelerations.

Following load combination was used for

$$1.2D + 2W + 1.25EP + 1EQ$$

Where, D – Dead load

W – Load of the wearing surface

EP – Earth pressure (Soil pressure + Surcharge)

EQ – Earthquake loading

AS 5100 part 2, part5 and AS 1170 part 4 were used as design standards. In addition to the above the structure was checked to the British Standards of Design (BS 5400 part 4). When analyzing the bridge, it was used SAP 2000 v14.1.0 bridge wizard to generate the bridge model for getting the structural responses.

Bending moments and shear forces occurred due to earthquake loading of the superstructure were compared with the originally designed bending moment and shear force envelope.

Bending moments, shear forcesand torsional moments occurred due to earthquake loading of the substructure were compared with the calculated capacities of the existing structure.

The methodology discussed to find the bridge rank in this chapter is applied and find the bridge rank for the considered bridges under this study. Results of that are discussed in the next chapter.

CHAPTER 4 EVALUATION OF EXISTING BRIDGES

Methodology of evaluating the existing bridges to prepare a priority list (Bridge Rank) is discussed under the chapter 3. In this chapter, it is applied to prepare that list.

4.1 Sample calculation

Bridge No 1/1 on PeliyagodaPuttalam Road (Japanese Friendship Bridge) was evaluated as a sample calculation. Details of the bridge were taken by referring asbuilt drawings. Some extractions are attached under annex II.

Selection of the Seismic Retrofitting Category

This bridge is expected to function immediately after an earthquake. Therefore this bridge is categorized as an Essential bridge.

Year of construction of this bridge = 1992

Age of the bridge = 22 years

Anticipated service life assuming the

Design life of the bridge is 75 years = (75-age of the bridge)

= 53 years

Service life category (from table 3.1) = ASL 3 (Since anticipated service life >50)

It was considered the upper level of ground motion

This bridge is an Essential bridge and it is categorized as ASL 3

From table 3.2,

Performance level of the bridge if it is retrofitted = PL 2

The bridge was constructed on pile foundation. The piles were anchored on bed rock.

The site class of this bridge site =E

Acceleration coefficient (S1) = 0.1

From table 3.6,

F_v = 2.4

$S_{D1}=F_vS_1$

S_{D1} = 0.24

From table 3.3,

Seismic hazard level relevant to this bridge = II

From table 3.7,

Seismic retrofitting category relevant to this bridge = B

(During upper level of ground motion, PL2 and Seismic hazard level II)

Selection of bridge vulnerability

This bridge is classified as Seismic Retrofit Category (SRC) "B". Therefore the vulnerability rating for bearings, transverse restraints, and support length need to be calculated along with a rating for liquefaction effects for bridges on liquefiable soils.

The bridge is a square bridge. Hence the skew angle < 20^0

Bridge bearings are elastomeric bearings

Minimum required seat length (N)

$$N = \left[100 + 1.7L + 7.0H + 50\sqrt{1 + (2B/L)^2}\right]\frac{(1 + 1.25F_vS_1)}{cos\alpha}$$

Where, N – required seat length

L –Length of the bridge deck. (From seat to adjacent expansion joint)

H – Average height of piers/columns supporting the bridge deck.

B – Width of the deck

α – angle of skew

L = 228 m

H = 8m

B = 23.93m

$\alpha = 0^0$

$F_vS_1 = 0.24$

$$N = \left[100 + 1.7\,x\,228 + 7.0\,x\,8 + 50\sqrt{1 + (223.93/228)^2}\right]\frac{(1 + 1.25\,x\,0.24)}{cos0}$$

N = 659.13mm

Provided seat length = 1060mm

As per the section 3.2.4.1,

$V_1 = 0$

$V_T = 0$

$V_L = 0$

Therefore superstructure vulnerability (V_1) is zero

This bridge is categorized as SCR "B"

As per the section 3.2.4.2,

The Column/pier vulnerability (CVR) = 0

The Abutment vulnerability (AVR) = 0

Soil condition of this site is categorized under site class E (i.e. soil with N < 15)

Therefore this bridge has high susceptibility to fail.

S_{D1} = 0.24

From table 3.9,

The potential for liquefaction related damage of this bridge is "Moderate"

As per the section 3.2.4.2,

Liquefaction vulnerability (LVR) = 5

$V_2 = CVR + AVR + LVR \leq 10$

$V_2 = 0 + 0 + 5$

$V_2 = 5$

Bridge vulnerability = max (V_1, V_2)

= 5

Selection of bridge ranking

As per the section 3.2.5,

Seismic hazard rating (E) = 10 $S_{D1} \leq 10$

E = 10 x 0.24

E = 2.4

As per the section 3.2.6,

Bridge rank (R) = VE

R = 5 x 2.4

R = 12

4.2 Results Obtained from Bridge Ranking

As discussed in the Chapter 3, the bridges were selected to the analysis. By going through the as built drawings, data was input to get the bridge ranking. Since the unavailability of all the as built drawing, this study was limited to 40 nos of bridges. From that 40, 7 bridges felt under the seismic retrofitting category “A” and all others felt under seismic retrofitting category “B”.

Table 4-1Seismic retrofitting categories of selected bridges

Road No	Bridge No	Road Name	Bridge's Name	Siesmic Retrofitting catogory
AA001	94/7	Colombo - Kandy		B
AA002	31/3	Colombo - Galle - Hambantota - Wellawaya (CGHW)	Thalpitiya Bridge	B
AA002	42/2	Colombo - Galle - Hambantota - Wellawaya (CGHW)	Kaluthara Bridge 1	B
AA002	43/1	Colombo - Galle - Hambantota - Wellawaya (CGHW)	Kaluthara Bridge 2	B
AA002	60/1	Colombo - Galle - Hambantota - Wellawaya (CGHW)	Kaluwamodara Bridge	B
AA002	62/1	Colombo - Galle - Hambantota - Wellawaya (CGHW)	Benthota Bridge	B
AA002	62/2	Colombo - Galle - Hambantota - Wellawaya (CGHW)	Benthota Bridge	B
AA002	81/1	Colombo - Galle - Hambantota - Wellawaya (CGHW)		B
AA002	133/2	Colombo - Galle - Hambantota - Wellawaya (CGHW)	Kathalu Bridge (Pol Oya Bridge)	A
AA002	161/2	Colombo - Galle - Hambantota - Wellawaya (CGHW)	Mahanama Bridge	B
AA003	1/1	Peliyagoda - Puttalam	Japan-Sri Lanka Friendship	B
AA003	43/1	Peliyagoda - Puttalam	Gin Oya Bridge	B
AA003	94/4	Peliyagoda - Puttalam	BattuluOya Bridge	B

Road No	Bridge No	Road Name	Bridge's Name	Siesmic Retrofitting catogory
AA004	42/1	Colombo - Ratnapura - Wellawaya - Batticaloa (CRWB)	Kaluaggala Bridge	B
AA004	146/1	Colombo - Ratnapura - Wellawaya - Batticaloa (CRWB)	Oluganthota Bridge	B
AA004	157/6	Colombo - Ratnapura - Wellawaya - Batticaloa (CRWB)	BelihulOya Bridge	B
AA004	243/5	Colombo - Ratnapura - Wellawaya - Batticaloa (CRWB)		B
AA004	285/3	Colombo - Ratnapura - Wellawaya - Batticaloa (CRWB)		A
AA004	375/1	Colombo - Ratnapura - Wellawaya - Batticaloa (CRWB)	Kaliodai Bridge	B
AA005	21/4	Peradeniya - Badulla - Chenkaladi (PBC)	Gampola Bridge (New)	B
AA006	8/1	Ambepussa - Kurunegala - Trincomalee (AKT)	Alawwa New Bridge	B
AA007	1/5	Avissawella - Hatton - NuwaraEliya	Seethawaka Bridge	B
AA007	12/7	Avissawella - Hatton - NuwaraEliya	NugahamulaBokkuwa	B
AA007	14/3	Avissawella - Hatton - NuwaraEliya		B
AA007	19/6	Avissawella - Hatton - NuwaraEliya	VeeOya Bridge	B
AA008	8/2	Panadura - Nambapana - Ratnapura	Bolgoda Bridge	A
AA009	5/2	Kandy - Jaffna	Katugasthota Bridge (New)	B
AA009	309/1	Kandy - Jaffna	Kaithady Bridge	B
AA009	314/2	Kandy - Jaffna	Bailey bridge	B
AA010	48/1	Katugastota - Kurunegala - Puttalam	Maspotha Bridge	B
AA011	80/2	Maradankadawela - Habarana - Tirikkondiadimadu	Manampitiya Bridge (Peace Bridge)	B
AA012	74/3	Puttalam - Trincomalee	MALWATHU Oya Bridge	A

Road No	Bridge No	Road Name	Bridge's Name	Siesmic Retrofitting catogory
AA014	85/1	Medawachchiya - Mannar - Talaimannar	New Mannar bridge	B
AA017	46/1	Galle - Deniyaya - Madampe	Hulandawa Bridge	B
AA021	36/3	Kegalle - Bulathkohupitiya - Karawanella	Warawala Bridge	B
AA026	5/1	Kandy - Mahiyangana - Padiyatalawa	Tennakumbura Bridge	A
AA026	73/1	Kandy - Mahiyangana - Padiyatalawa	Weragantota Bridge	B
AA028	31/2	Padeniya- Anuradhapura	Siyambalangamuwa Bridge	A
AA028	48/2	Padeniya- Anuradhapura		B
AA028	75/4	Padeniya- Anuradhapura	Deduruoya bridge	A

Bridges felt under Seismic Retrofitting category “A” do not need further analysis and retrofit.

The bridges felt under the seismic retrofitting category “B” were further analyzed and the ranks obtained are listed out as follows.

Table 4-2Calculated bridge ranks of the selected bridges

Road No	Bridge No	Road Name	Bridge's Name	Bridge Rank (R=VE)
AA001	94/7	Colombo - Kandy		0
AA002	31/3	Colombo - Galle - Hambantota - Wellawaya (CGHW)	Thalpitiya Bridge	12
AA002	42/2	Colombo - Galle - Hambantota - Wellawaya (CGHW)	Kaluthara Bridge 1	12
AA002	43/1	Colombo - Galle - Hambantota - Wellawaya (CGHW)	Kaluthara Bridge 2	12
AA002	60/1	Colombo - Galle - Hambantota - Wellawaya (CGHW)	Kaluwamodara Bridge	12
AA002	62/1	Colombo - Galle - Hambantota - Wellawaya (CGHW)	Benthota Bridge	0
AA002	62/2	Colombo - Galle - Hambantota - Wellawaya (CGHW)	Benthota Bridge	0

Road No	Bridge No	Road Name	Bridge's Name	Bridge Rank (R=VE)
AA002	81/1	Colombo - Galle - Hambantota - Wellawaya (CGHW)		12
AA002	161/2	Colombo - Galle - Hambantota - Wellawaya (CGHW)	Mahanama Bridge	12
AA003	1/1	Peliyagoda - Puttalam	Japan-Sri Lanka Friendship	12
AA003	43/1	Peliyagoda - Puttalam	Gin Oya Bridge	24
AA003	94/4	Peliyagoda - Puttalam	BattuluOya Bridge	0
AA004	42/1	Colombo - Ratnapura - Wellawaya - Batticaloa (CRWB)	Kaluaggala Bridge	12
AA004	146/1	Colombo - Ratnapura - Wellawaya - Batticaloa (CRWB)	Oluganthota Bridge	0
AA004	157/6	Colombo - Ratnapura - Wellawaya - Batticaloa (CRWB)	BelihulOya Bridge	8
AA004	243/5	Colombo - Ratnapura - Wellawaya - Batticaloa (CRWB)		0
AA004	375/1	Colombo - Ratnapura - Wellawaya - Batticaloa (CRWB)	Kaliodai Bridge	12
AA005	21/4	Peradeniya - Badulla - Chenkaladi (PBC)	Gampola Bridge (New)	12
AA006	8/1	Ambepussa - Kurunegala - Trincomalee (AKT)	Alawwa New Bridge	0
AA007	1/5	Avissawella - Hatton - NuwaraEliya	Seethawaka Bridge	12
AA007	12/7	Avissawella - Hatton - NuwaraEliya	NugagahamulaBokkuwa	8
AA007	14/3	Avissawella - Hatton - NuwaraEliya		12
AA007	19/6	Avissawella - Hatton - NuwaraEliya	VeeOya Bridge	12
AA009	5/2	Kandy - Jaffna	Katugasthota Bridge (New)	12
AA009	309/1	Kandy - Jaffna	Kaithady Bridge	0
AA009	314/2	Kandy - Jaffna	Bailey bridge	0
AA010	48/1	Katugastota - Kurunegala - Puttalam	Maspotha Bridge	0
AA011	80/2	Maradankadawela - Habarana - Tirikkondiadimadu	Manampitiya Bridge (Peace Bridge)	12

Road No	Bridg No	Road Name	Bridge's Name	Bridge Rank (R=VE)
AA014	85/1	Medawachchiya - Mannar - Talaimannar	New Mannar bridge	0
AA017	46/1	Galle - Deniyaya - Madampe	Hulandawa Bridge	17
AA021	36/3	Kegalle - Bulathkohupitiya - Karawanella	Warawala Bridge	0
AA026	73/1	Kandy - Mahiyangana - Padiyatalawa	Weragantota Bridge	12
AA028	48/2	Anuradhapura - Padeniya		0

Since the vulnerability (V) and the seismic hazard rating (E) are varied from 0 to 10, the maximum value of the bridge rank is 100. The maximum value get for the bridge rank from this analysis is 24 and it is for the Gin Oya Bridge on PeliyagodaPuttlam road (AA-003). Second highest value is 17 and that is for "Hulandawa Bridge" on Galle DeniyayaMadampe road. There are proposals to reconstruct these two bridges in near future. The value of Bridge No1/1 on PeliyagodaPuttalam road (Japanese Friendship Bridge) is 12 and it has been taken for the case study to do a detailed analysis and capacity demand check for structural elements of the bridge since it is very important bridge near commercial capital of the country and its remaining design life is more than 75 years.

As discussed in the objectives, the study includes a case study for a selected bridge from the developed bridge rank. The bridge no 1/1 on PeliyagodaPuttalam road (Japanese Friendship Bridge) was selected for detailed seismic evaluation based on the developed bridge rank. This bridge was analyzed using a Finite element model developed using SAP 2000 Vr. 14.1.0 and the capacities of the elements of the bridge was checked. Details of that are discussed in the next chapter of this report.

CHAPTER 5 CASE STUDY

5.1 General

Bridges were ranked in chapter 4 to find the priority of the bridges to further investigate and retrofit if necessary. In this study theBridge no 1/1 on PeliyagodaPuttalam road (Japanese Friendship Bridge) was selected to do a detailed structural analysis and find the seismic capacity of it.

5.2 Structural Analysis

Finite element model was developed using SAP 2000 vr.14.1.0. In that model, structural idealization of each element of the bridge is shown in the table 5.0.The three dimensional SAP 2000 model is shown in fig. 5.1.

Table 5-1Structural Idealization of element of the bridge

	Structural Idealization
Superstructure	Superstructure was defined using the Bridge wizard of the SAP 2000. Bearings are represented using link elements
Substructure	Abutments, wingwalls and piers are modeled using area elements
Foundation	Winkler models are used for foundations Pile caps are modeled using shell elements Piles are modeled using frame elements and the soil is modeled using springs

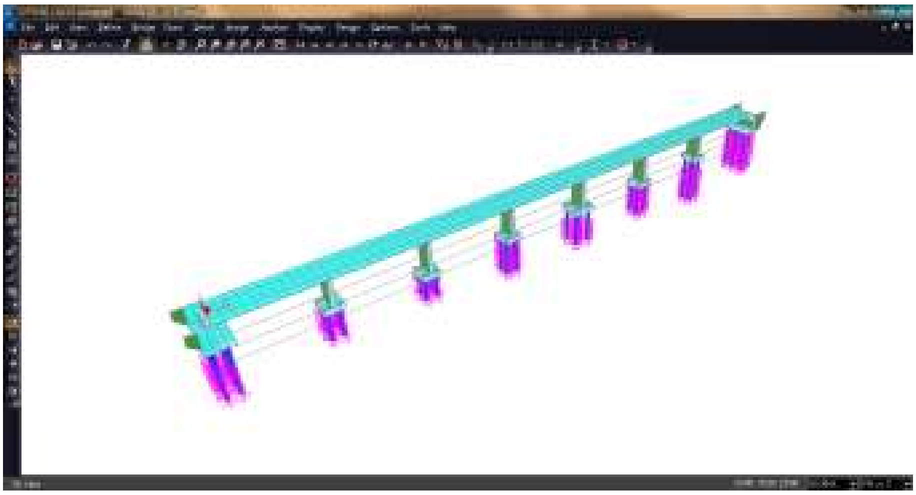

Figure 5-1 Finite Element Model of the Bridge

Earthquake loadings are selected as per the clause 14.2 of AS 5100.2-2004 to calculate the ultimate limit state actions of the elements of the bridge.

Bridge classification

The bridge is located at the city of Colombo on a national road connecting two districts. Other than that it carries lifelines such as water, electricity supplies. Hence this bridge is an essential bridge that requires to post earthquake recovery. Therefore this bridge is classified as Type III as per the clause 14.3.2 of AS 5100.2-2004.

Acceleration coefficient (a)

As mentioned in the section 2.4, peak ground acceleration is taken as 0.1g. Therefore the acceleration coefficient (a) is 0.1

$$a = 0.1$$

Site factor (S)

As per the as built drawings, soil profile at this site contains 6 to 12m silt and loose sand. Therefore the site factor for this site is selected as 1.5 from the table 2.4(a) of the AS 1170.4-1993.

$$S = 1.5$$

Therefore, $aS = 0.15$

As per the table 14.3.1 of AS 5100.2-2004, Bridge earthquake design category is BEDC-3

Therefore horizontal and vertical earthquake loads shall be considered for analysis. As per the clause 14.6 of AS 5100.2-2004, the bridges categorized under BEDC-3 should be analyzed using response spectrum analyze method or time history analyze method.

As discussed in the chapter 2, Indian response spectrum is used to do this case study.

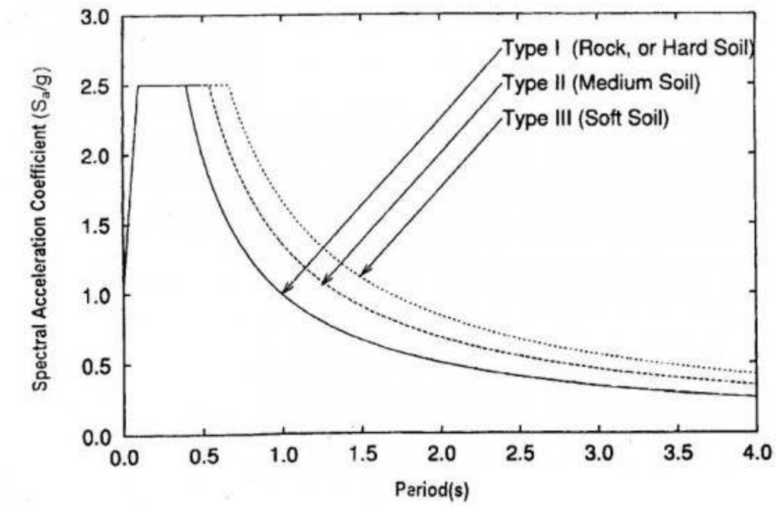

Figure 5-2Indian response spectrum[18]

Definitions of the response spectrum and analysis cases are explained in Annex II.

5.3 Results

Modal Analysis

Using first 50 numbers of free modes of vibration around 99% participation mass ratio could be obtained from all load combinations.

Table 5-2 Modal Contribution to Participating Mass Ratio

Direction of base reaction	Mode number	Nature of Mode	Period (s)	Participant Mass Ratio
X	1	Longitudinal displacement	1.153	0.48
	3	Longitudinal displacement	0.361	0.09
	2	Longitudinal displacement	0.530	0.08
Y	50	Transverse displacement + Bending	0.004	0.53
	2	Longitudinal displacement	0.53	0.06
	47	Transverse displacement + Bending	0.012	0.06
Z	16	Bending	0.149	0.30
	35	Bending	0.052	0.21
	37	Bending	0.048	0.099

Graphical representations of dominant modal shapes are given in annex III.

Sample Calculations

Reference	Description	Out put
	Check for Flexure for Australian Standards	
AS 5100-5 Cl.8.1.3 & Cl.8.1.4	for $k_u \leq 0.4$, design strength in bending = Φ M_{uo}	
	$M_{uo} = 1.2\left\{z\left(f_{cf'} + \frac{P}{A_g}\right) + Pe\right\}$	
	From 1st principles	
	$k_u = \frac{0.003 A_s E_s}{0.85 f_c' b d \gamma} \times (1 - k_u)$	
	Where,	
	M_{uo} Ultimate strength in bending without axial forces	
	Z Section modulus of the uncracked section	
	f_{cf}' Characteristic flexural strength of the concrete	
	P Prestressing force	
	A_g Gross cross sectional area of the member	
	e Eccentricity of the prestressing force	
	k_u Neutral axis parameter	
	Where $\gamma = [0.85 - 0.007(f_c' - 28)]$	
	Therefore $k_u = \frac{\left\lfloor\sqrt{(\alpha^2 + 4\alpha)}\right\rfloor - \alpha}{2}$	
	Results of the P1 from the FEM modal were taken for this calculation.	
	Pier stem	
	Design Bending moment = 424kNm/m	
	width of the section (mm) = 1000	
	Depth of the section (mm) = 1500	
	Cover to r/f (mm) = 110	
	Diameter of main r/f (mm) = 32	
	Spacing of the main r/f (mm) = 125	

A_s Provided (mm^2) = 6433.98

f_c' (N/mm^2) = 30

f_{cf}' (N/mm^2) = $0.6\sqrt{f_c'}$ = 3.29

f_y (N/mm^2) = 340

E_s (kN/mm^2) = 200

d (mm)=(1500-110-32/2) = 1374

γ = [0.85-0.007(30-28) = 0.836

$$\alpha = \frac{0.003 A_s E_s}{0.85 f_c' b d \gamma} = \frac{0.003 \times 6433.98 \times 200000}{0.85 \times 30 \times 1000 \times 1374 \times 0.836}$$

α = 0.13

$$k_u = \frac{\left\lfloor \sqrt{\left(0.13^2 + 4 \times 0.13\right)} \right\rfloor - 0.13}{2}$$

k_u = 0.30< 0.4

I = 1.25E+11

d_{NA} = 500

Z = 250000000

P = 0

e = 0

A_g = 1500000

$$M_{uo} = 1.2\left\{ z\left(f_{cf'} + \frac{P}{A_g} \right) + Pe \right\} = 1.2 \times \left\{ 3.29 + \frac{0}{1500000} \right\} + 0$$

M_{uo} = 985.90kNm/m

ϕ = 0.8

$\phi\ M_{uo}$ = 788.72kNm/m

$M_{applied}$ = 424.00kNm/m

Check for Shear for Australian Standards

Design Shear force = 330.1kN/m

Design shear strength = ϕV_u

$V_u = V_{uc} + V_{us}$

Bending Capacity 788.72kNm/m

Where,

V_{uc} Shear strength excluding shear r/f

V_{us} Shear strength contributed by shear r/f

$$V_{uc} = \beta_1 \beta_2 \beta_3 b_v d_0 \left[\frac{A_{st} f_c'}{b_v d_0} \right]^{\frac{1}{3}}$$

Where, $$\beta_1 = 1.1\left[1.6 - \frac{d_0}{1000}\right] \geq 1.1$$

$$\beta_2 = 1.0$$

or

$$1 - \left[\frac{N^*}{3.5A_g}\right] \geq 0$$

for members subjected to axial tension

$$1 + \left[\frac{N^*}{14A_g}\right] \geq 0$$

for members subjected to axial compression

$\beta 3$ = 1.0 or

= $2d_0/a_v$ but not greater than 2

$$V_{us} = \frac{A_s f_{sy,f} d_0}{s} Cot\theta_v$$

In abutments & Piers it was not used shear reinforcem

Therefore V_{us} will be zero

Applied Shear Force V (kN) = 207

AS 5100.5 Cl. 8.2

d_0 (mm) = 1,374

$$\beta_1 = 1.1\left[1.6 - \frac{1374}{1000}\right] = 0.248 < 1.1$$

$\beta 1$ = 1.1

Ag (mm^2) = 1,500,000

N* = 0

β_2 = 1 (Since N* = 0)

β_3 = 2

AS 5100.5 Cl. 8.2.7	b_v (mm)	= 1,000	
	f'_c (N/mm^2)	= 30.0	
	A_{st}(mm^2)	= 6,434	
	V_{uc} (kN)	= 1,571	
	V_{us} (kN)	= 0	
AS 5100.5 Cl. 8.2.10	V_u (kN)	= 1,571	Shear Capacity 1100kN/m
	ϕ	= 0.7	
	ϕVu (kN)	= 1,100>Applied shear force	

5.4 Summary of the Results of Case Study

Japanese Friendship Bridge was analyzed using SAP 2000 vr. 14.1.0 and maximum load effects derived from the analysis and the calculated capacities of the elements are as follows.

Substructure

Table 5-3 Applied bending moments &calculated bending capacities of Pier & Abutment stems

	Ultimate Bending Moment (kNm)	Moment capacity According to British Standards (kNm)	Moment capacity According to Australian Standards (kNm)
P_1	424.00	2484.21	788.72
P_2	423.00	2484.21	788.72
P_3	418.00	2484.21	788.72
P_4	320.00	2484.21	788.72
P_5	410.00	2484.21	788.72
P_6	428.00	2484.21	788.72
A_1	246.00	1124.21	1130.50
A_2	214.00	1124.21	1130.50

Table 5-4 Applied Shear forces &Calculated Shear capacities of Pier & Abutment Stems

	Ultimate Shear Force (kN)	Ultimate Shear Stress (N/mm^2)	Shear capacity According to British Standards (N/mm^2)	Shear capacity According to Australian Standards (kN)
P_1	207.32	0.15	0.40	1099.97
P_2	236.35	0.17	0.40	1099.97
P_3	228.85	0.17	0.40	1099.97
P_4	330.10	0.24	0.40	1099.97
P_5	239.07	0.17	0.40	1099.97
P_6	231.11	0.17	0.40	1099.97
A_1	164.00	0.08	0.22	963.02
A_2	133.00	0.07	0.22	963.02

Table 5-5 Applied bending moments & calculated bending capacities of pile caps

	Ultimate Bending Moment (kNm)	Moment capacity According to British Standards (kNm)	Moment capacity According to Australian Standards (kNm)
P_1	1021.00	2799.78	1051.63
P_2	972.00	2799.78	1051.63
P_3	963.00	2799.78	1051.63
P_4	1025.00	3406.30	1051.63
P_5	944.00	2799.78	1051.63
P_6	998.00	2799.78	1051.63
A_1	427.00	1041.45	1051.63
A_2	429.00	1041.45	1051.63

Table 5-6 Applied Shear forces & Calculated Shear capacities of pile caps

	Ultimate Shear Force (kN)	Ultimate Shear Stress (N/mm^2)	Shear capacity According to British Standards (N/mm^2)	Shear capacity According to Australian Standards (kN)
P_1	579.76	0.27	0.32	1156.41
P_2	518.31	0.26	0.32	1156.41
P_3	494.97	0.26	0.32	1156.41
P_4	666.73	0.35	0.34	1234.19
P_5	515.25	0.27	0.32	1156.41
P_6	544.90	0.29	0.32	1156.41
A_1	527.41	0.28	0.23	831.96
A_2	429.18	0.23	0.23	831.96

Superstructure

Bending moments

Table 5-7 Calculated superstructure moments

	From EQ Analysis	
Distance (m)	M_{Max} (kNm)	M_{Min} (kNm)
0.0	-1161.40	-1285.82
14.5	18471.69	18369.20
17.5	16226.50	16124.46
32.0	-25848.91	-25986.73
46.8	8478.32	8407.36
49.7	8926.53	8861.96
64.5	-20638.19	-20741.03
79.3	10814.45	10786.41
82.2	10719.46	10687.04
97.0	-21587.56	-21659.47
111.8	10252.18	10173.11
114.7	10224.72	10138.53
129.5	-21754.94	-21873.69
144.3	10718.59	10637.40
147.2	10795.12	10722.31
162.0	-20611.59	-20745.49
176.8	8902.91	8817.13
179.7	8412.54	8326.42
194.5	-25868.29	-25979.87
209.0	16765.07	16597.12
212.0	19112.67	18896.79
226.5	-24.25	-467.31

Table 5-8Superstructure moments extracted from original Design report

	From Original Analysis	
Distance (m)	M_{Max} (kNm)	M_{Min} (kNm)
0	0	0
10.67	22955	15755
16	22655	14619
26.67	-3600	-9000
32	-22555	-29257
48.25	12918	5480
64.5	-15982	-23024
80.75	15264	7905
97	-17584	-24619
113.25	15077	7690
129.5	-17584	-24620
145.75	15264	7876
162	-15985	-23023
178.25	12919	5486
194.5	-22555	-29254
199.83	-3600	-9000
210.5	22661	14619
215.83	22957	15757
226.5	0	0

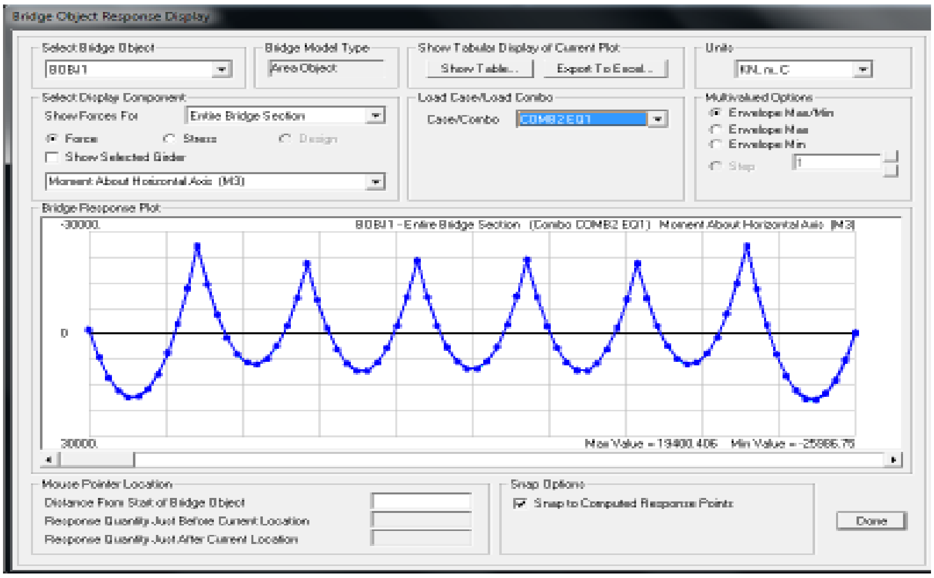

Figure 5-3 Bending moment envelope of the superstructure for seismic loadings

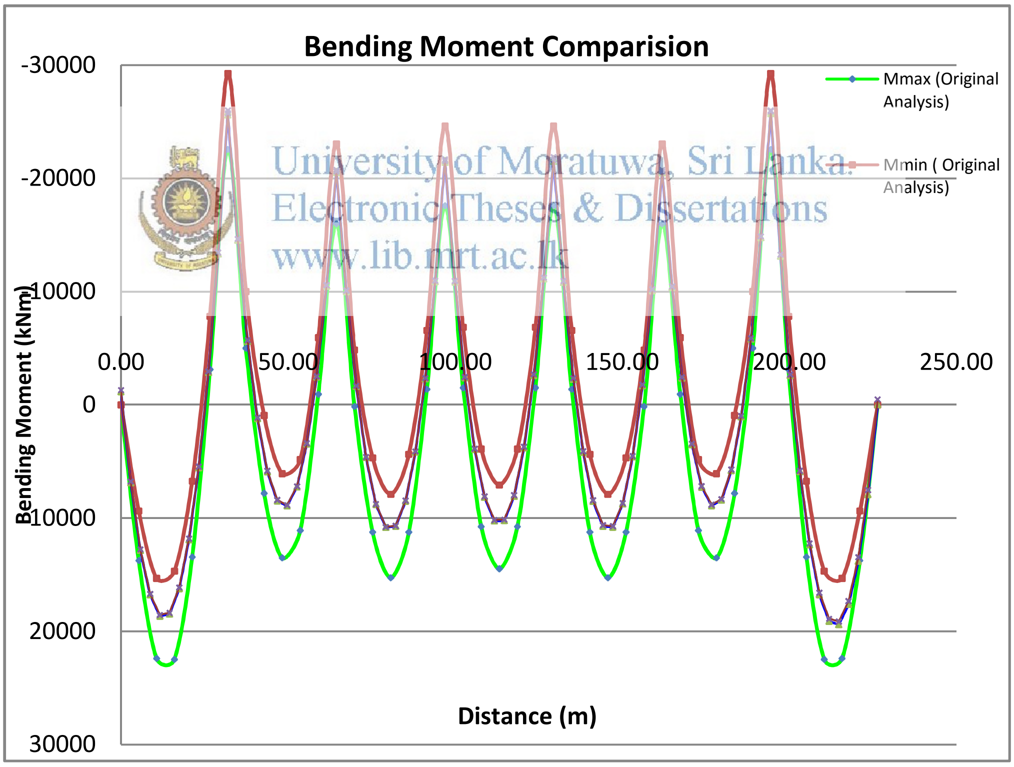

Figure 5-4 Comparison of the bending moment envelope of the superstructure for seismic loading with the originally designed bending moment envelope

Shear forces

Table 5-9 Calculate and originally designed shear forces of the superstructure

	Original Design		From EQ Analysis	
Distance (m)	SF_{max}(kN)	SF_{min} (kN)	SF_{max}(kN)	SF_{min} (kN)
0	-3014	-4008	-3110.45	-3127.65
32	5272	4121	4671.92	4652.95
32	-3530	-4760	-4097.78	-4110.85
64.5	4389	3141	3805.69	3791.81
64.5	-3278	-4525	-3915.01	-3922.27
97	4629	3380	3987.42	3981.43
97	-3333	-4582	-3933.92	-3948.16
129.5	4581	3332	3968.27	3955.79
129.5	-3380	-4628	-3972.29	-3985.42
162	4534	3287	3931.76	3916.66
162	-3141	-4388	-3778.87	-3792.15
194.5	4760	3530	4124.51	4110.61
194.5	-4120	-5268	-4674.75	-4704.23
226.5	4011	3015	3103.48	3080.51

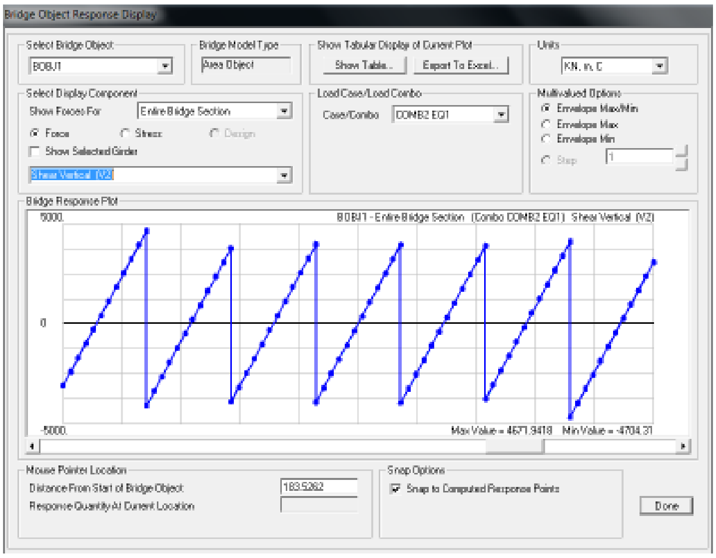

Figure 5-5 Shear force envelope of the superstructure for seismic loadings

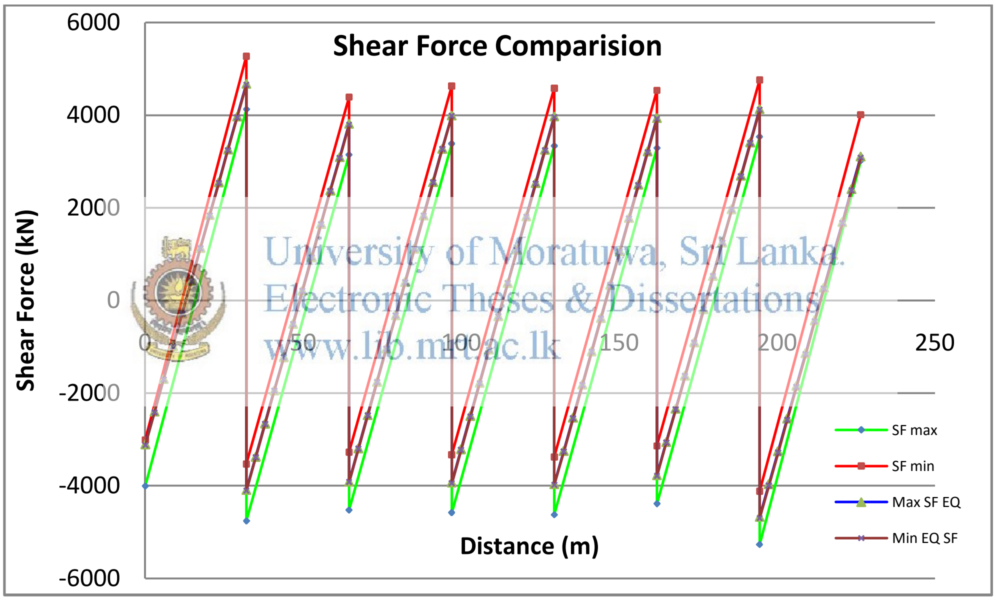

Figure 5-6 Comparison of shear force envelope of the superstructure for seismic loading with the originally designed shear force envelope

Bridge Bearings

Maximum displacement at piers	– 4.2mm
Maximum rotation at piers	– 0.0025mm
Maximum displacement at abutment	– 13.3mm
Maximum rotation at abutment	– 0.0025mm
Maximum axial force at SLS on bearing at piers	– 3619kN
Shear force at Pier P_4	– 80.09kN

Table 5-10 Results of the bridge bearing checks

Location	Applied Shear Strain	Allowable Shear Strain	Applied Compressive Stress	Allowable Compressive Stress	Applied Rotational limitations	Allowable Rotational limitations
Piers	1.23	2.89	5.10	15.00	0.83	0.81
Abutment	0.96	2.89	5.04	15.00	0.47	1.58

It also satisfied the overall stability test of the bearings. Since the rotational limitation of the bridge bearings of the piers are fail, it is necessary to replace those to strengthen it.

CHAPTER 6 CONCLUSION

There are more than 4000 bridges on National highways in the country. Those bridges are not designed to cater for seismic effects. Therefore it is necessary to find the response of these bridges with respect to possible earthquake risks.

The methodology proposed in the Seismic Retrofitting Manual for Highway Bridges Published by the Federal Highway Administration (Report No. FHWA-RD-94-052) is used to rank the bridges to identify the priority of those to retrofit. Higher the rank implies that detailed evaluation required for retrofitting.

The ranking of the bridges felt between 0 and 24 on the scale of 100. Therefore the bridges consider under this study has low risk to fail due to an earthquake considered under this study.

The bridge rank of the bridge called “Japanese Friendship Bridge” is 12 and it has been analyzed to possible earthquake loading. The analysis and design of the bridge (accordance with AS 5100) has indicated that the bridge bearings need to be replaced.

It is recommended to carry out similar study for the all national highway bridges and take appropriate measures to reduce possible seismic risks under local conditions.

Also it is recommended to do proper earthquake resisting detailing to enhance the earthquake resisting capacity of the bridges that will be constructed in the future.

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APPENDIX – A

SELECTION OF SEISMIC RETROFITTING CATEGORY

APPENDIX – B

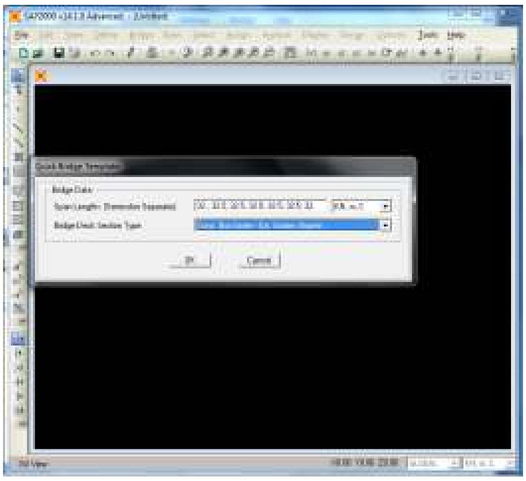
PREPARATION OF BRIDGE MODEL USING SAP 2000 VR. 14.1.0

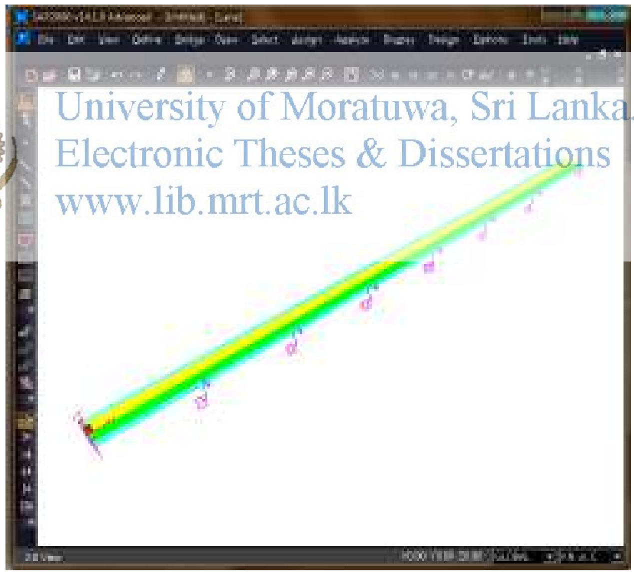
PREPARATION OF BRIDGE MODEL USING SAP 2000 Vr. 14.1.0

Some Important Steps of Building of the FEM

SAP 2000 version 14.1.0 was used to prepare the bridge model.

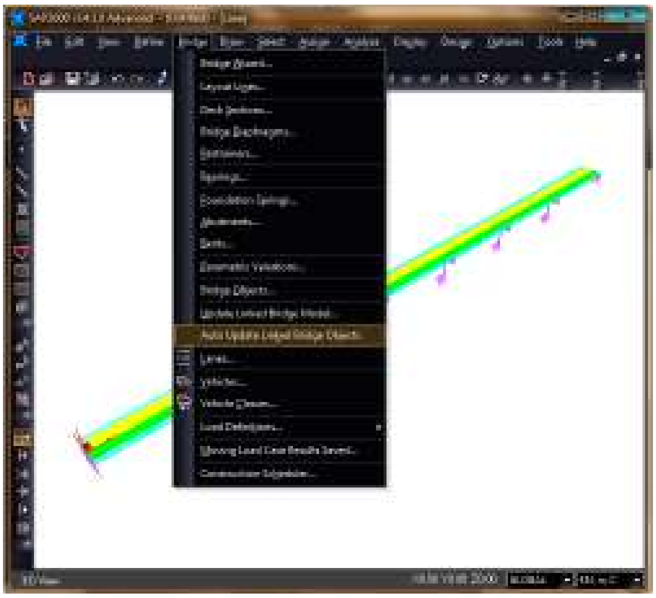
File ⟶ New Model ⟶ Quick Bridge



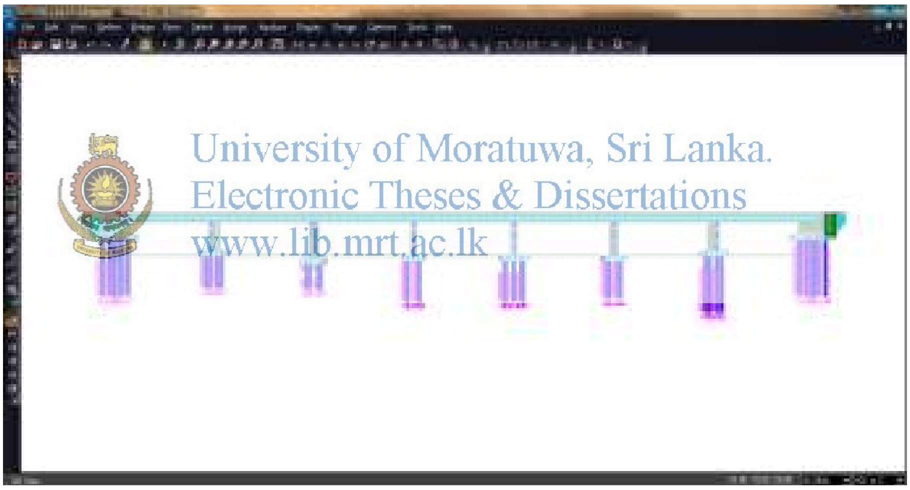


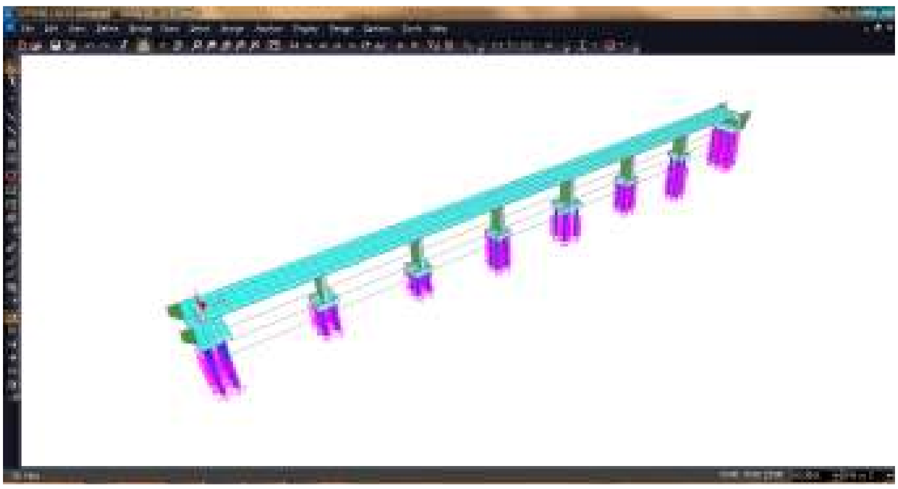
Once it is prepared the primary modal the geometry and the material properties can be changed as you wish using Bridge wizard. In the bridge modeler wizard, it can be defined and modified all the material properties, section properties and also it can be assigned the same.

In this case study, only the superstructure was defined using the bridge wizard and substructure was defined and connected to the superstructure manually using area elements (for pile caps, abutments, piers and wing walls), frame elements (for abutment cap, pier cap and piles) and link elements (for bearings). Also make sure to offline the “Auto update linked bridge objects” in the bridge menu of the SAP 2000.

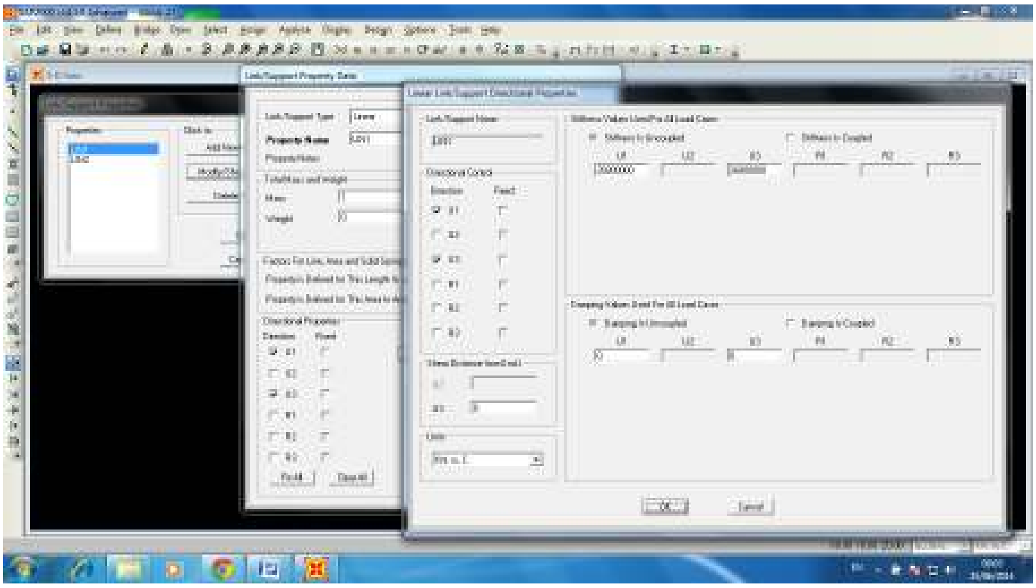


After completing model building, it was as follows.

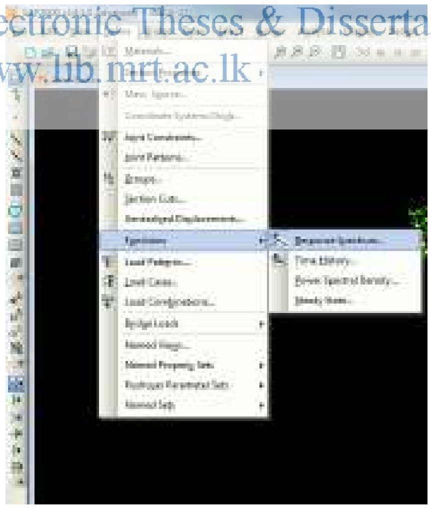


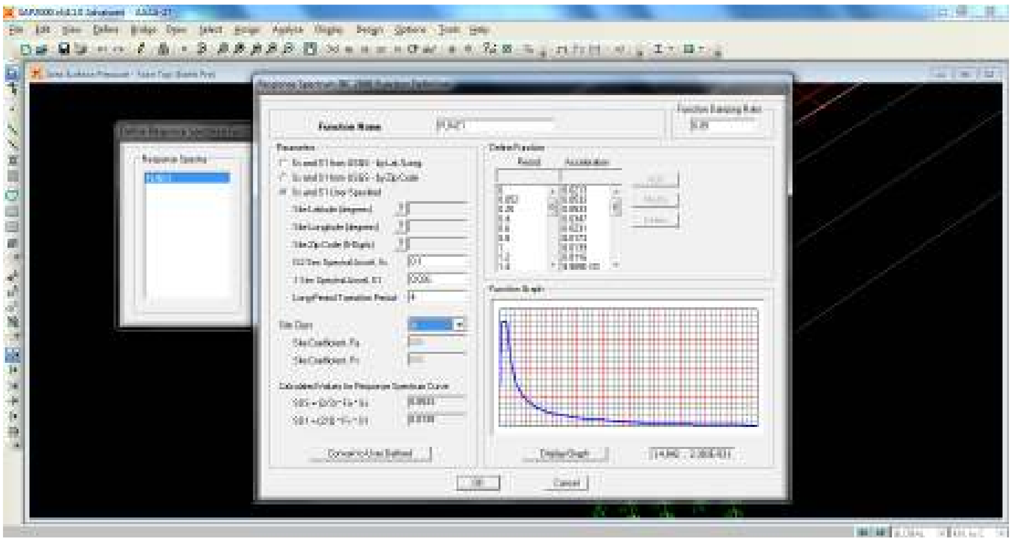


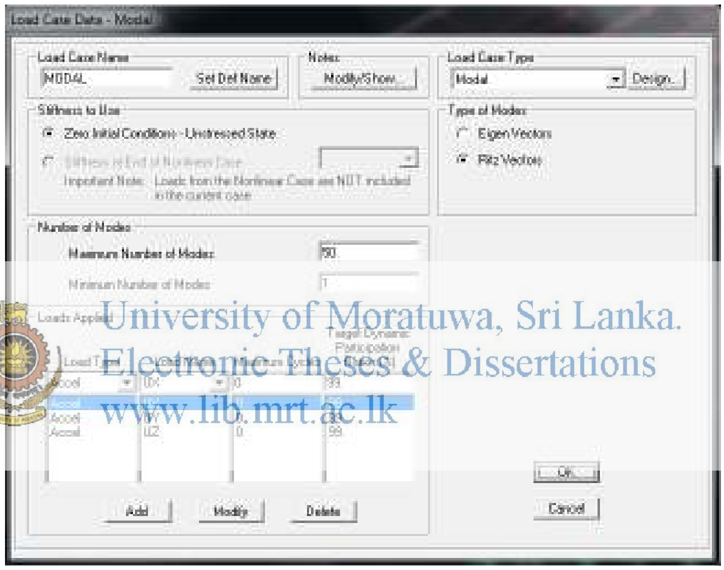
When it defines the link object properties to define the bearings, two objects were defined to get the fixed and free connections.

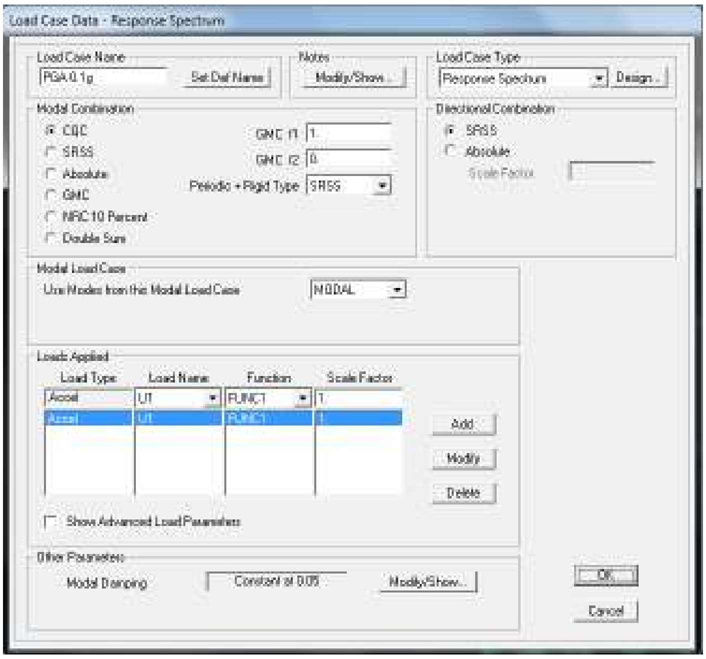


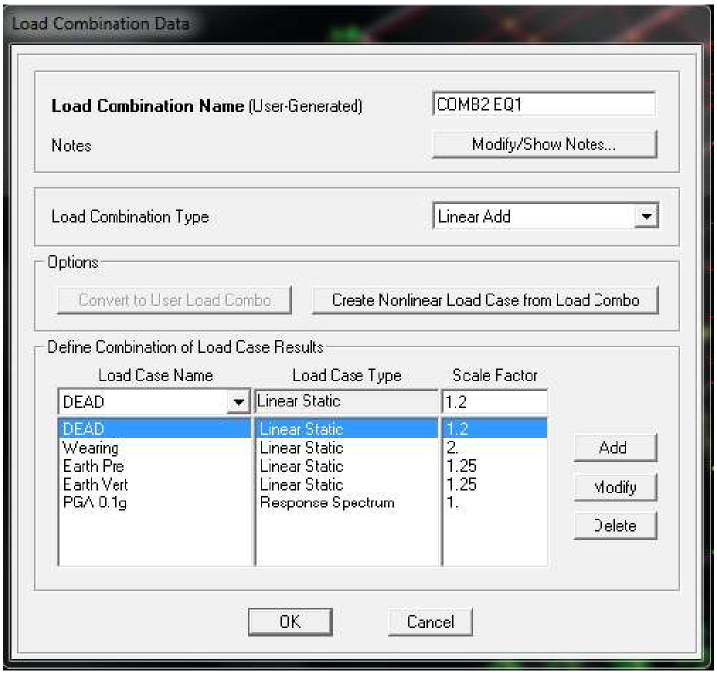
Soil properties were assigned to the model using springs. The values of the springs were taken using the N values (1500N). The N values were extracted from the as built drawings. The drawing was annexed.




Function Name
Function Graph
Display Graph
OK
Cancel


Load Case Data - Modal
Load Case Name
MODAL
Set Def Name
Notes
Modify/Show...
Load Case Type
Modal
Design...
Stiffness to Use
Zero Initial Conditions - Unstressed State
Stiffness at End of Nonlinear Case
Important Note: Loads from the Nonlinear Case are NOT included in the current case
Type of Modes
Eigen Vectors
Ritz Vectors
Number of Modes
Maximum Number of Modes
50
Minimum Number of Modes
1
Loads Applied
Load Type
Target Dynamic Participation Ratio (%)
Accel
UX
0
99
Accel
Accel
Accel
UZ
0
99
Add
Modify
Delete
OK
Cancel


Load Case Data - Response Spectrum
Load Case Name
PGA 0.1g
Set Def Name
Notes
Modify/Show...
Load Case Type
Response Spectrum
Design...
Modal Combination
CQC
SRSS
Absolute
GMC
NRC 10 Percent
Double Sum
GMC f1
1.
GMC f2
0.
Periodic + Rigid Type
SRSS
Directional Combination
SRSS
Absolute
Scale Factor
Modal Load Case
Use Modes from this Modal Load Case
MODAL
Loads Applied
Load Type
Load Name
Function
Scale Factor
Accel
U1
FUNC1
1.
Accel
U1
FUNC1
1.
Add
Modify
Delete
Show Advanced Load Parameters
Other Parameters
Modal Damping
Constant at 0.05
Modify/Show...
OK
Cancel


Load Combination Data
Load Combination Name (User-Generated)
COMB2 EQ1
Notes
Modify/Show Notes...
Load Combination Type
Linear Add
Options
Convert to User Load Combo
Create Nonlinear Load Case from Load Combo
Define Combination of Load Case Results
Load Case Name
Load Case Type
Scale Factor
DEAD
Linear Static
1.2
DEAD
Linear Static
1.2
Wearing
Linear Static
2.
Earth Pre
Linear Static
1.25
Earth Vert
Linear Static
1.25
PGA 0.1g
Response Spectrum
1.
Add
Modify
Delete
OK
Cancel

APPENDIX – C

RESULTS OBTAINED FROM BRIDGE MODEL DEVELOPED USING SAP 2000 VR.14.1.0

RESULTS OBTAINED FROM BRIDGE MODEL DEVELOPED USING SAP 2000 Vr. 14.1.0

Modal Analysis

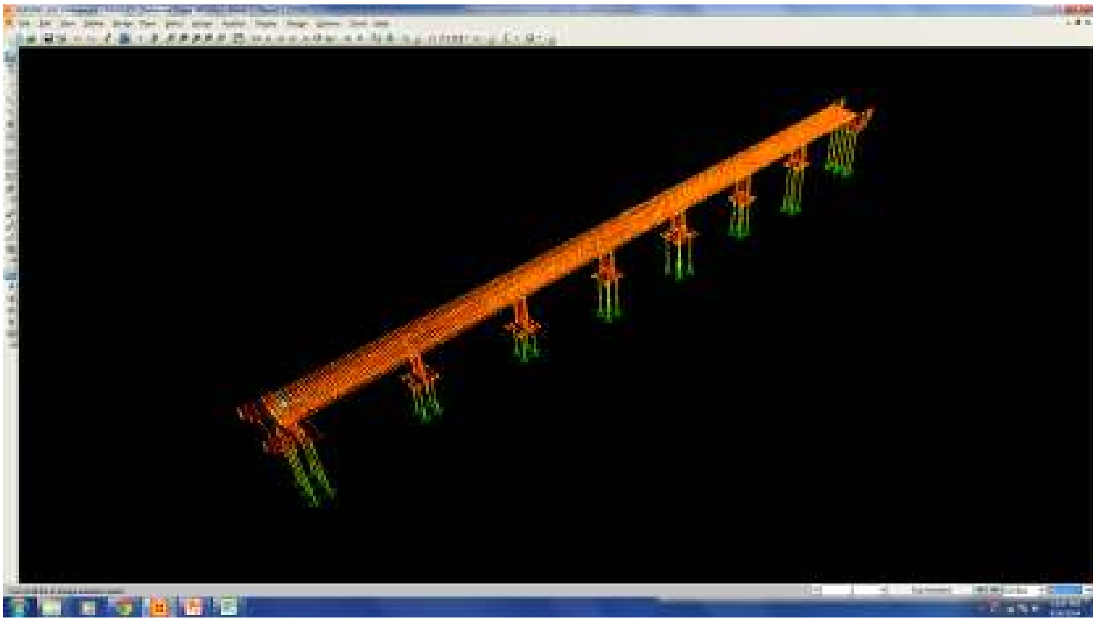


Fig Aiii-1; Mode No.1 – translation mode

Fig Aiii-2; Mode No.8 – Bending mode

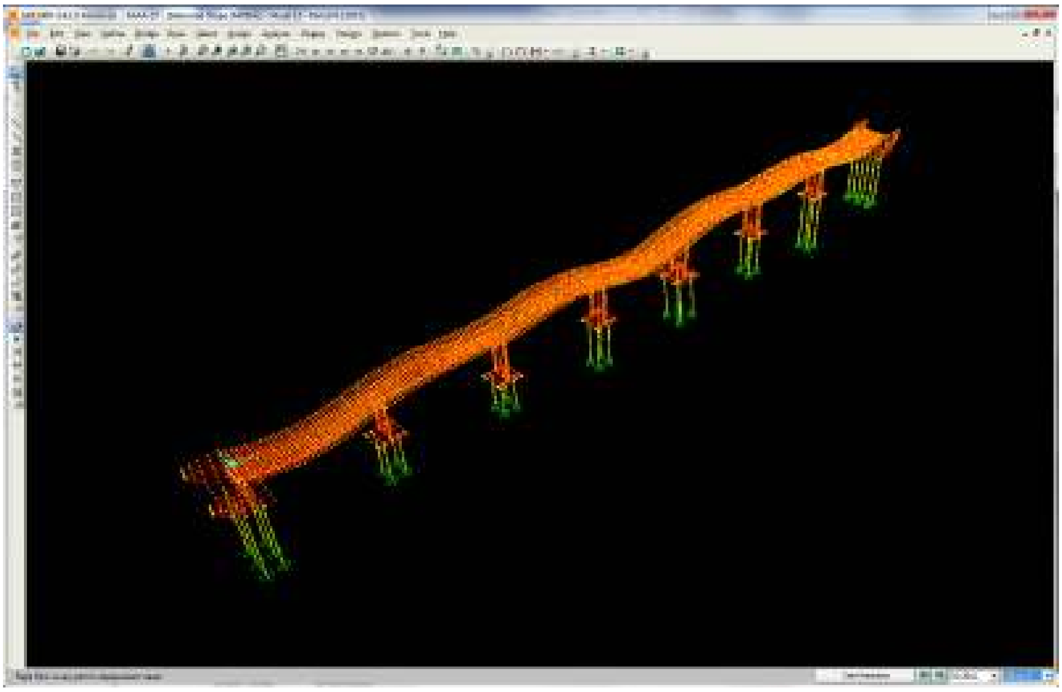


Fig Aiii-3; Mode No.12 – Bending mode

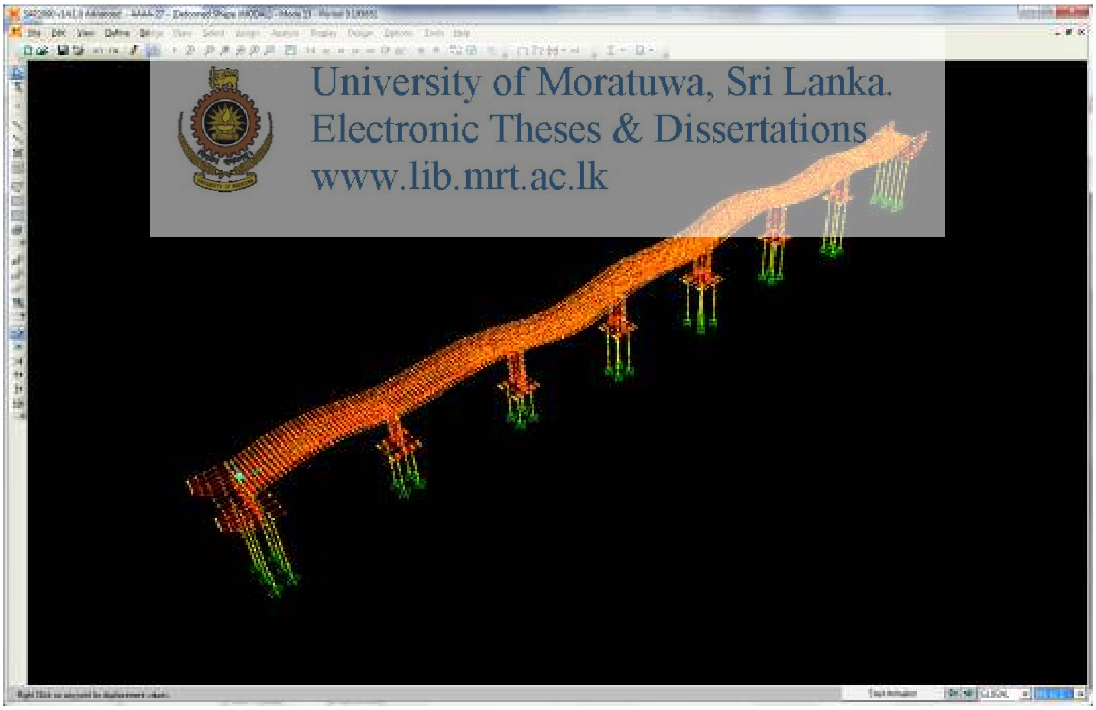


Fig Aiii-4; Mode No.13 – Bending mode

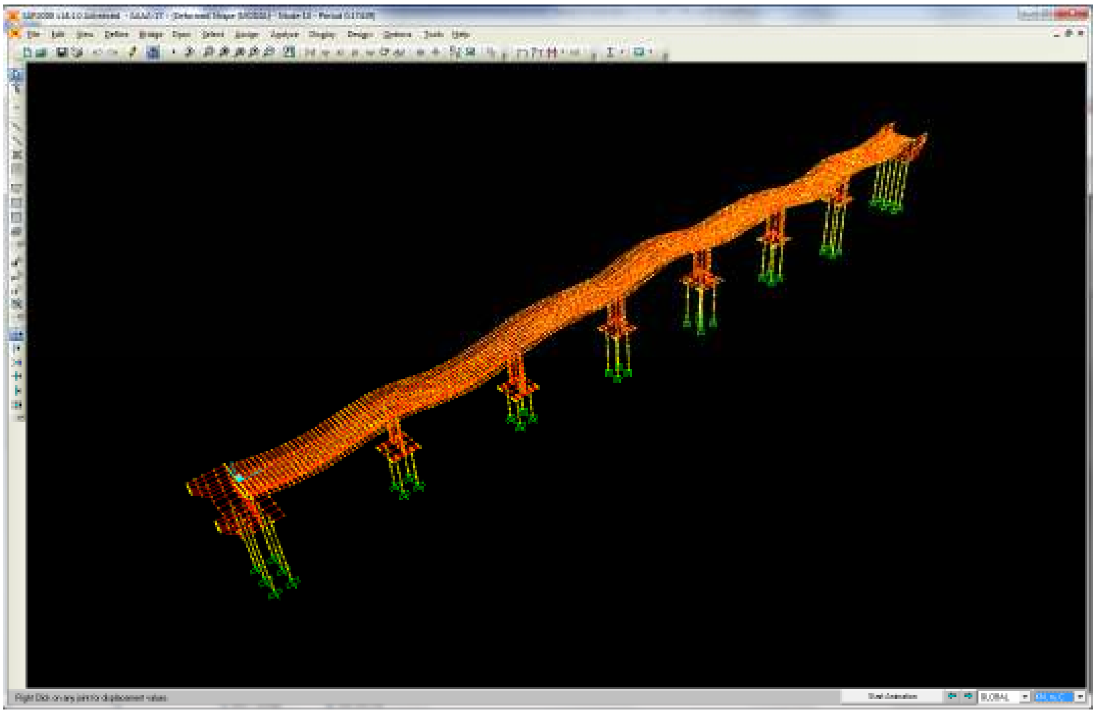


Fig Aiii-5; Mode No.14 – Bending mode

Fig Aiii-6; Mode No.41 – Bending mode

Results (Superstructure)

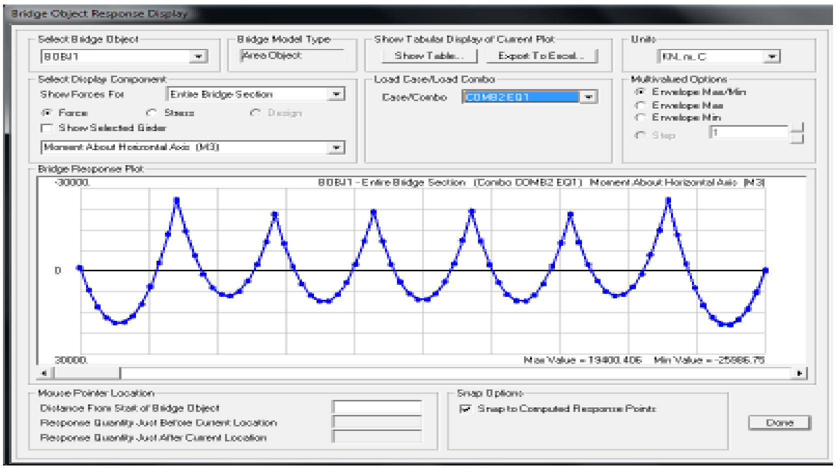


Fig Aiii-7; Bending moment envelope (Com2 EQ1)

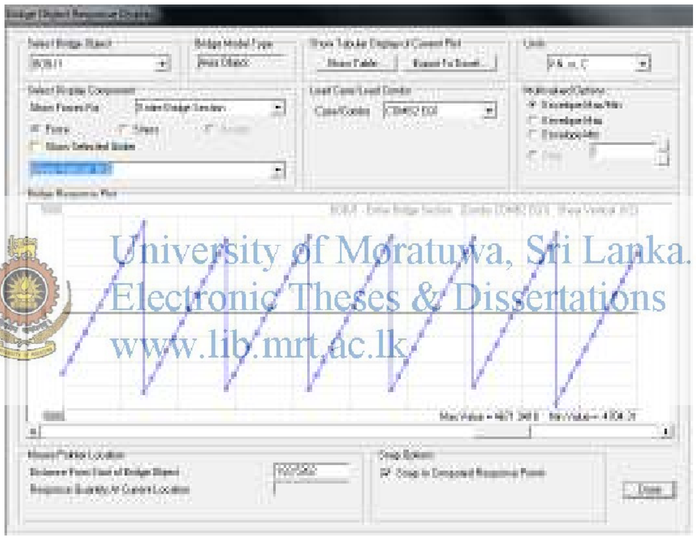


Fig Aiii-8; Shear force envelope (Com2 EQ1)

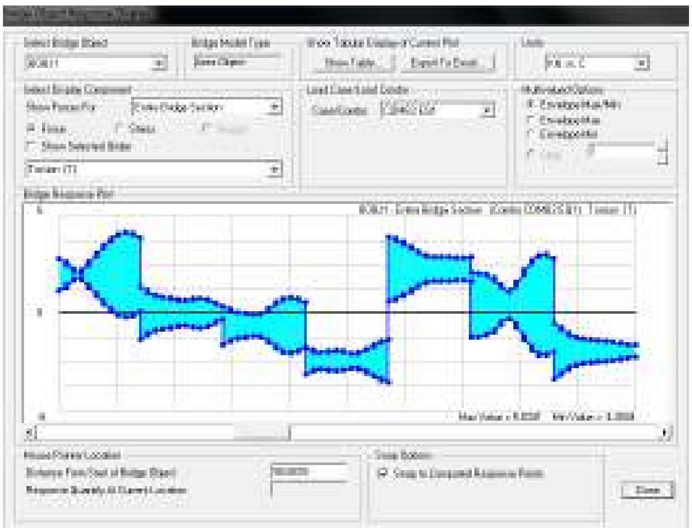


Fig Aiii-9; Torsion envelope (Com2 EQ1)

Results (Substructure)

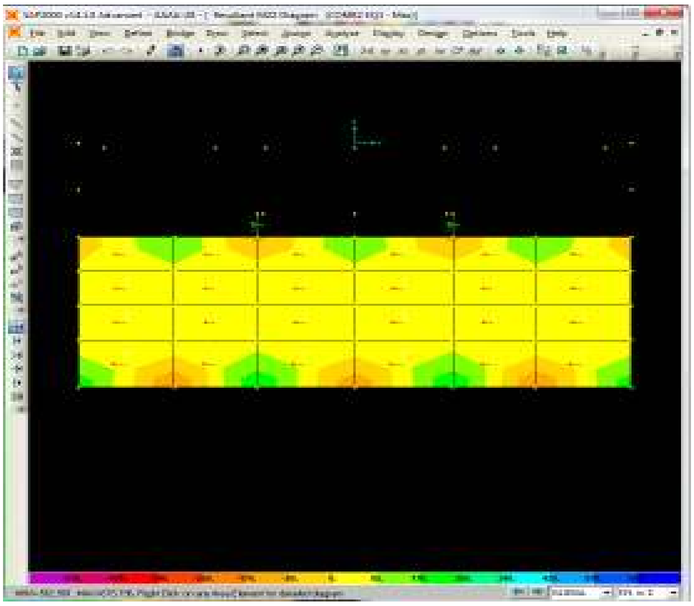


Fig Aiii-10; Bending moment distribution - Abutment A1 (Com2 EQ1)

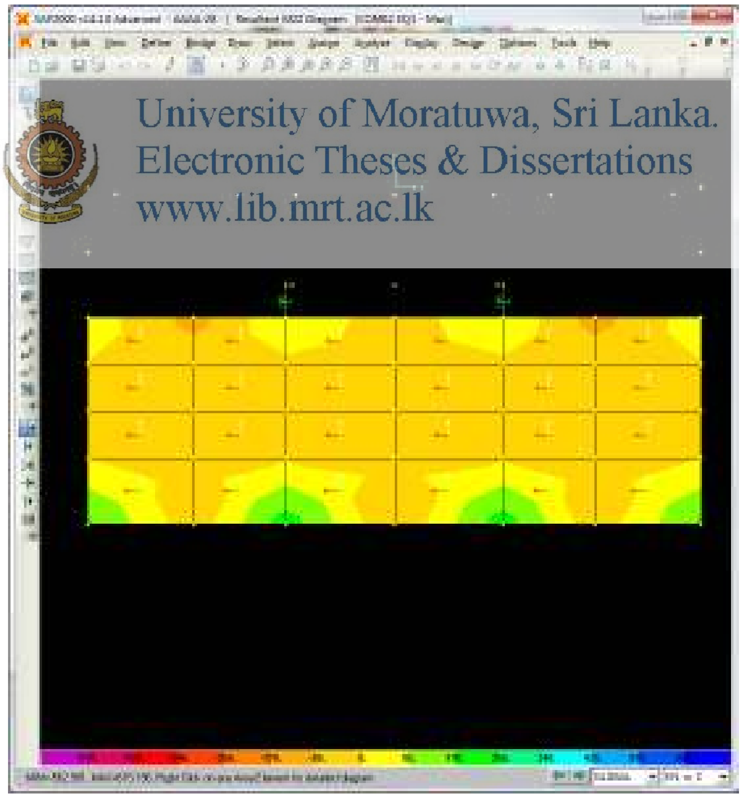


Fig Aiii-11; Bending moment distribution - Abutment A2 (Com2 EQ1)

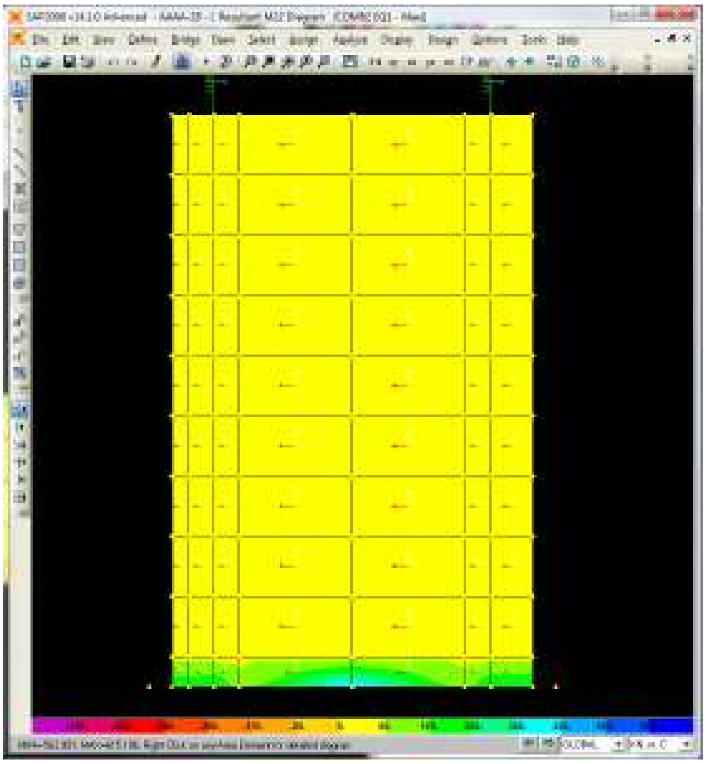


Fig Aiii-12; Bending moment distribution – Pier P1 (Com2 EQ1)

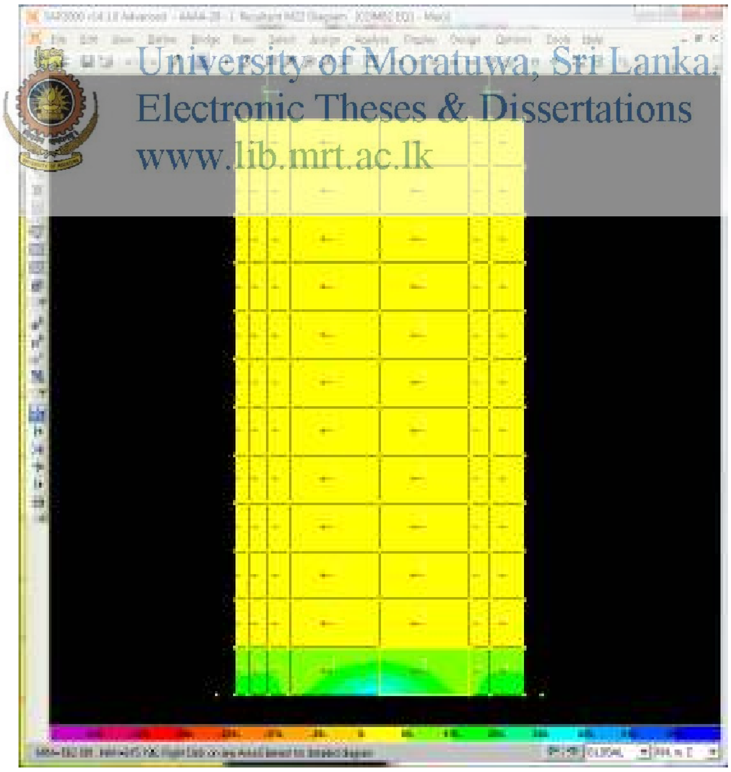


Fig Aiii-13; Bending moment distribution – Pier P2 (Com2 EQ1)

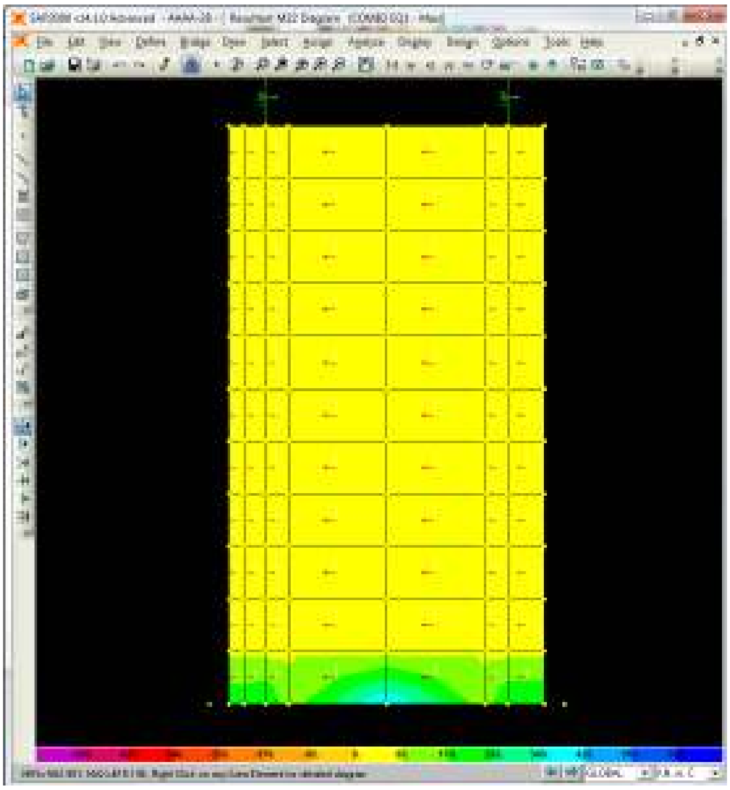


Fig Aiii-14; Bending moment distribution – Pier P3 (Com2 EQ1)

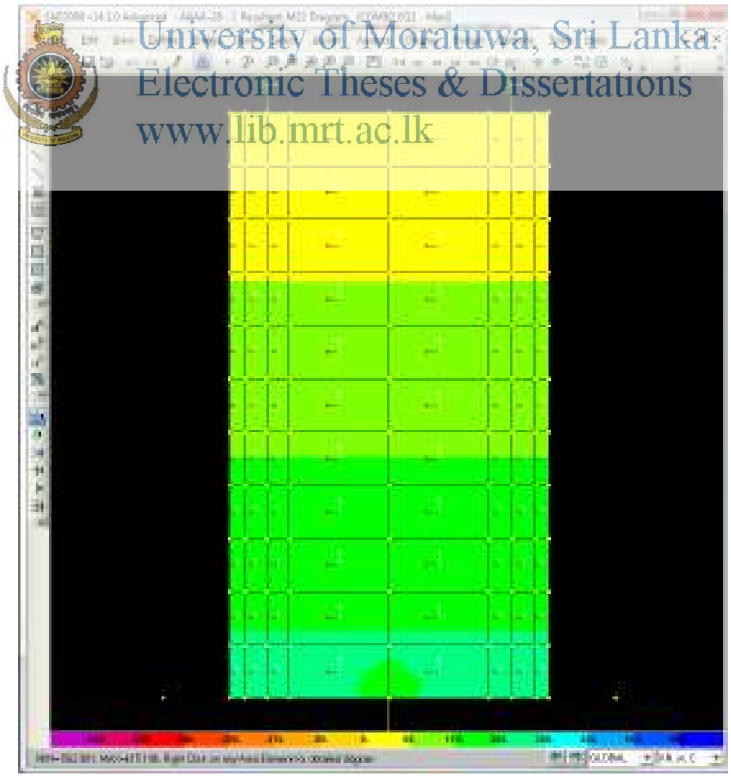


Fig Aiii-14; Bending moment distribution – Pier P4 (Com2 EQ1)

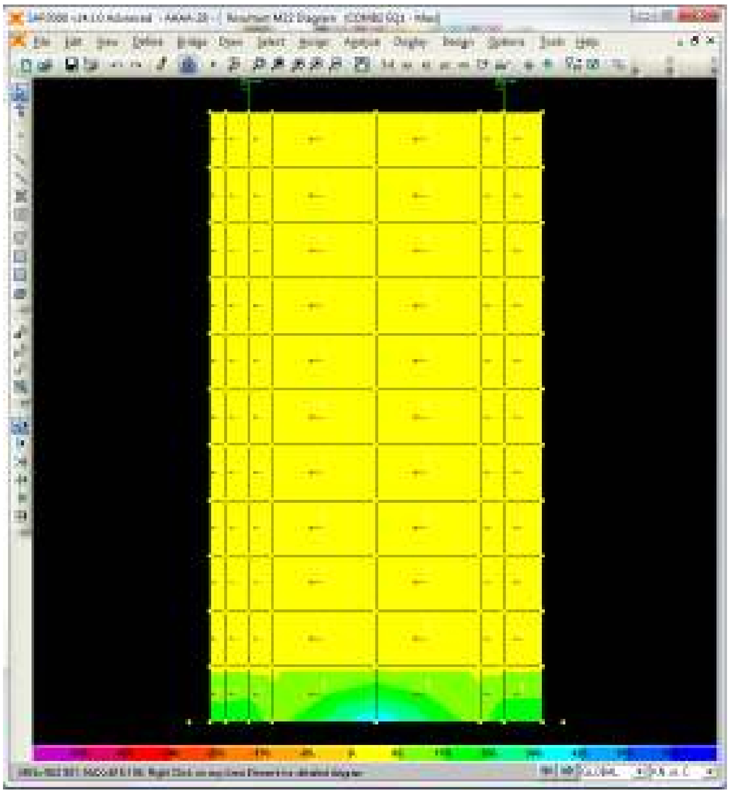


Fig Aiii-15; Bending moment distribution – Pier P5 (Com2 EQ1)

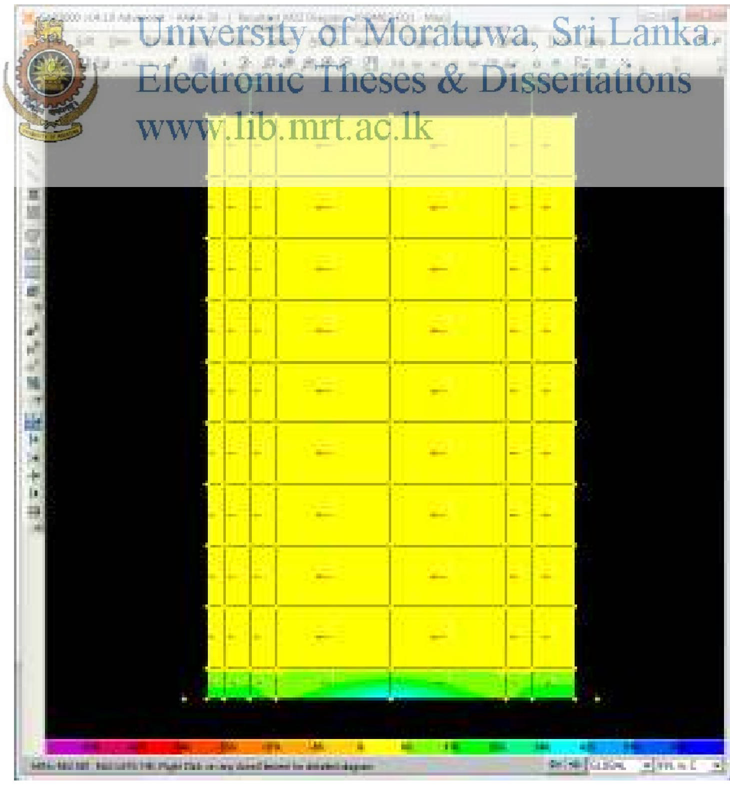


Fig Aiii-16; Bending moment distribution – Pier P6 (Com2 EQ1)

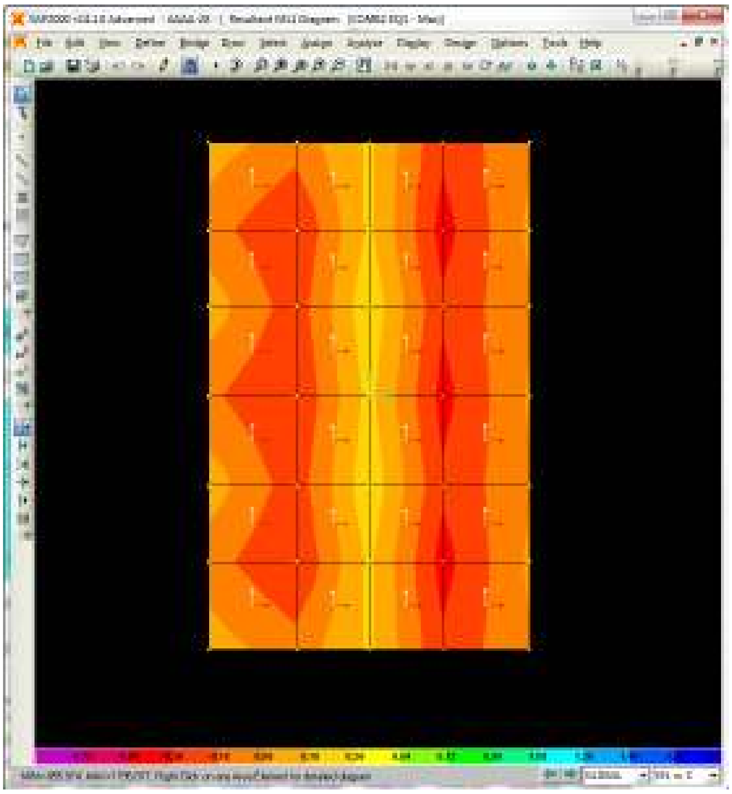


Fig Aiii-17; Bending moment distribution – Pile cap A1 (Com2 EQ1)

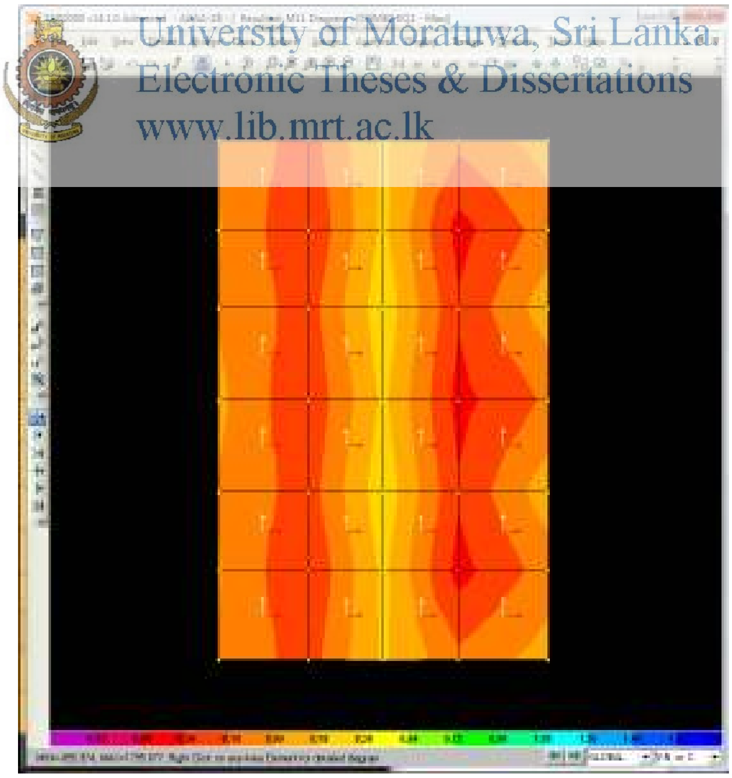


Fig Aiii-18; Bending moment distribution – Pile cap A2 (Com2 EQ1)

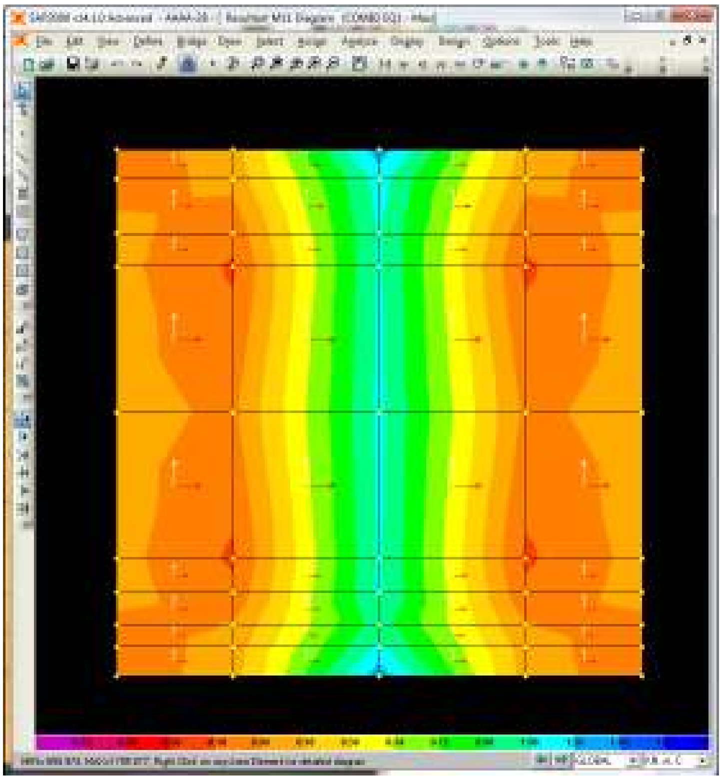


Fig Aiii-19; Bending moment distribution – Pile cap P1 (Com2 EQ1)

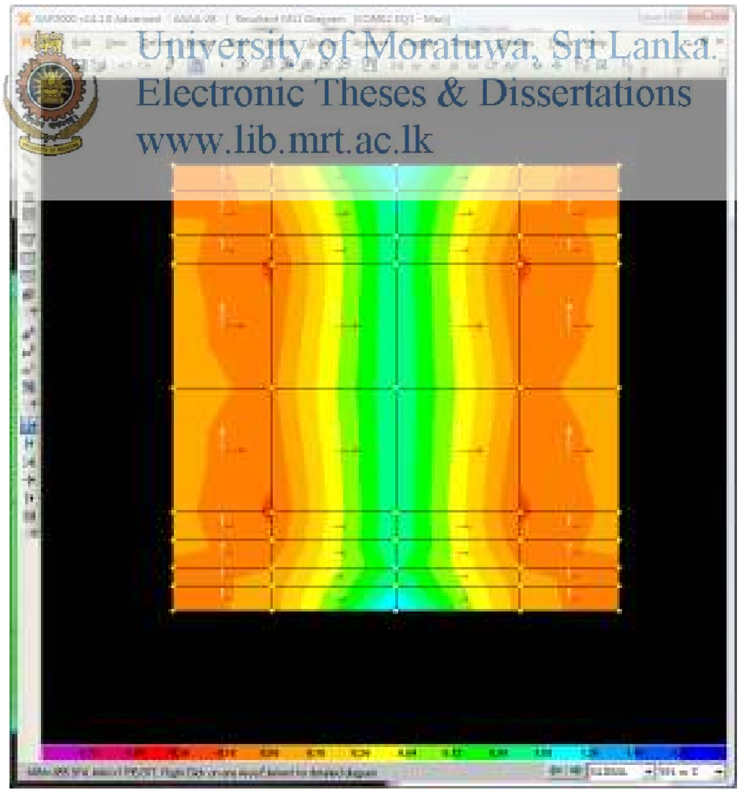


Fig Aiii-20; Bending moment distribution – Pile cap P2 (Com2 EQ1)

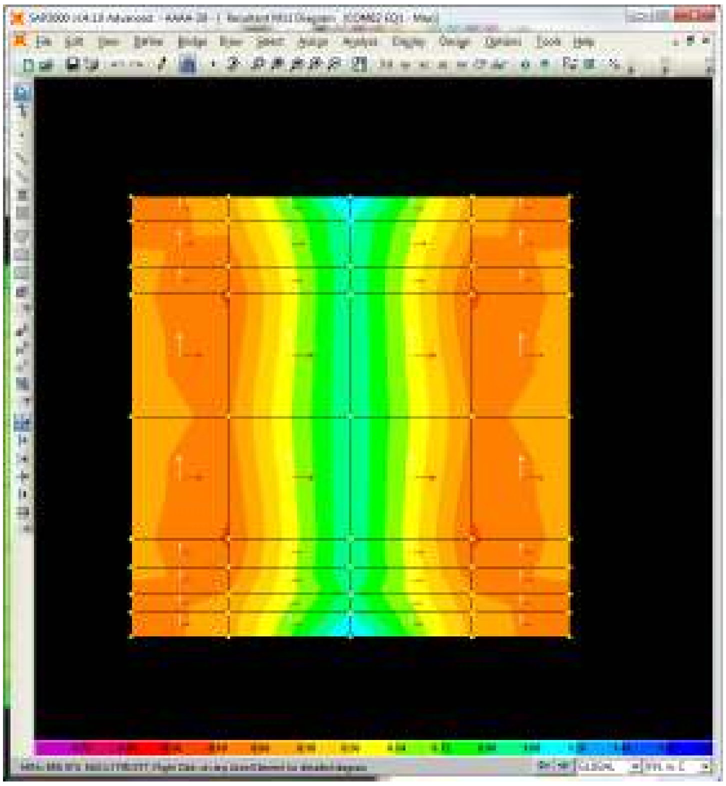


Fig Aiii-21; Bending moment distribution – Pile cap P3 (Com2 EQ1)

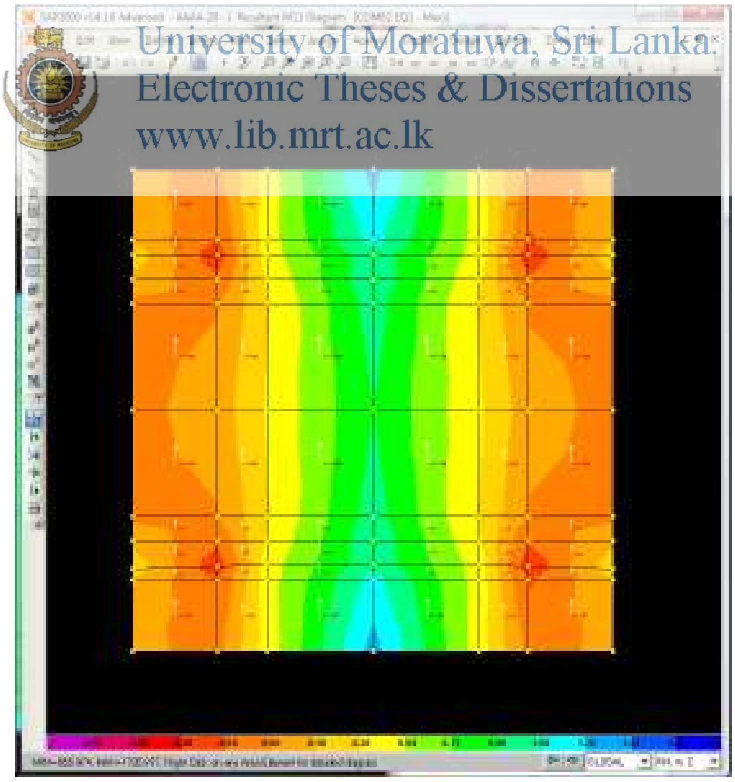


Fig Aiii-22; Bending moment distribution – Pile cap P4 (Com2 EQ1)

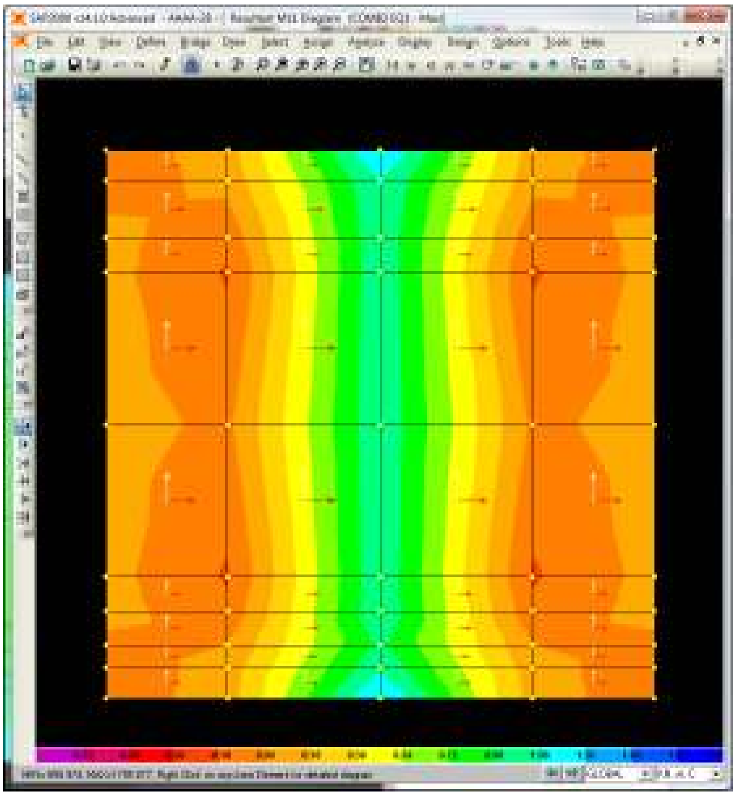


Fig Aiii-23; Bending moment distribution – Pile cap P5 (Com2 EQ1)

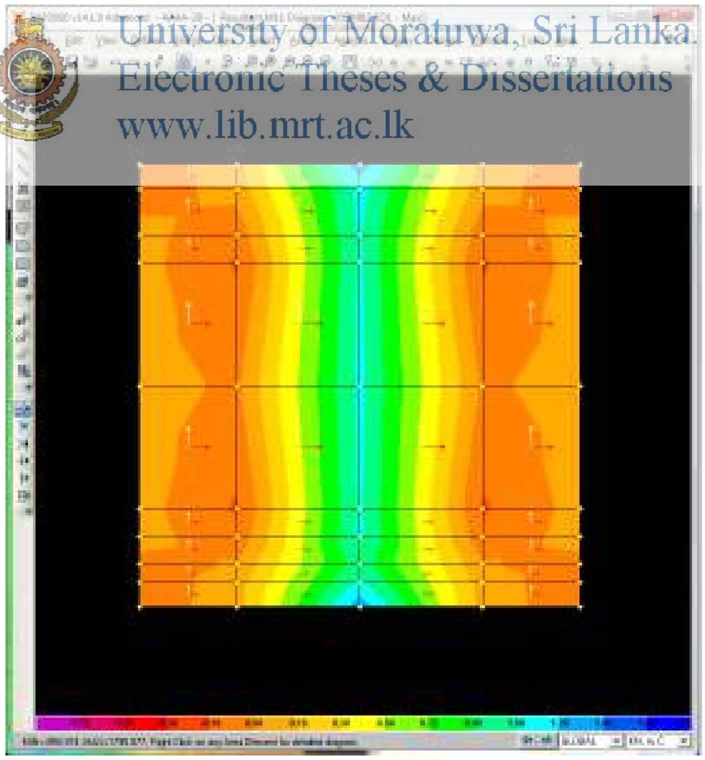


Fig Aiii-24; Bending moment distribution – Pile cap P6 (Com2 EQ1)

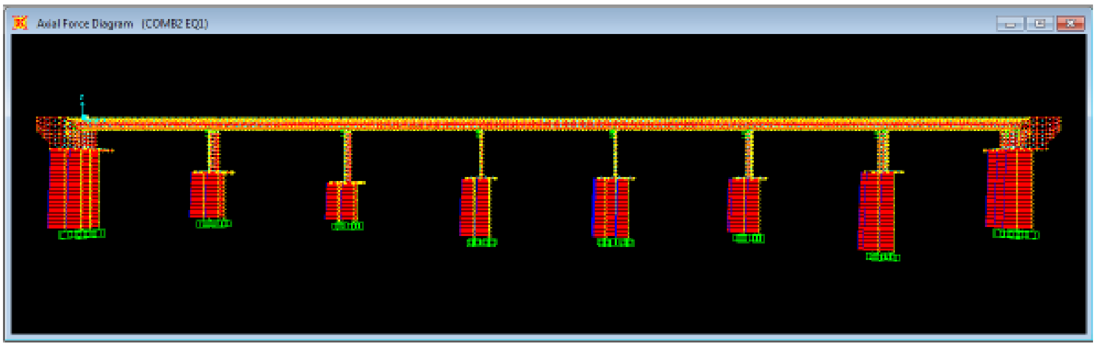


Fig Aiii-25; Axial force distribution – Piles (Com2 EQ1)

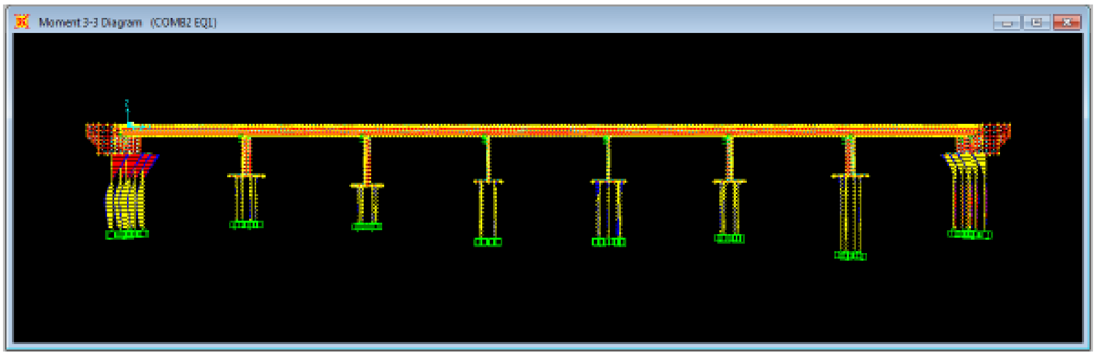


Fig Aiii-26; Bending moment distribution – Piles (Com2 EQ1)

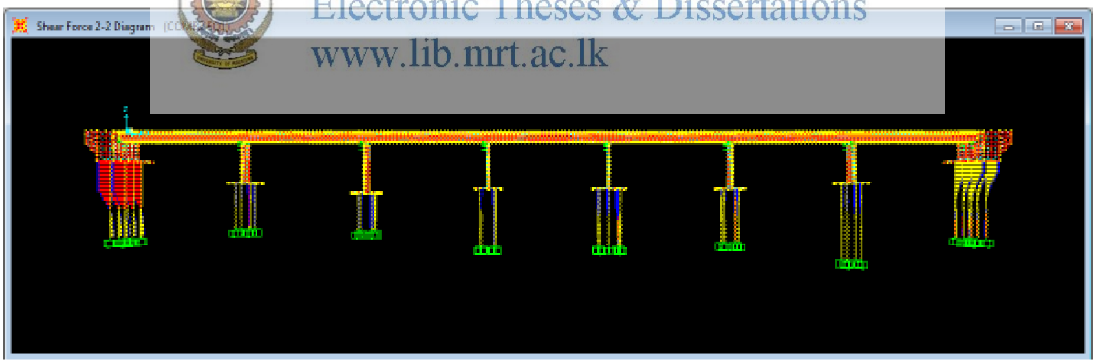


Fig Aiii-27; Shear force distribution – Piles (Com2 EQ1)

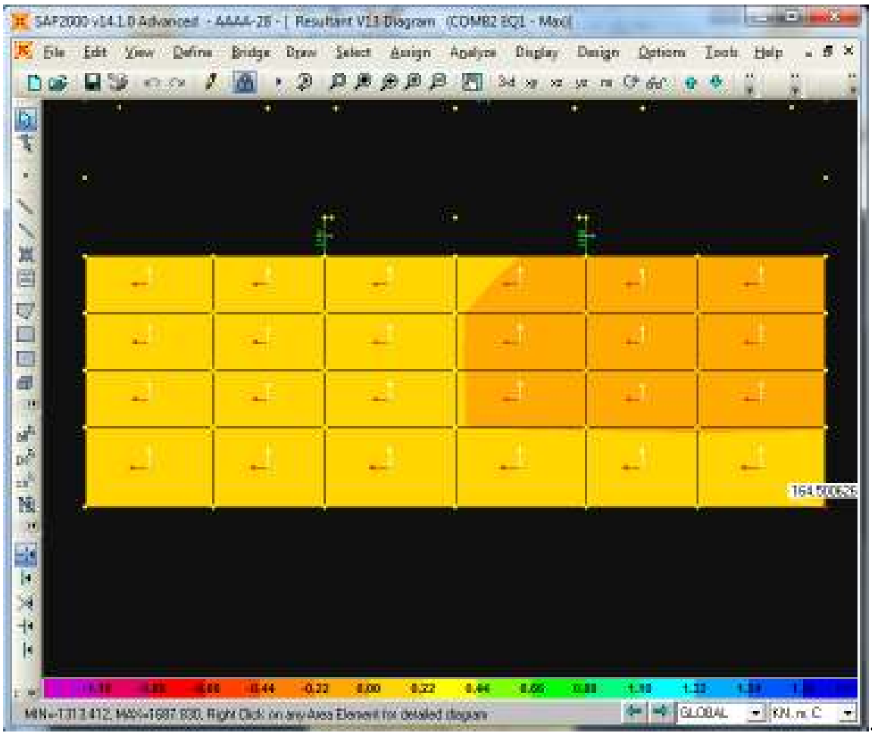


Fig Aiii-28; Shear force distribution – Abutment A1 (Com2 EQ1)

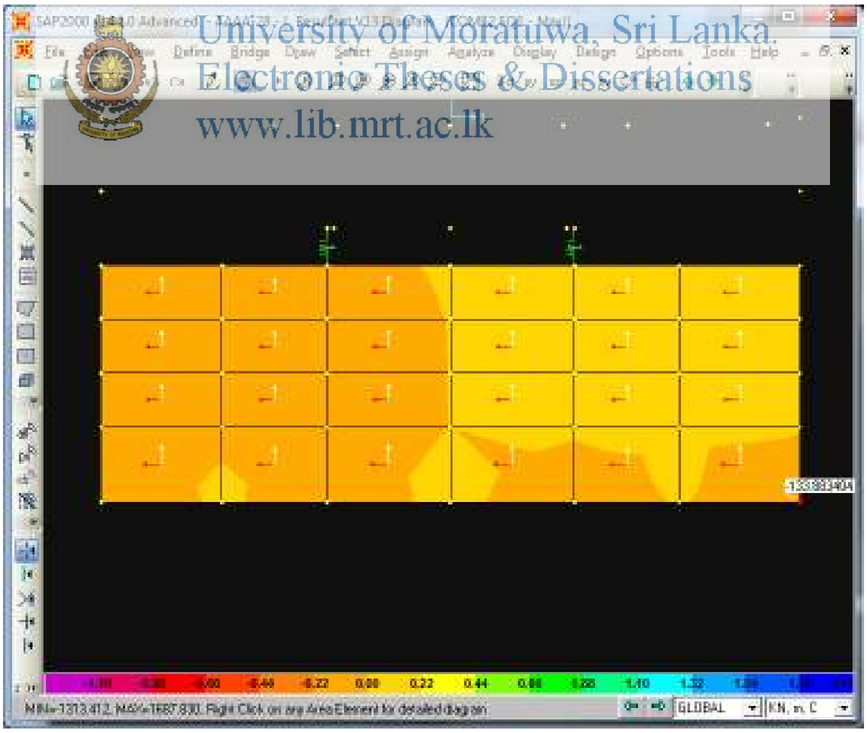


Fig Aiii-29; Shear force distribution – Abutment A2 (Com2 EQ1)

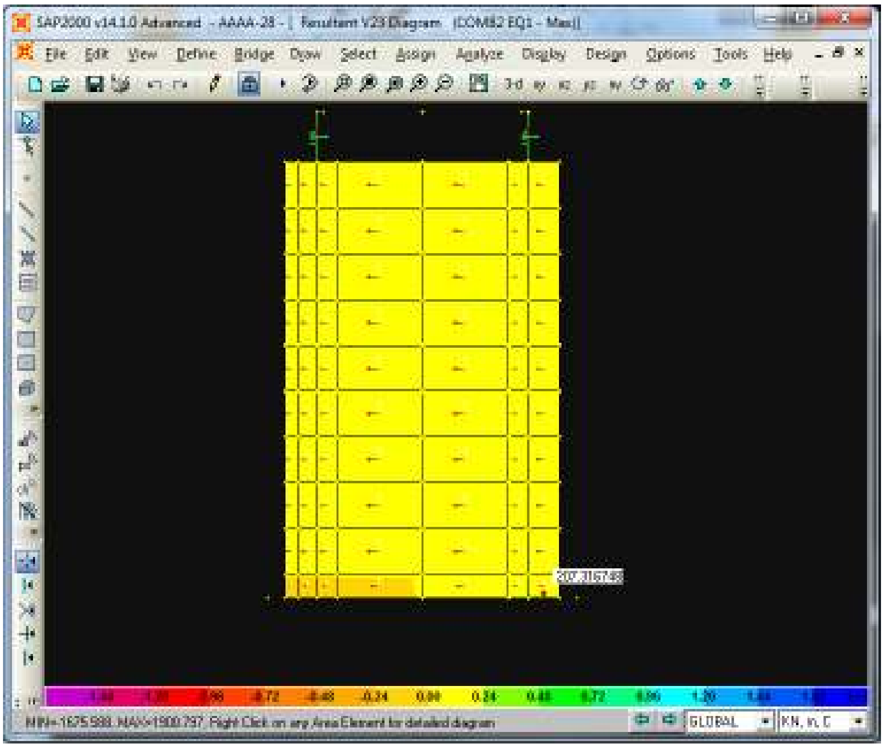


Fig Aiii-30; Shear force distribution – Pier P1 (Com2 EQ1)

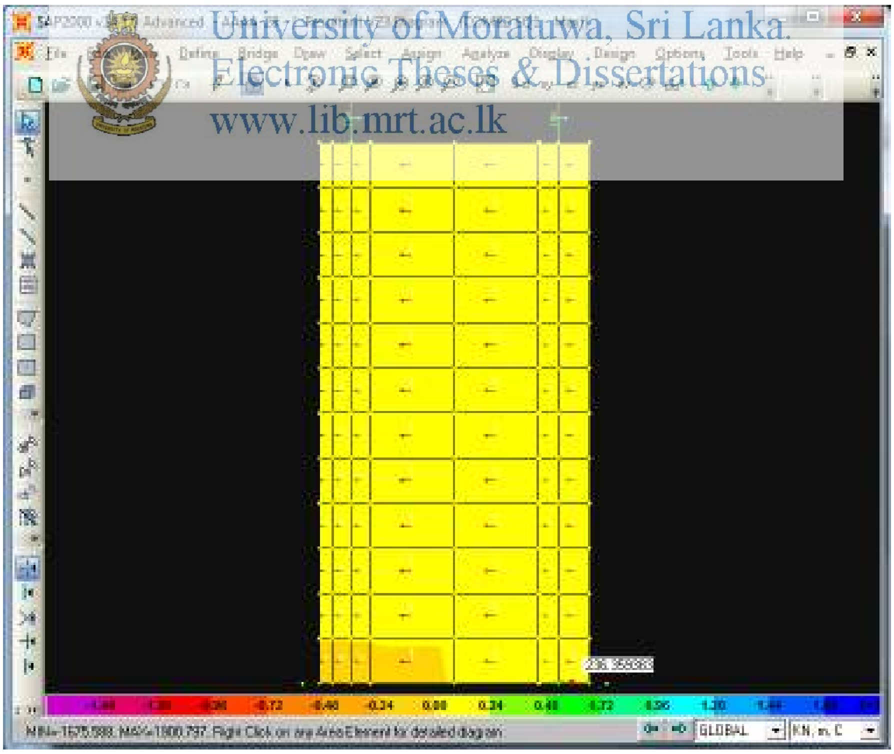


Fig Aiii-31; Shear force distribution – PierP2 (Com2 EQ1)

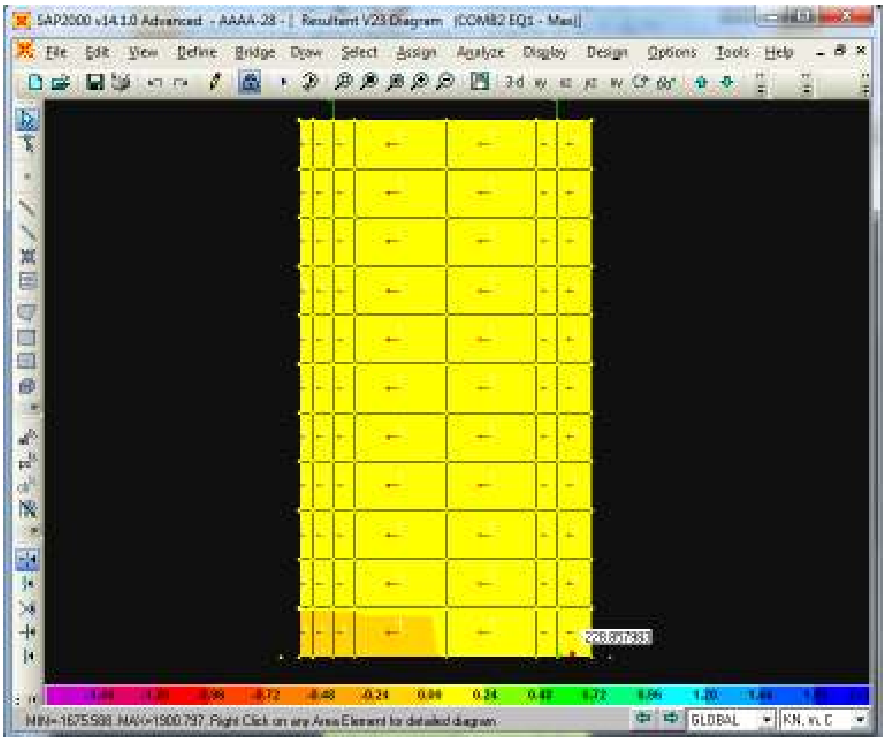


Fig Aiii-32; Shear force distribution – Pier P3 (Com2 EQ1)

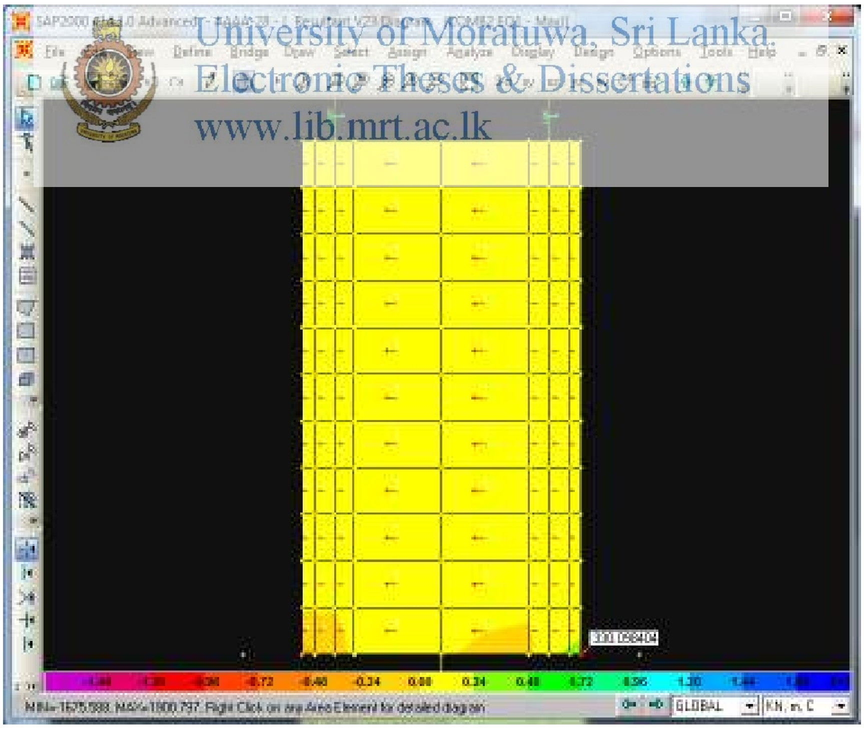


Fig Aiii-33; Shear force distribution – Pier P4 (Com2 EQ1)

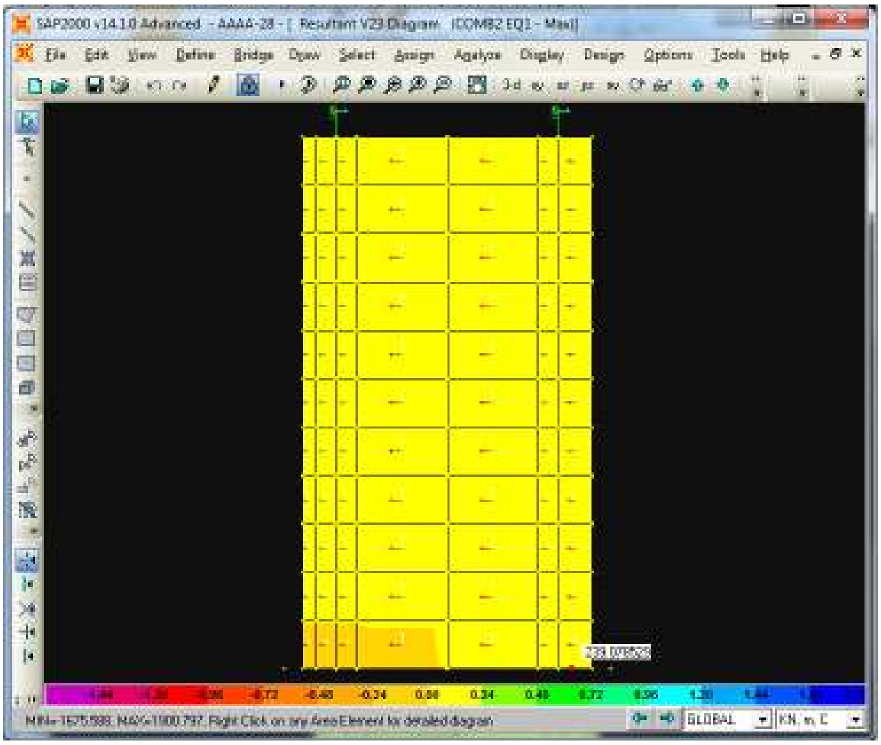


Fig Aiii-34; Shear force distribution – Pier P5 (Com2 EQ1)

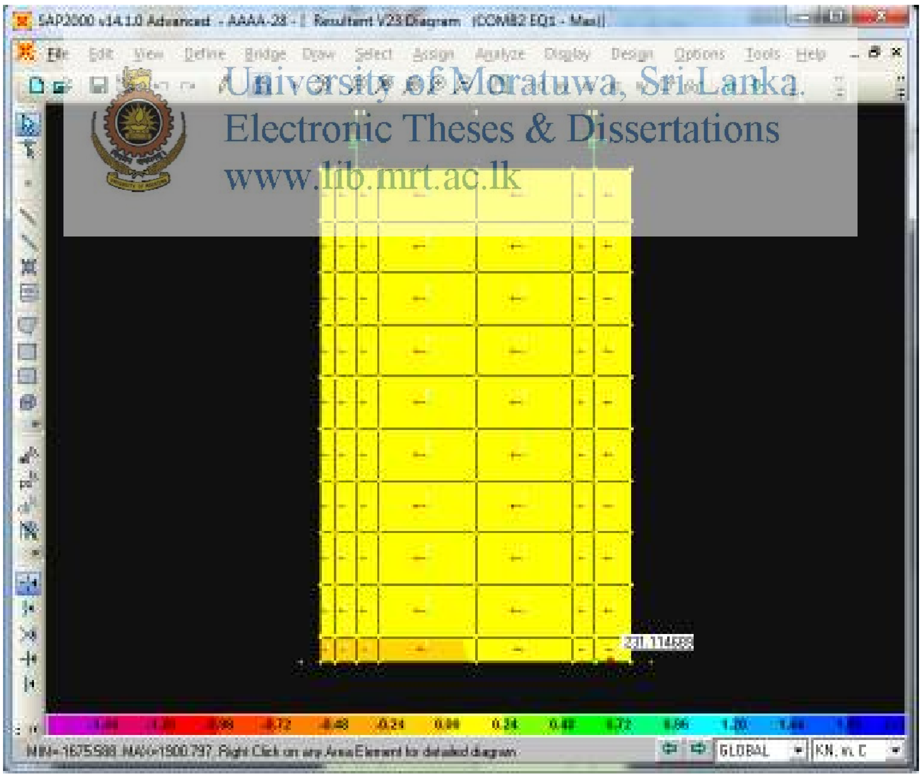


Fig Aiii-35; Shear force distribution – Pier P6 (Com2 EQ1)

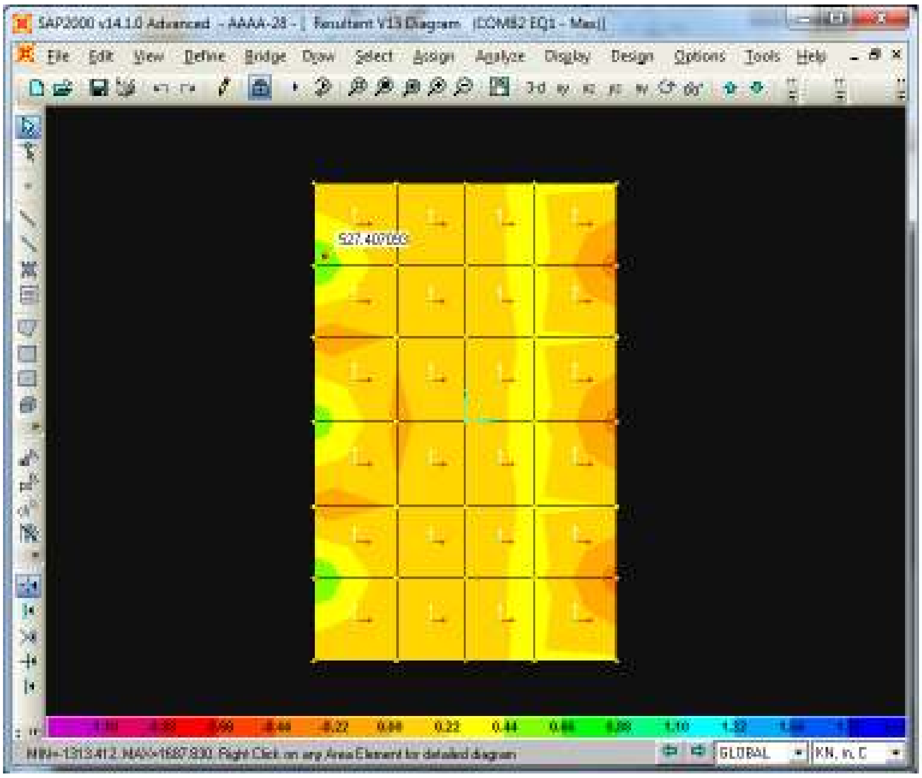


Fig Aiii-36; Shear force distribution – Pile cap A1 (Com2 EQ1)

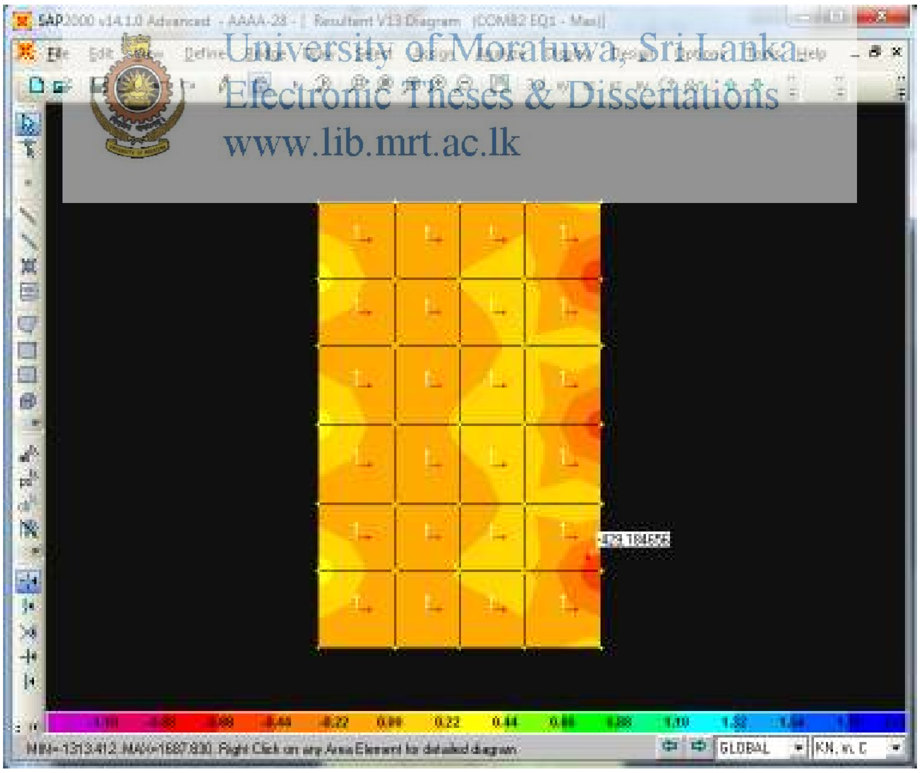


Fig Aiii-37; Shear force distribution – Pile cap A2 (Com2 EQ1)

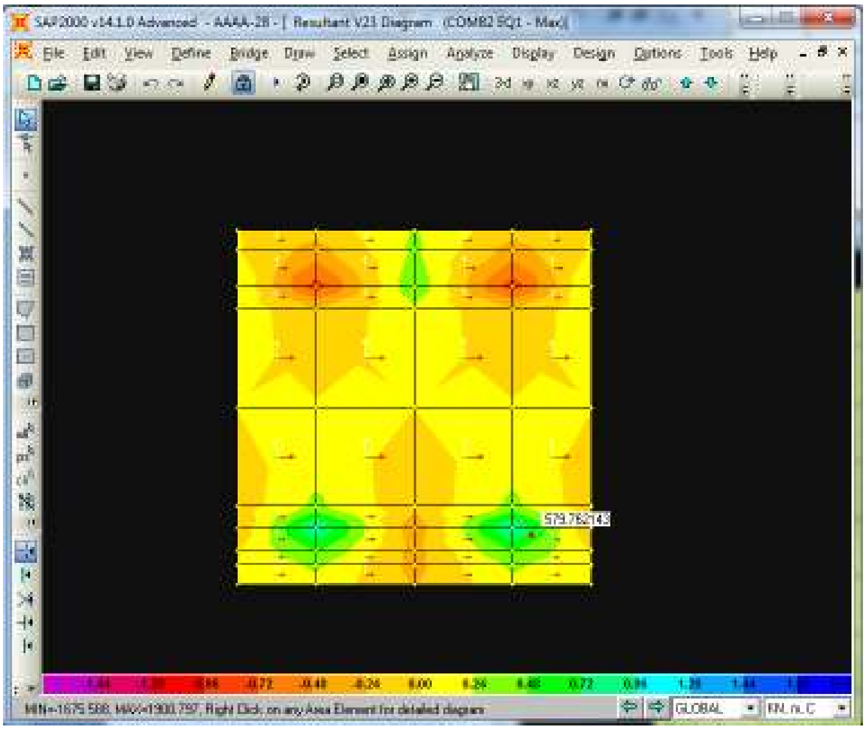


Fig Aiii-38; Shear force distribution – Pile cap P1 (Com2 EQ1)

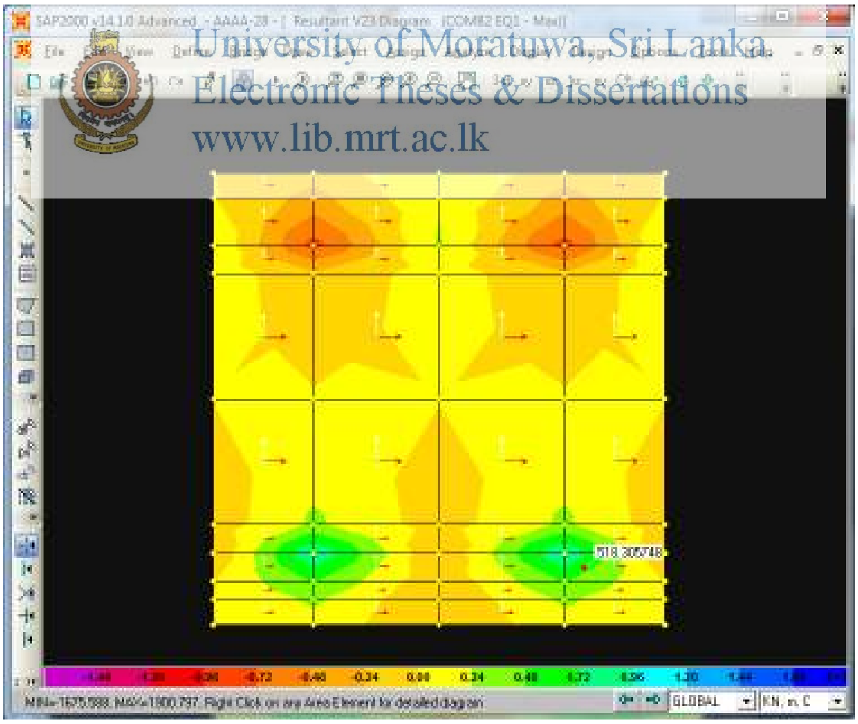


Fig Aiii-39; Shear force distribution – Pile cap P2 (Com2 EQ1)

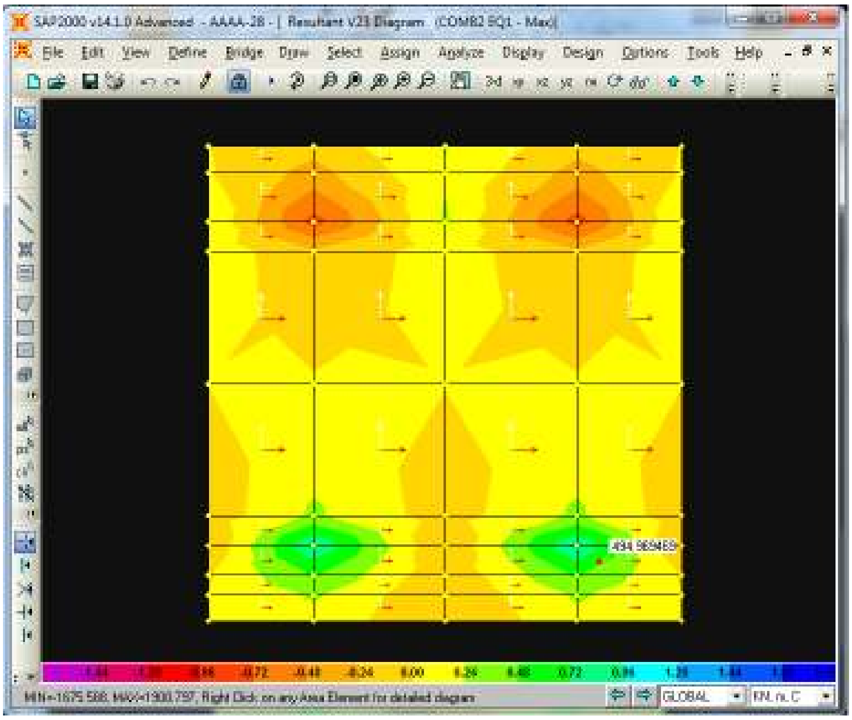


Fig Aiii-40; Shear force distribution – Pile cap P3 (Com2 EQ1)

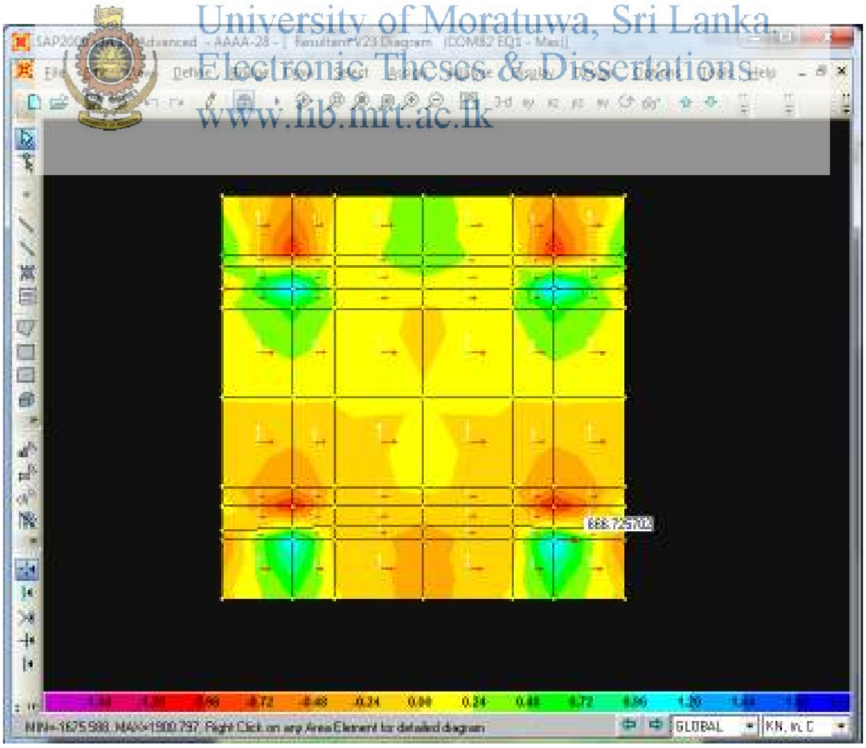


Fig Aiii-41; Shear force distribution – Pile cap P4 (Com2 EQ1)

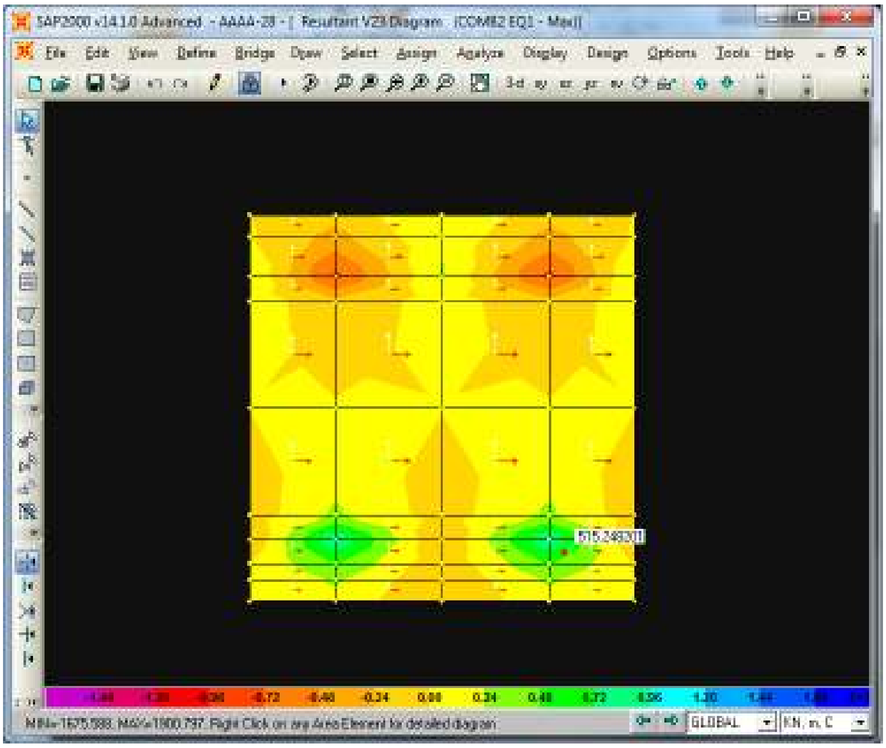


Fig Aiii-42; Shear force distribution – Pile cap P5 (Com2 EQ1)

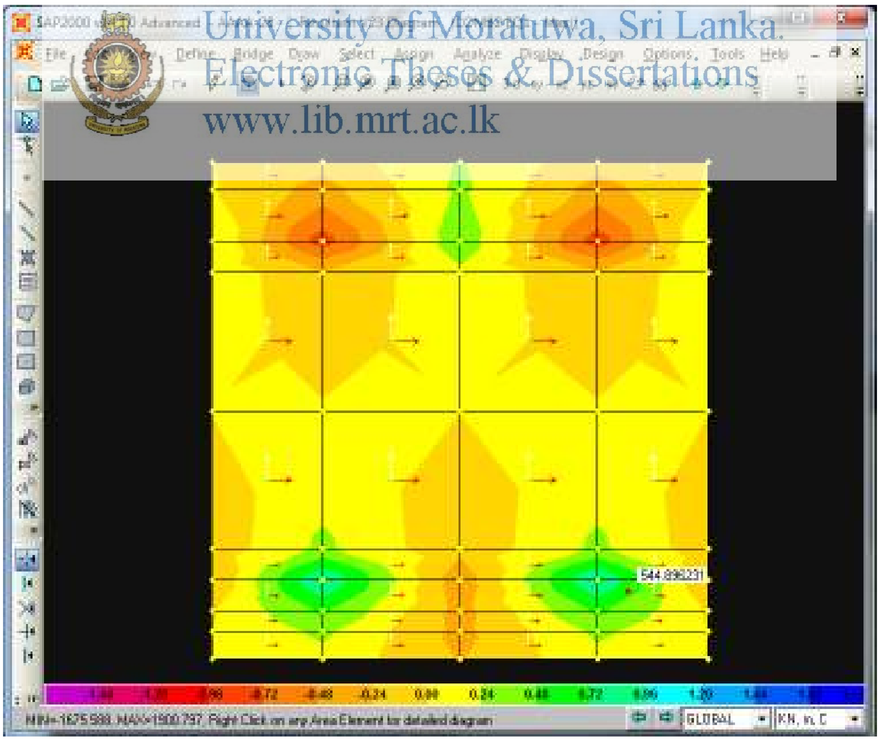


Fig Aiii-43; Shear force distribution – Pile cap P6 (Com2 EQ1)

APPENDIX – D

CAPACITY CALCULATIONS OF THE ELEMENTS

Check for Flexure for Australian Standards

for $k_u \leq 0.4$, design strength in bending = ΦM_{uo}

$$M_{uo} = 1.2\left\{ z\left(f_{cf'} + \frac{P}{A_g} \right) + Pe \right\}$$

Where,

M_{uo}	Ultimate strength in bending without axial forces
Z	Section modulus of the uncracked section
$f_{cf'}$	Characteristic flexural strength of the concrete
P	Prestressing force
A_g	Gross cross sectional area of the member
e	Eccentricity of the prestressing force
k_u	Neutral axis parameter

From 1st principles

$$k_u = \frac{0.003 A_s E_s}{0.85 f_c^{'} b d \gamma} \times (1 - k_u)$$

Where $\gamma = \left[0.85 - 0.007\left(f_c^{'} - 28\right)\right]$

Therefore $k_u = \dfrac{\left|\sqrt{(\alpha^2 + 4\alpha)}\right| - \alpha}{2}$

	Stems							
	P_1	P_2	P_3	P_4	P_5	P_6	A_1	A_2
width of the section (mm)	1000	1000	1000	1000	1000	1000	1000	1000
Depth of the section (mm)	1500	1500	1500	1500	1500	1500	2150	2150
Cover to r/f (mm)	110	110	110	110	110	110	100	100
Diameter of main r/f (mm)	32	32	32	32	32	32	25	25
Spacing of the main r/f (mm)	125	125	125	125	125	125	250	250
A_s Provided (mm^2)	6433.982	6433.98	6433.98	6433.98	6433.982	6433.98	1963.50	1963.50
$f_c^{'}$ (N/mm^2)	30	30	30	30	30	30	30	30
$f_{cf}^{'}$ (N/mm^2)	3.29	3.29	3.29	3.29	3.29	3.29	3.29	3.29
f_y (N/mm^2)	340	340	340	340	340	340	340	340

E_s (kN/mm^2)	200	200	200	200	200	200	200	200
d (mm)	1374	1374	1374	1374	1374	1374	2037.5	2037.5
γ	0.836	0.836	0.836	0.836	0.836	0.836	0.836	0.836
α	0.13	0.13	0.13	0.13	0.13	0.13	0.03	0.03
k_u	0.30	0.30	0.30	0.30	0.30	0.30	0.15	0.15
	< 0.4	< 0.4	< 0.4	< 0.4	< 0.4	< 0.4	< 0.4	< 0.4
I	1.25E+11	1.3E+11	1.3E+11	1.3E+11	1.25E+11	1.3E+11	1.8E+11	1.79E+11
d_{NA}	500	500	500	500	500	500	500	500
Z	2.5E+08	2.5E+08	2.5E+08	2.5E+08	2.5E+08	2.5E+08	3.6E+08	3.58E+08
P	0	0	0	0	0	0	0	0
e	0	0	0	0	0	0	0	0
A_g	1500000	1500000	1500000	1500000	1500000	1500000	2150000	2150000
M_{uo}	985.9006	985.901	985.901	985.901	985.9006	985.901	1413.12	1413.124
ϕ	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
ϕM_{uo}	788.7205	788.72	788.72	788.72	788.7205	788.72	1130.5	1130.499
$M_{applied}$	424.00	423.00	418.00	320.00	410.00	428.00	246.00	214.00
	Safe	Safe	Safe	Safe	Safe	Safe	Safe	Safe

Pile Caps

	P_1	P_2	P_3	P_4	P_5	P_6	A_1	A_2
width of the section (mm)	1000	1000	1000	1000	1000	1000	1000	1000
Depth of the section (mm)	2000	2000	2000	2000	2000	2000	2000	2000
Cover to r/f (mm)	100	100	100	100	100	100	100	100
Diameter of main r/f (mm)	29	29	29	32	29	29	25	25
Spacing of the main r/f (mm)	125	125	125	125	125	125	250	250
A_s Provided (mm^2)	5284.16	5284.16	5284.16	6433.98	5284.16	5284.16	1963.50	1963.50
f_c' (N/mm^2)	30	30	30	30	30	30	30	30
f_{cf}' (N/mm^2)	3.29	3.29	3.29	3.29	3.29	3.29	3.29	3.29
f_y (N/mm^2)	340	340	340	340	340	340	340	340
E_s (kN/mm^2)	200	200	200	200	200	200	200	200
d (mm)	1885.5	1885.5	1885.5	1884	1885.5	1885.5	1887.5	1887.5
γ	0.836	0.836	0.836	0.836	0.836	0.836	0.836	0.836
α	0.08	0.08	0.08	0.10	0.08	0.08	0.03	0.03
k_u	0.24	0.24	0.24	0.27	0.24	0.24	0.16	0.16
	< 0.4	< 0.4	< 0.4	< 0.4	< 0.4	< 0.4	< 0.4	< 0.4
I	1.67E+11	1.7E+11	1.7E+11	1.7E+11	1.67E+11	1.7E+11	1.7E+11	1.67E+11
d_{NA}	500	500	500	500	500	500	500	500
Z	3.33E+08	3.33E+08	3.33E+08	3.33E+08	3.33E+08	3.33E+08	3.33E+08	3.33E+08
P	0	0	0	0	0	0	0	0
e	0	0	0	0	0	0	0	0
A_g	2000000	2000000	2000000	2000000	2000000	2000000	2000000	2000000
M_{uo}	1314.534	1314.53	1314.53	1314.53	1314.534	1314.53	1314.53	1314.534
ϕ	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
ϕM_{uo}	1051.627	1051.627	1051.627	1051.627	1051.627	1051.627	1051.627	1051.627
$M_{applied}$	1021.00	972.00	963.00	1025.00	944.00	998.00	427.00	429.00
	Safe	Safe	Safe	Safe	Safe	Safe	Safe	Safe

Check for Shear for Australian Standards

Design shear strength = ϕV_u

$V_u = V_{uc} + V_{us}$

Where,

V_{uc} Shear strength excluding shear r/f

V_{us} Shear strength contributed by shear r/f

$$V_{uc} = \beta_1 \beta_2 \beta_3 b_v d_0 \left[\frac{A_{st} f'_c}{b_v d_0} \right]^{\frac{1}{3}}$$

Where,

$$\beta_1 = 1.1 \left[1.6 - \frac{d_0}{1000} \right] \geq 1.1$$

$$\beta_2 = 1.0$$

or

$$1 - \left[\frac{N^*}{3.5 A_g} \right] \geq 0$$

for members subjected to axial tension

$$1 + \left[\frac{N^*}{14 A_g} \right] \geq 0$$

for members subjected to axial compression

$\beta_3 =$ 1.0 or

$2d_0/a_v$ but not greater than 2

$$V_{us} = \frac{A_{sv} f_{sy,f} d_0}{s} Cot\theta_v$$

In abutments & Piers it was not used shear reinforcements. Therefore V_{us} will be zero

	Stems							
	P_1	P_2	P_3	P_4	P_5	P_6	A_1	A_2
Applied Shear Force V (kN)	207	236	229	330	239	231	164	133
d_0 (mm)	1,374	1,374	1,374	1,374	1,374	1,374	2,038	2,038
β_1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
A_g (mm^2)	1,500,000	1,500,000	1,500,000	1,500,000	1,500,000	1,500,000	2,150,000	2,150,000
N^*								
β_2	1	1	1	1	1	1	1	1
β_3	2	2	2	2	2	2	2	2
b_v (mm)	1,000	1,000	1,000	1,000	1,000	1,000	1,000	1,000
f'_c (N/mm^2)	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0
A_{st} (mm^2)	6,434	6,434	6,434	6,434	6,434	6,434	1,963	1,963
V_{uc} (kN)	1,571	1,571	1,571	1,571	1,571	1,571	1,376	1,376

V_{us} (kN)	-	-	-	-	-	-	-	-
V_u (kN)	1,571	1,571	1,571	1,571	1,571	1,571	1,376	1,376
ϕ	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7
ϕV_u(kN)	1,100	1,100	1,100	1,100	1,100	1,100	963	963
	Satisfy	Satisfy	Satisfy	Satisfy	Satisfy	Satisfy	Satisfy	Satisfy

	Pile caps							
	P_1	P_2	P_3	P_4	P_5	P_6	A_1	A_2
Applied Shear Force V (kN)	580	518	495	667	515	545	527	429
d_0 (mm)	1,886	1,886	1,886	1,884	1,886	1,886	1,888	1,888
β_1	1	1	1	1	1	1	1	1
A_g (mm^2)	2,000,000	2,000,000	2,000,000	2,000,000	2,000,000	2,000,000	2,000,000	2,000,000
N^*								
β_2	1	1	1	1	1	1	1	1
β_3	2	2	2	2	2	2	2	2
b_v (mm)	1,000	1,000	1,000	1,000	1,000	1,000	1,000	1,000
f'_c(N/mm^2)	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0
A_{st} (mm^2)	5,284	5,284	5,284	6,434	5,284	5,284	1,963	1,963
V_{uc} (kN)	1,652	1,652	1,652	1,763	1,652	1,652	1,189	1,189
V_{us} (kN)	-	-	-	-	-	-	-	-
V_u (kN)	1,652	1,652	1,652	1,763	1,652	1,652	1,189	1,189
ϕ	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7
ϕV_u(kN)	1,156	1,156	1,156	1,234	1,156	1,156	832	832
	Satisfy	Satisfy	Satisfy	Satisfy	Satisfy	Satisfy	Satisfy	Satisfy

Check for Flexure for British Standards

Ultimate Bending Capacity

$$M_u = 0.87 f_y A_s Z$$

Where,

$$Z = \left(1 - \frac{1.1 f_y A_s}{f_{cu} b d}\right) d$$

or

Z = 0.95d

Z will be selected the minimum of above

$$K = \frac{M_u}{f_{cu} b d^2}$$

Where,

M_u	Ultimate resistance moment
f_y	Yield strength of reinforcement
A_s	Area of tension reinforcement
Z	liver arm
f_{cu}	characteristic strength of concrete
b	width of the section
d	Effective depth to tension reinforcement

	Pier shaft						Abutment	
	P_1	P_2	P_3	P_4	P_5	P_6	A_1	A_2
Ultimate Bending Moment (kNm/m)	424.00	423.00	418.00	320.00	410.00	428.0 0	246.00	214.00
Width of the section (mm)	1000	1000	1000	1000	1000	1000	1000	1000
Depth of the section (mm)	1500	1500	1500	1500	1500	1500	2150	2150
Diameter of main r/f (mm)	32	32	32	32	32	32	25	25
cover to r/f (mm)	110	110	110	110	110	110	100	100
Strength of concrete (fcu) (N/mm^2)	30	30	30	30	30	30	30	30
Strength of main r/f (fy) (N/mm2)	340	340	340	340	340	340	340	340
Effective depth -d (mm)	1374	1374	1374	1374	1374	1374	2037.5	2037.5
$K = \frac{M_u}{f_{cu} b d^2}$	0.007	0.007	0.007	0.006	0.007	0.008	0.002	0.002
	No compression r/f required	No compression r/f required	No compression r/f required	No compression r/f required	No compression r/f required	No compression r/f required	No compression r/f required	No compression r/f required
Z-method I	1360.87	1360.90	1361.06	1364.11	1361.31	1360.74	2032.40	2033.06
0.95d	1305.30	1305.30	1305.30	1305.30	1305.30	1305.30	1935.63	1935.63
Z	1305.30	1305.30	1305.30	1305.30	1305.30	1305.30	1935.63	1935.63
mm^2	1098.14	1095.55						
Main r/f T	**32**	**32**	**32**	**32**	**32**	**32**	**25**	**25**
spacing @	**125**	**125**	**125**	**125**	**125**	**125**	**250**	**250**
A_s provided	**6434**	**6434**	**6434**	**6434**	**6434**	**6434**	**1963**	**1963**
Moment Capasity	**2484.21**	**2484.21**	**2484.21**	**2484.21**	**2484.21**	**2484.21**	**1124.21**	**1124.21**
Applied Moment	**424.00**	**423.00**	**418.00**	**320.00**	**410.00**	**428.00**	**246.00**	**214.00**
	OK	OK	OK	OK	OK	OK	OK	OK

Pile cap

	P_1	P_2	P_3	P_4	P_5	P_6	A_1	A_2
Ultimate Bending Moment (kNm/m)	1021.00	972.00	963.00	1025.00	944.00	998.00	427.00	429.00
Width of the section (mm)	1000	1000	1000	1000	1000	1000	1000	1000
Depth of the section (mm)	2000	2000	2000	2000	2000	2000	2000	2000
Diameter of main r/f (mm)	29	29	29	32	29	29	25	25
cover to r/f (mm)	100	100	100	100	100	100	100	100
Strength of concrete (fcu) (N/mm^2)	30	30	30	30	30	30	30	30
Strength of main r/f (fy) (N/mm2)	340	340	340	340	340	340	340	340
Effective depth -d (mm)	1885.5	1885.5	1885.5	1884	1885.5	1885.5	1887.5	1887.5
$K = \frac{M_u}{f_{cu} b d^2}$	0.010	0.009	0.009	0.010	0.009	0.009	0.004	0.004
	No compression r/f required	No compression r/f required	No compression r/f required	No compression r/f required	No compression r/f required	No compression r/f required	No compression r/f required	No compression r/f required
Z-method I	1862.39	1863.52	1863.72	1860.78	1864.16	1862.92	1877.92	1877.87
0.95d	1791.23	1791.23	1791.23	1789.80	1791.23	1791.23	1793.13	1793.13
Z	1791.23	1791.23	1791.23	1789.80	1791.23	1791.23	1793.13	1793.13
mm^2	1926.98	1834.50						
Main r/f T	**29**	**29**	**29**	**32**	**29**	**29**	**25**	**25**
spacing @	**125**	**125**	**125**	**125**	**125**	**125**	**250**	**250**
A_s provided	**5284**	**5284**	**5284**	**6434**	**5284**	**5284**	**1963**	**1963**
Moment Capasity	**2799.78**	**2799.78**	**2799.78**	**3406.30**	**2799.78**	**2799.78**	**1041.45**	**1041.45**
Applied Moment	**1021.00**	**972.00**	**963.00**	**1025.00**	**944.00**	**998.00**	**427.00**	**429.00**
	OK	OK	OK	OK	OK	OK	OK	OK

Check for shear for British Standards

	Pier shaft						Abutment	
	P_1	P_2	P_3	P_4	P_5	P_6	A_1	A_2
Shear force - V (kN)	207.32	236.35	228.85	330.10	239.07	231.11	164.00	133.00
Shear stress - v=V/bd (N/mm^2)	0.15	0.17	0.17	0.24	0.17	0.17	0.08	0.07
Shear capasity of concrete = $0.75(f_{cu})^{0.5}$ (N/mm^2)	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11
	satisfy	satisfy	satisfy	satisfy	satisfy	satisfy	satisfy	satisfy
$v_c = \frac{0.27}{\gamma_m}\left[\frac{100A_s}{bd}\right]^{1/3} f_{cu}^{1/3}$	0.52	0.52	0.52	0.52	0.52	0.52	0.31	0.31
$\xi_s = (500/d)^{1/4}$	0.78	0.78	0.78	0.78	0.78	0.78	0.70	0.70
Shear capacity $=\xi_s v_c (N/mm^2)$	0.40	0.40	0.40	0.40	0.40	0.40	0.22	0.22
	Satisfy	Satisfy	Satisfy	Satisfy	Satisfy	Satisfy	Satisfy	Satisfy

	Pile cap							
	P_1	P_2	P_3	P_4	P_5	P_6	A_1	A_2
Shear force - V (kN)	579.76	518.31	494.97	666.73	515.25	544.90	527.41	429.18
Shear stress - v=V/bd (N/mm^2)	0.31	0.27	0.26	0.35	0.27	0.29	0.28	0.23
Shear capasity of concrete = $0.75(f_{cu})^{0.5}$ (N/mm^2)	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11
	satisfy	satisfy	satisfy	satisfy	satisfy	satisfy	satisfy	satisfy
$v_c = \frac{0.27}{\gamma_m}\left[\frac{100A_s}{bd}\right]^{1/3} f_{cu}^{1/3}$	0.44	0.44	0.44	0.47	0.44	0.44	0.32	0.32
$\xi_s = (500/d)^{1/4}$	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72
Shear capacity $=\xi_s v_c (N/mm^2)$	0.32	0.32	0.32	0.34	0.32	0.32	0.23	0.23
	Satisfy	Satisfy	Satisfy	need shear r/f	Satisfy	Satisfy	need shear r/f	need shear r/f

calculations	Output

	Pier P_1,P_2,P_3,P_5,P_6	Abutment A_1,A_2
Bearing Length L(mm)	1000	560
Bearing width W (mm)	710	560
Bearing thickness H (mm)	104	122
Total elastomer thickness H_r(mm)	96	112
Thickness of one elastomer layer Hri (mm)	16	16
Thickness of one steel layer H_s (mm)	1	1
Gross plan area A (mm^2)	710000	313600
Elastomer Second moment of inertia I (mm^4)	64502257.5	80207727
Shape factor S		
Shear Modulus (G) (N/mm^2)	0.9	0.9
Bulk modulus E_c (N/mm^2)	604.22	357.52

Calculation stiffness to input the FEM

Lateral Stiffness K_H	$\boxed{K_H = \frac{GA}{H_r}}$	6656.25	2520	kN/m
Vertical Stiffness K_v	$\boxed{K_v = \frac{E_c A}{H}}$	26812262.5	7007392	kN/m
Rotational Stiffness K_θ	$\boxed{K_\theta = \frac{EI}{H_r}}$	405.97	256.03	kN/m

Design check for bearing pads

Check for maximum shear strain

$$\varepsilon_{sc} + \varepsilon_{sR} + \varepsilon_{sh} \langle \frac{2.6}{\sqrt{G}}$$

ε_{sc} shear strain at edge of bonded surface due to loads normal to bearing surface $= 6S\varepsilon_c$

ε_{sr} shear strain at edge of bonded surface due to relative rotation of bearing surface to bearing surface

ε_{sh} shear strain at edge of bonded surface due to force tangential to the surface or movement of the structure or both

$$\varepsilon_c = \frac{N}{3A_{eff}G(1+2S^2)}$$

Where,

$$A_{eff} = A_b\left[1 - \frac{\delta_a}{a} - \frac{\delta_b}{b}\right]$$

N -	Compressive load on a bearing at serviceability limit state
δ_a-	maximum shear displacement tangential to the bearing surface in the direction of dimension "a" due to movement of the structure and tangential forces
a -	plan dimension of the edge of the bonded surface of rectangular bearings parallel to the span of the bridge
δ_b-	maximum shear displacement tangential to the bearing surface in the direction of dimension "b" due to movement of the structure and tangential forces
b -	plan dimension of the edge of the bonded surface of rectangular bearings transverse to the span of the bridge

$$S = \frac{A_b}{Pt_e}$$

A_b -	bonded surface area
P -	Surface perimeter
t_e -	effective thickness of the individual elastomer layer in compression(due to vertical load or rotation)

$$\varepsilon_{sr} = \frac{\alpha_a a^2 + \alpha_b b^2}{2t_i t}$$

α_a -	angle of rotation parallel to the span of the bridge
α_b -	angle of rotation transverse to the span of the bridge

$$\varepsilon_{sh} = \frac{\delta_s}{t}$$

δ_s-	maximum resultant vector shear displacement tanngential to the bearing surface in the direction of "a" and "b"

Check for compressive stress

Mean compressive stress (N/A_b) < 15Mpa

Check for rotational limitation

$$d_c \geq \frac{\alpha_a a + \alpha_b b}{3}$$

where,

$$d_c = \sum (t_n \varepsilon_c)$$

t_n - layer thickness of elastomer

$\varepsilon_c = \frac{N}{EA_b}$ compressive strain of a layer

$$E = E_h + \left[\frac{C_1 G S^2}{1 + \left(\frac{C_1 G S^2}{0.75B} \right)} \right]$$

$$E_h = 4G\left[1 + \left(\frac{q}{1+q^2}\right)^2\right]$$

$$C_1 = 4 + q(6 - 3.3q)$$

q = a/b or b/a whichever is the lesser

check for stability

$$N \leq \frac{2 b_e G S A_{eff}}{3t}$$

where,

b_e - lesser of a and b

	Pier P_1,P_2,P_3,P_5,P_6	Abutment A_1,A_2
Size of the bearing	710 X 1000	560 X 560
thickness of the bearing (mm)	104	122
Inner layer thickness (mm)	16	16
No of inner layers	4	5
Steel layer thickness (mm)	1	1
Outer layer thickness (mm)	16	16
Hardness (IRHD)	60	60
Shear Modulus (G) (N/mm^2)	0.9	0.9
Bulk Modulus (B) (N/mm^2)	2000	2000
N (kN)	3,619	1,579
A_b (mm^2)	710,000	313,600
P (mm)	3420	2240
t_e (mm)	16	16
S	12.975	8.750
a (mm)	1000	560
δ_a (mm)	4.2	13.3
b (mm)	710	560
δ_b(mm)	0	0
δ_s(mm)	4.200	13.300
A_{eff} (mm^2)	707018	306152
ε_c	0.0056	0.0124
ε_{sc}	0.437	0.651
α_a (rad)	0.0025	0.0025
α_b (rad)	0	0
ε_{sr}	0.7512	0.2008
ε_{sh}	0.0404	0.1090

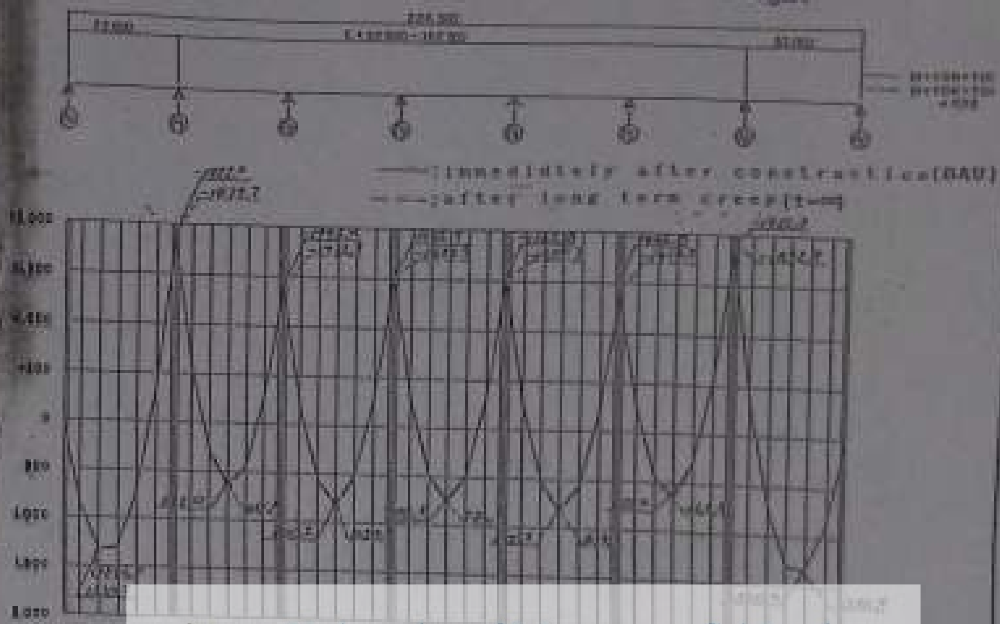
$\varepsilon_{sc}+\varepsilon_{sr}+\varepsilon_{sh}$	1.2286	0.9605
2.6/G	2.9	2.9
	shear strain OK	**shear strain OK**
N/A_b (Mpa)	5.097	5.035
	Compressive stress within the limit	**Compressive stress within the limit**
q	0.710	1.000
C_1	6.596	6.700
E_h (N/mm^2)	4.402	4.500
E (N/mm^2)	604.22	357.52
ε_c	0.008	0.014
$(\alpha_a a+\alpha_b b)/3$	0.833	0.467
d_c	0.810	1.577
	Rotational limitations fail	**Rotational limitations OK**
$\frac{2b_e GSA_{eff}}{3t}$	56365.10	11066.64
	Stability OK	**Stability OK**

APPENDIX – E

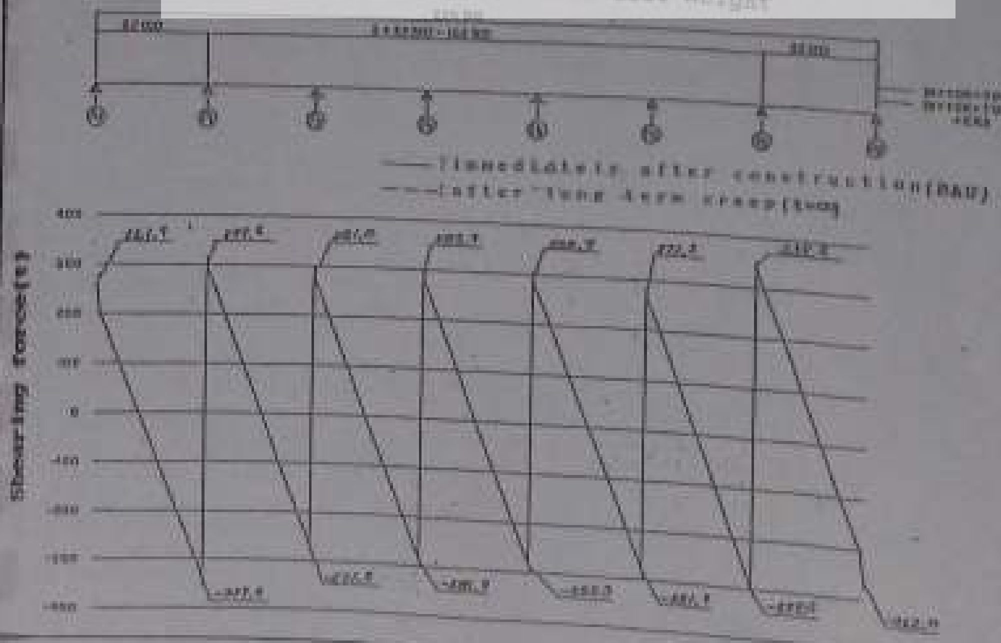
EXTRACTIONS OF ORIGINAL DESIGN REPORT

5) Calculation of Sectional Forces

Bending moment diagram due to self weight



Shearing force diagram due to self weight



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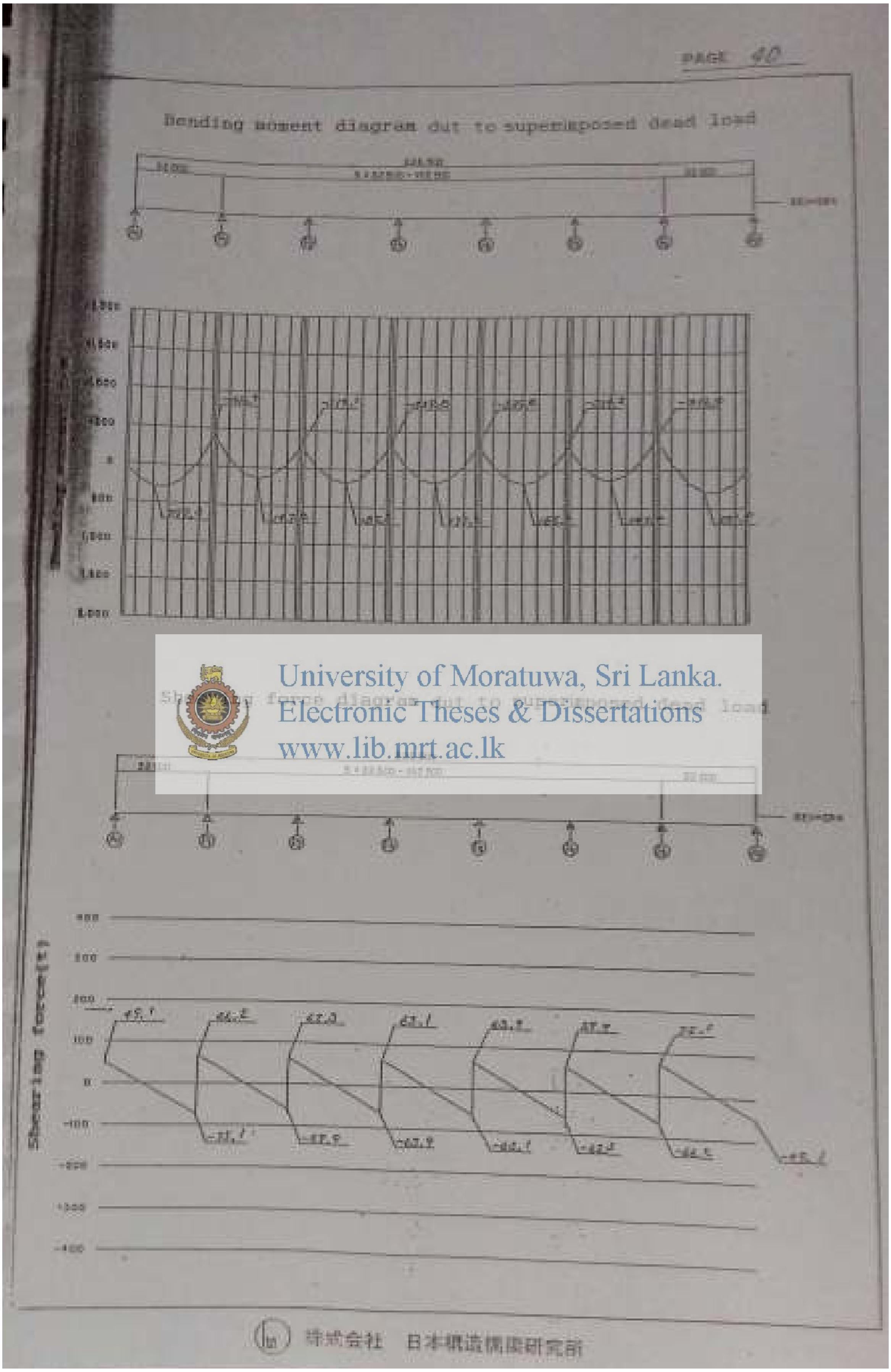
PAGE 40
Bending moment diagram dut to superimposed dead load
Shearing force diagram dut to superimposed dead load
Shearing force(t)
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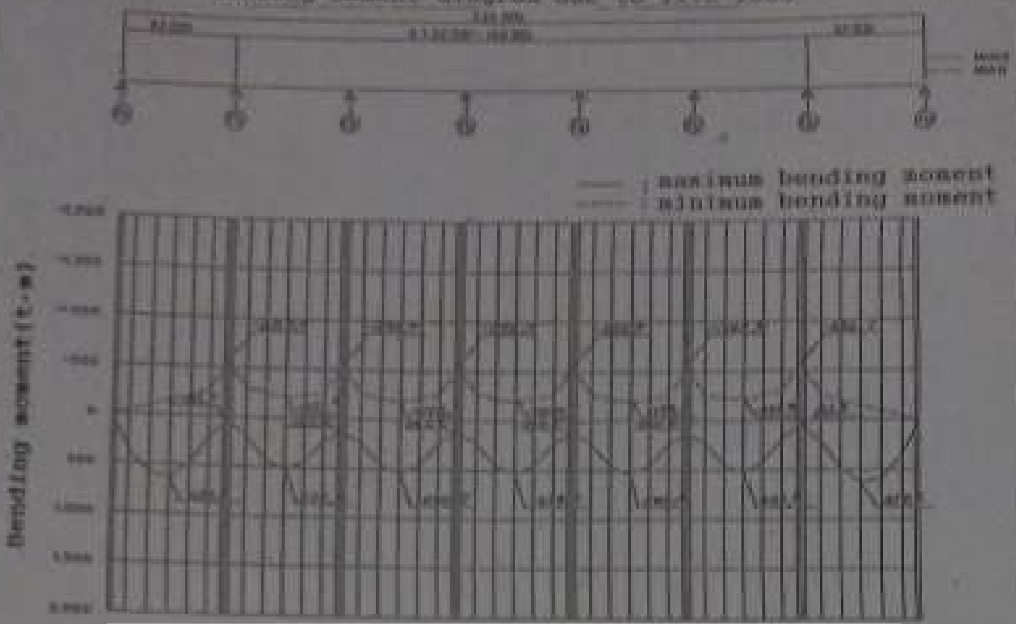
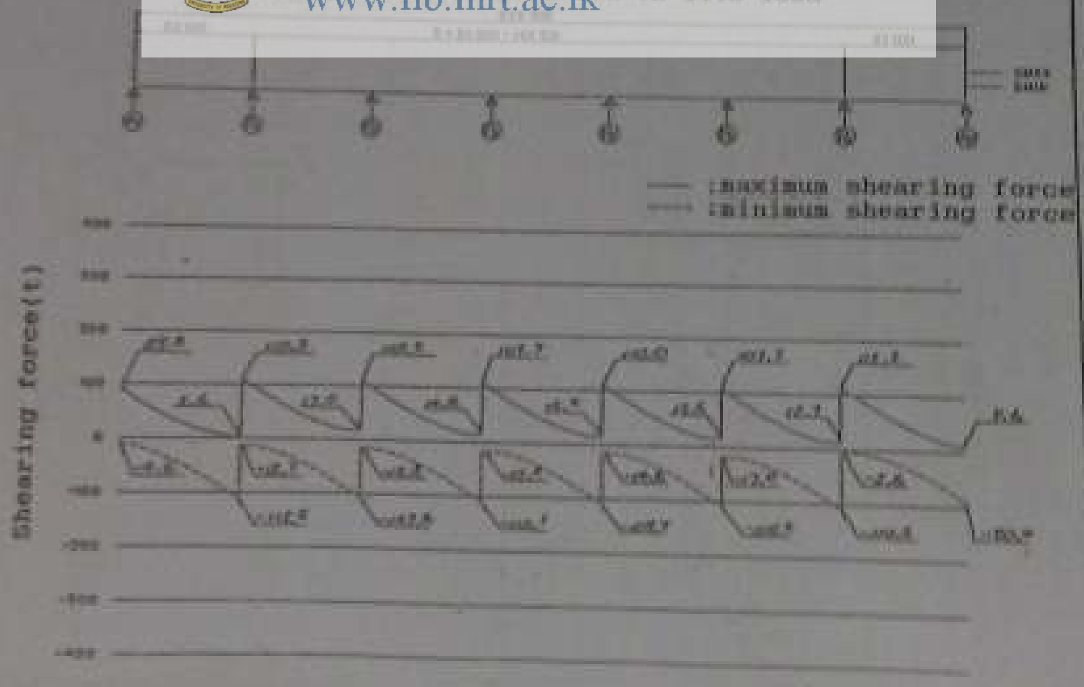
Bending moment diagram due to live load



Shearing force diagram due to live load



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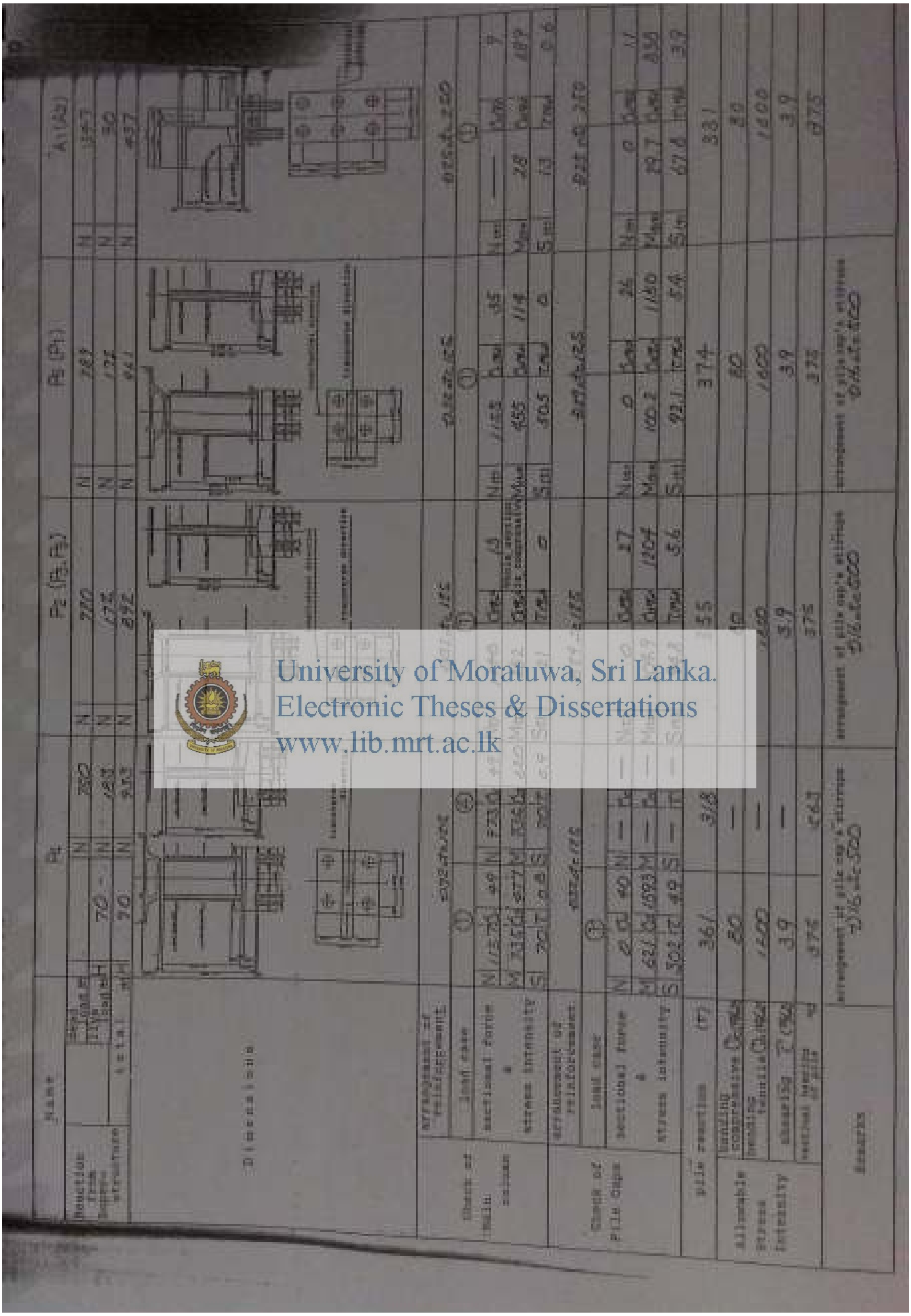

GENERAL NOTES

The following notes shall be taken into account as applicable when reading the drawings.

They are summarized here for ease of reference even if in their majority contained in the Specification.

1. All levels and dimensions as shown on the drawings are in meters and millimeters respectively, except where described.

2. Concrete for structural members

Design compressive strength and Young's modulus for structural members shall be as follows:

Table 1 Design Compressive Strength and Young's Modulus of Concrete

Structural members	Design compressive strength	Young's Modulus
Main girder	[illegible] kg/cm²	[illegible] × 10⁵ kg/cm²
Piers, Abutments, Piles, [illegible]	[illegible] kg/cm²	[illegible] × 10⁵ kg/cm²

3. Reinforcing bars

1) Reinforcing bars shall conform to SD35 according to JAPAN INDUSTRIAL STANDARD (JIS) G3112-1986 and shall have the minimum yield strength of [illegible] kg/mm².

2) Bending, cutting, hook and anchoring shall comply with Specifications for Highway Bridges (1977, Japan Road Association, hereinafter referred to as Japanese Bridge Specifications).

3) Spacing, splicing and development of reinforcing bars shall comply with Specifications for Highway Bridges.

4) Lap length shall be satisfied as follows.

Tension reinforcement .... la and 20D
Compression reinforcement .... 0.8 la and 20D

where, D is nominal diameter (mm) of reinforcing bars.
la is splicing length calculated from bond stress intensity of bar and concrete in accordance with Japanese Bridge Specifications.

5) Maximum length of reinforcing bars shall not exceed 12 m in general.

6) Cross sectional area, unit weight, etc. shall comply with JIS G3112-1985.

7) Minimum concrete cover for reinforcing bars shall be satisfied as following values:

Box girder outer and inner perimeter .... [illegible] mm
Bored piles .... 150 mm
Piers and abutments .... 100 mm

8) Symbol for reinforcing bars used in the drawings are as follows:

GIRDER

S : [illegible] upper slab
W : Reinforcement in web
F : Reinforcement in bottom slab
C : Curb
[illegible]

PIER, PILE

[illegible]

ABUTMENT

A : [illegible]
F : Pile cap
W : [illegible]
C : Approach slab

4. Prestressing

1) Low relaxation [illegible] strand (SWPR-7B, diameter 12.7 mm) as specified in JAPAN INDUSTRIAL STANDARD (JIS) G3536-1971) shall be used for longitudinal prestressing.

2) Prestressing bar ([illegible] 95/120, diameter 32 mm, as specified in JIS G3109-1971) shall be used for longitudinal, transverse and vertical prestressing.

3) Table 2 Design Values of Prestressing Tendons

		Cable	Bar
Notation		SWPR-7B	SBPR-95/120
Area	(mm²)	[illegible]	[illegible]
Weight	(kg/m)	[illegible]	[illegible]
Ultimate tensile strength	(kg/mm²)	190	120
0.2% proof stress	(kg/mm²)	160	95

4) Initial tensioning stress is as follows:

SWPR-7B = [illegible] kg/mm²
SBPR-95/120 = [illegible] kg/mm²

Internal diameter of sheaths of strip metal shall be specified as follows:

	Cable	Bar
General part	[illegible]	[illegible]
Sheath joint	[illegible]	[illegible]
Coupler	—	[illegible]

8) Symbol of cables and bars on the drawings are defined as follows:

- 1 to 7 are cables which are arranged in the main girder.
- 01 to 08 are bars which are arranged in the upper slab.
- 101 to 110, 201 and 202 are bars which are arranged in the bottom slab.
- A1 to A[illegible] and B1 to B[illegible] are bars for vertical prestressing.
- 1001 to 1424 are bars for transversal prestressing.

5. Miscellaneous facilities

1) Miscellaneous facilities of the bridge (bearing, expansion joint, drainage, railing, etc.) shall comply with the relevant articles of JIS.

2) All symbols and abbreviations on these drawings are in accordance with definition of [illegible] where described on each drawings.

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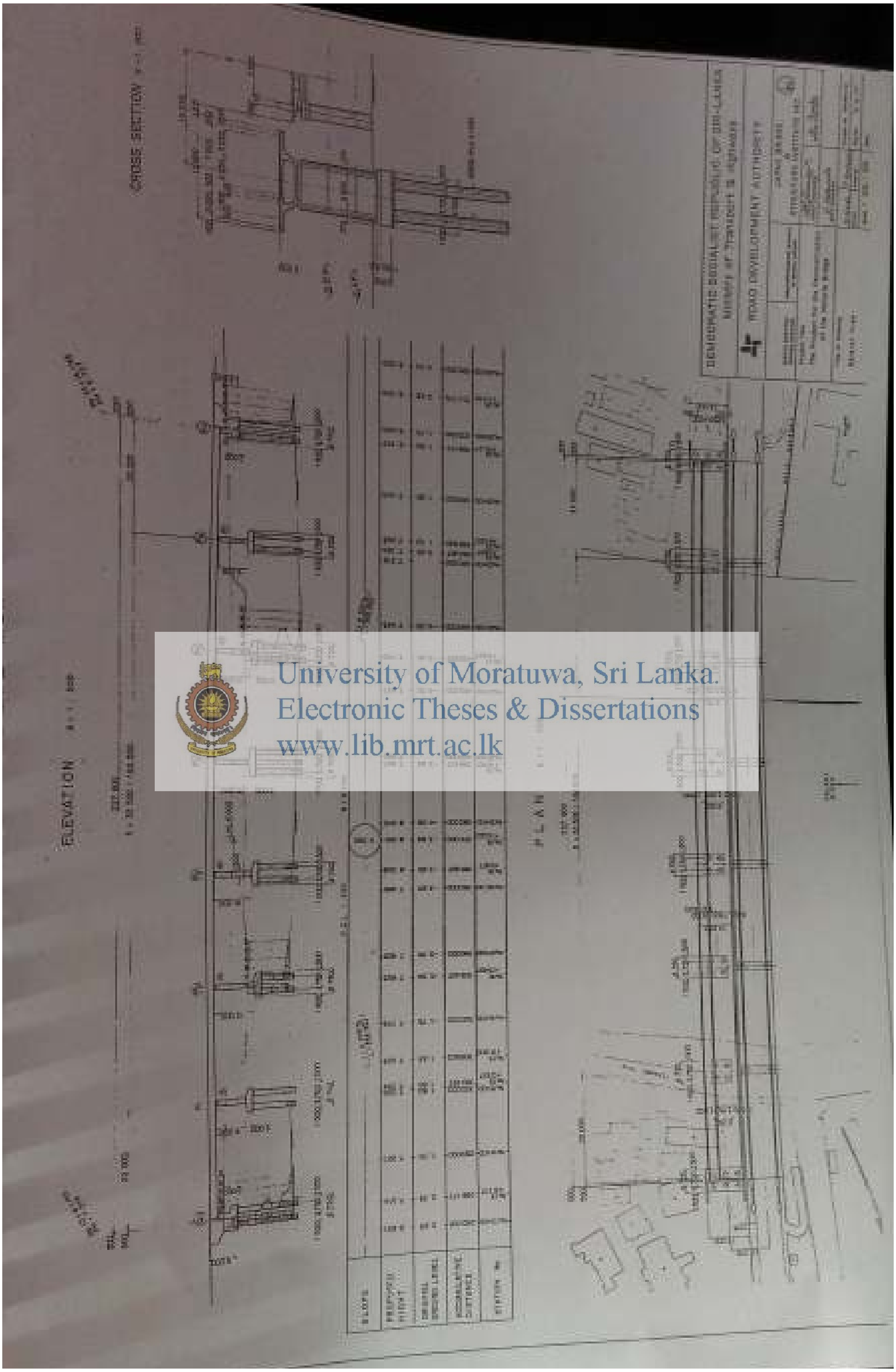

ELEVATION
CROSS SECTION
PLAN
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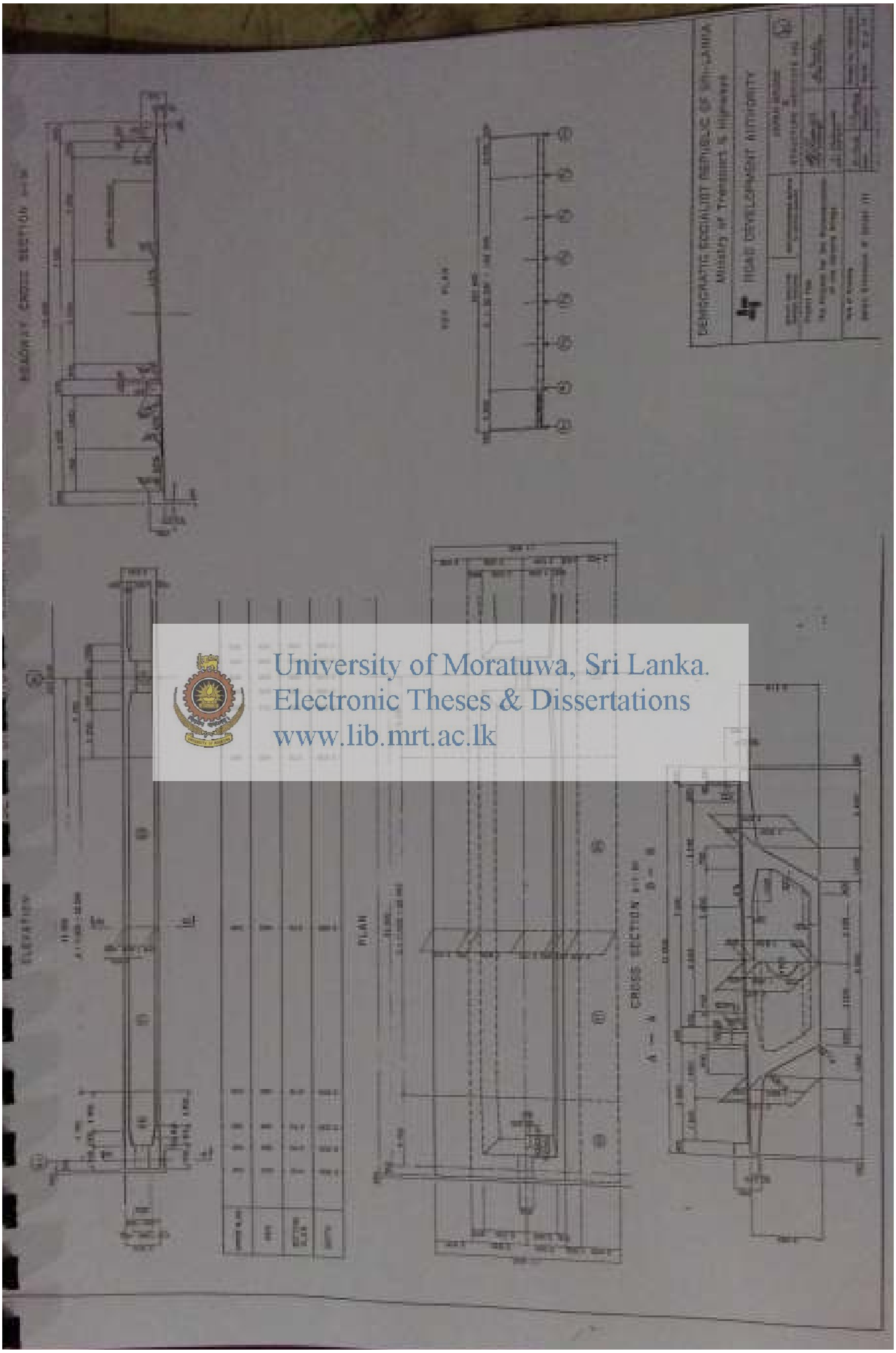
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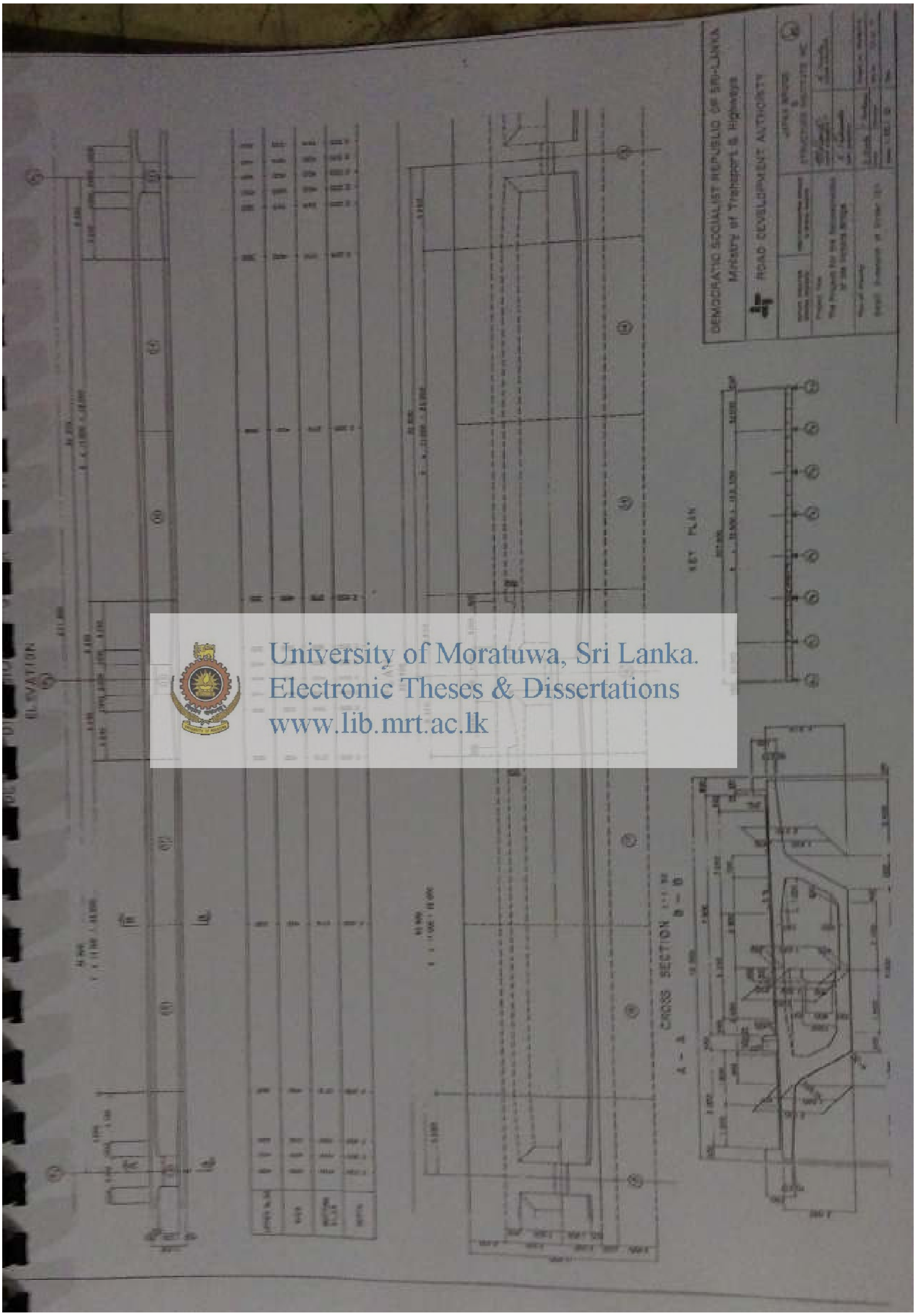
ROADWAY CROSS SECTION
ELEVATION
PLAN
CROSS SECTION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KEY PLAN
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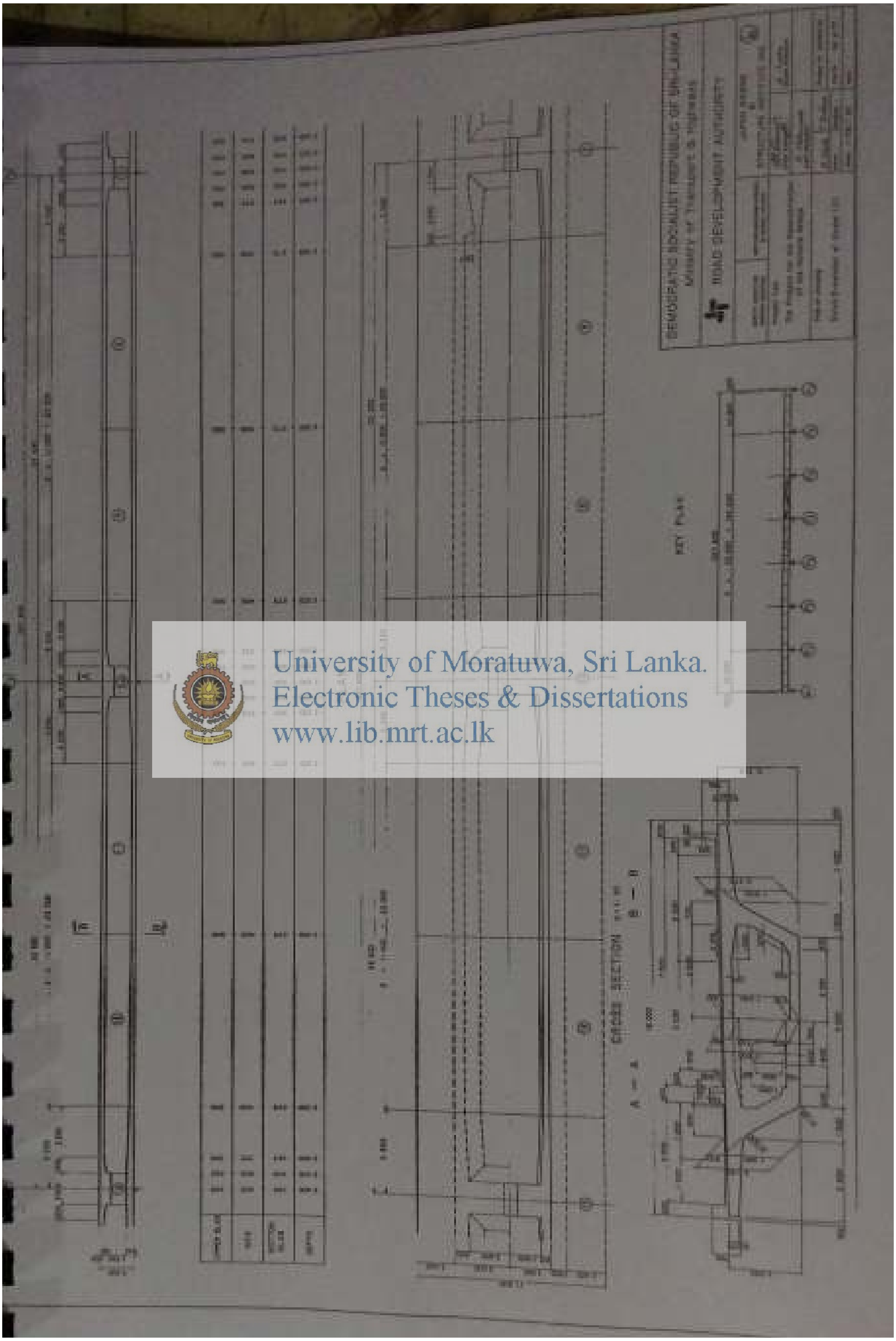
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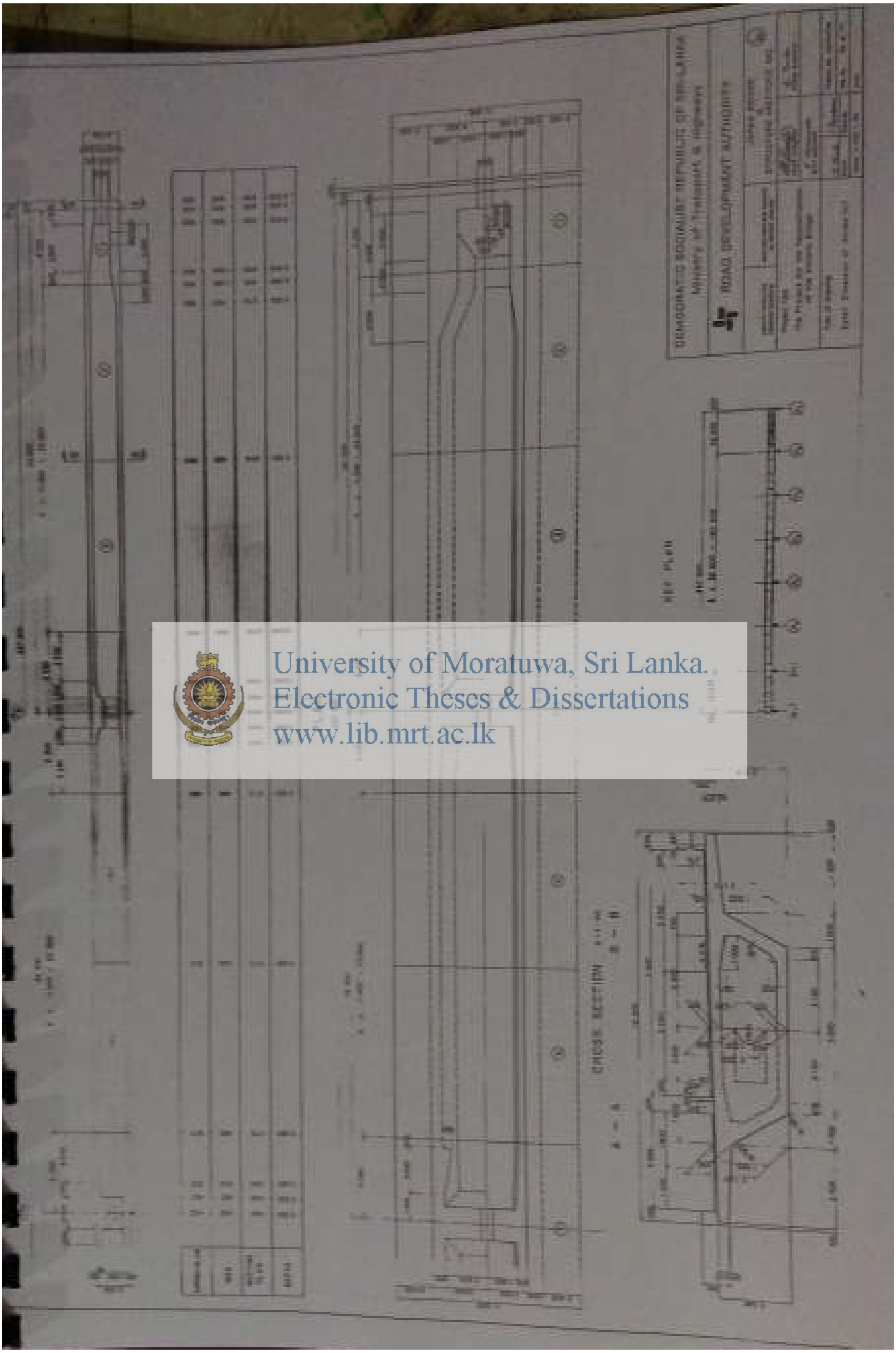
KEY PLAN
CROSS SECTION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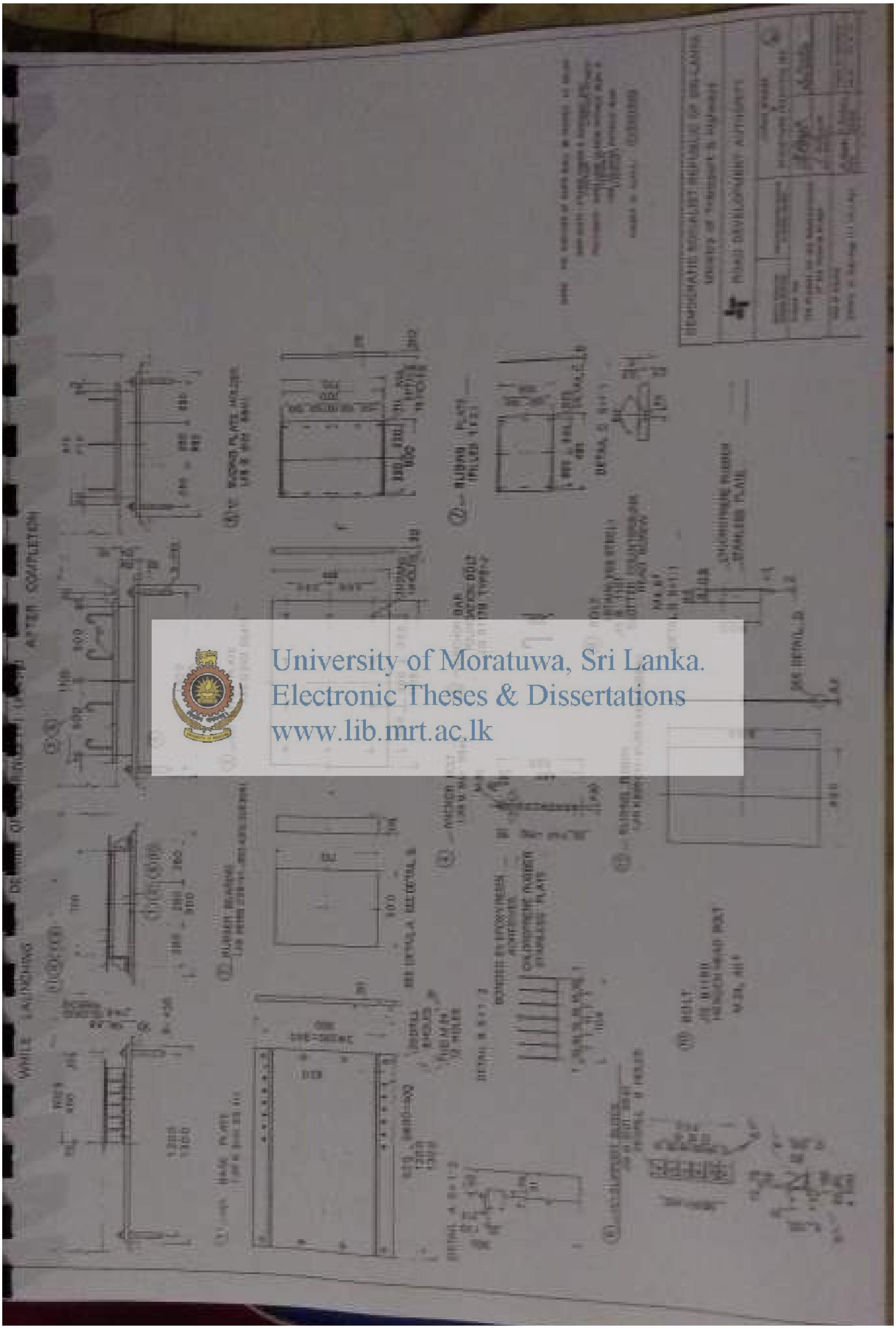
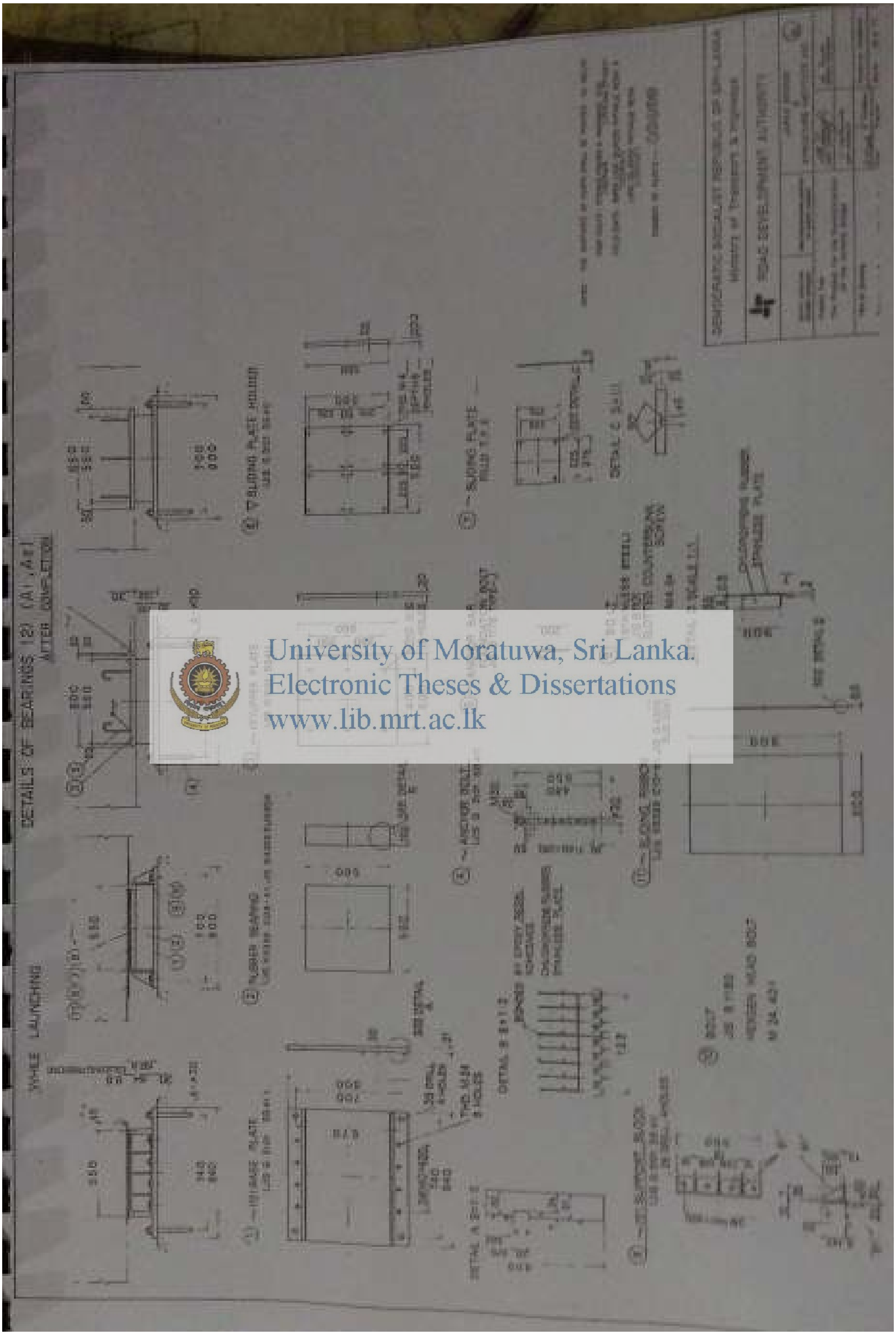
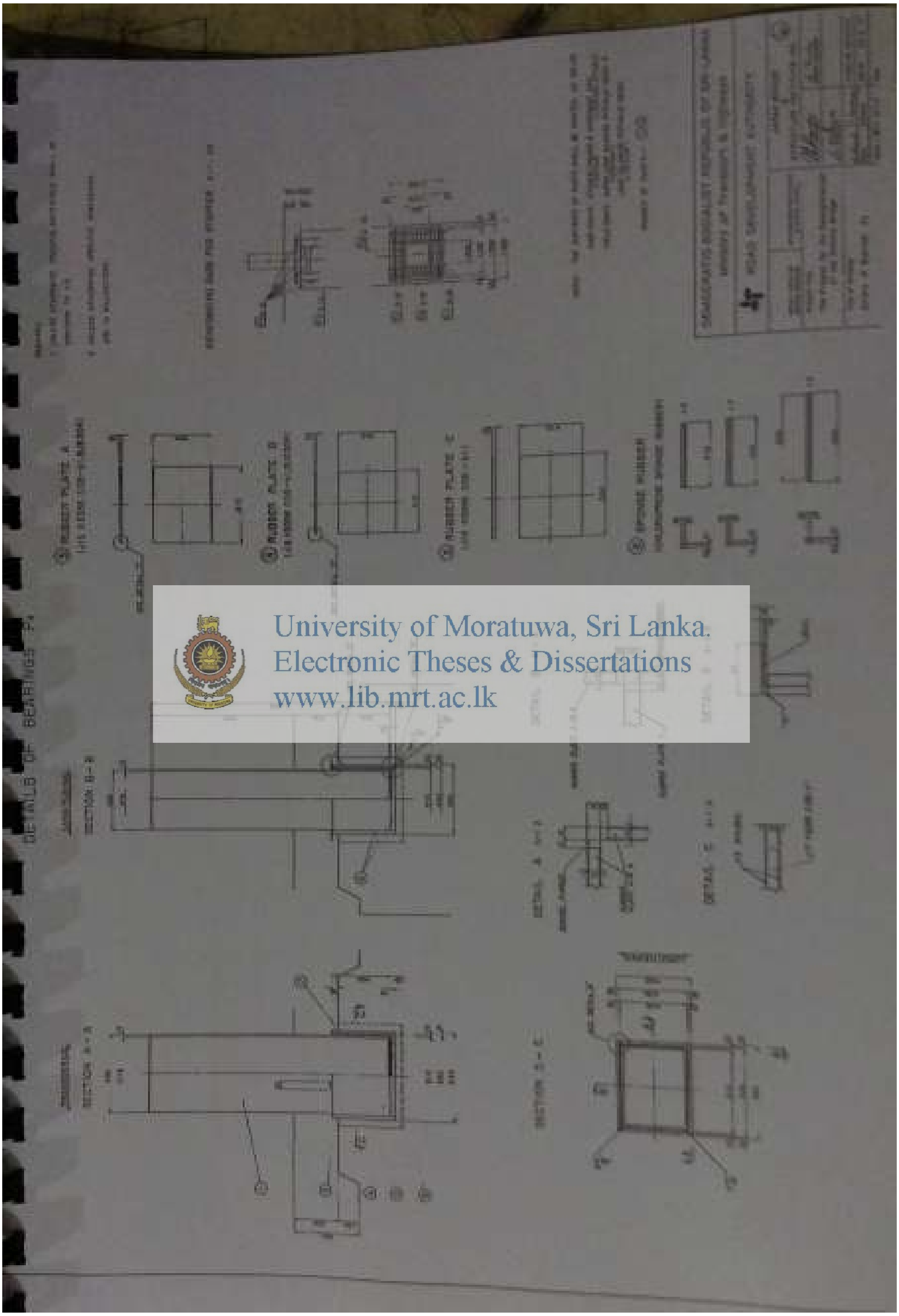
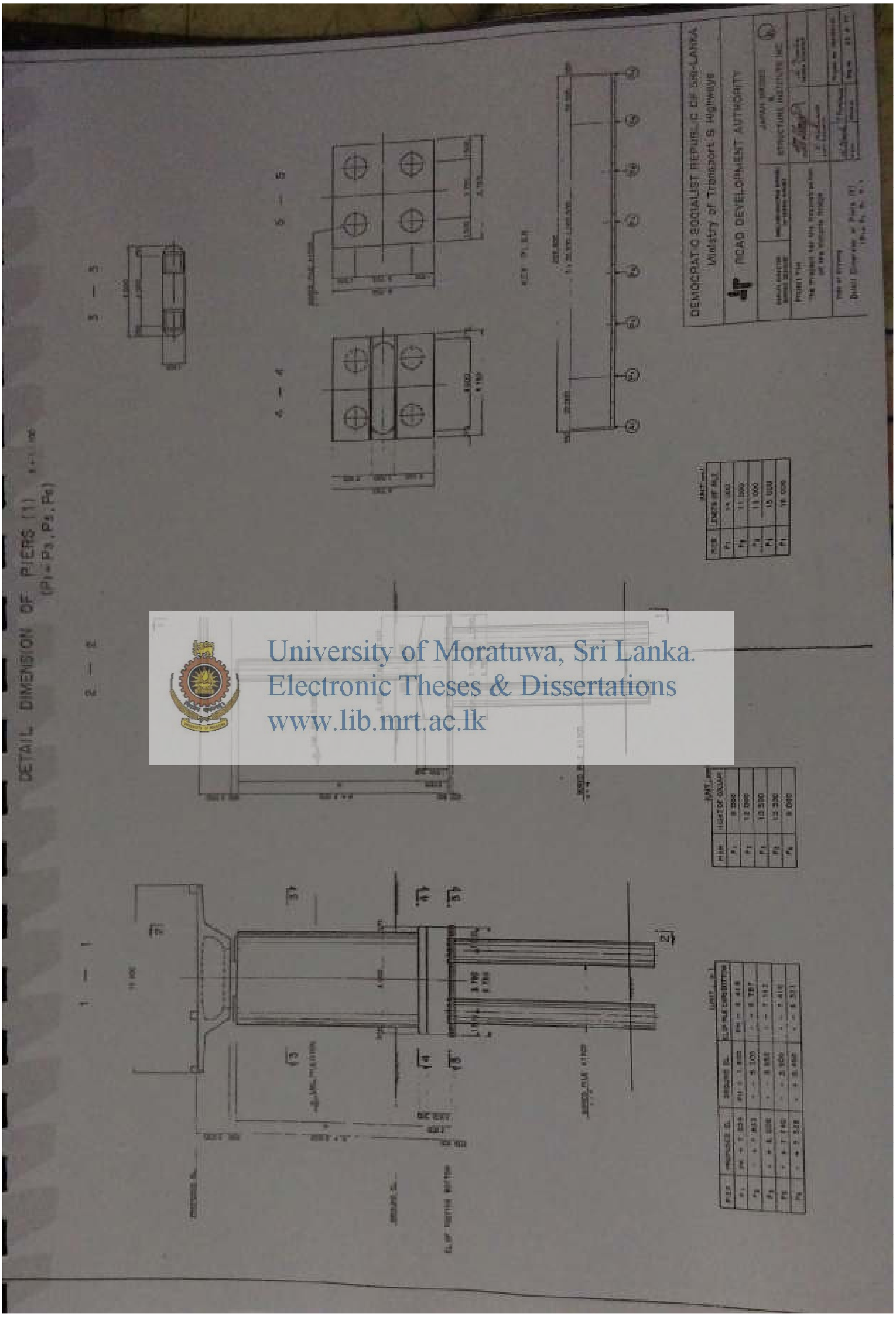
Ministry of Transport & Highways
ROAD DEVELOPMENT AUTHORITY
KEY PLAN
CROSS SECTION




DETAILS OF BEARINGS (2) (A1, A2)
AFTER COMPLETION
WHILE LAUNCHING




DETAIL DIMENSION OF ABUTMENTS
KEY PLAN
DEMOCRATIC SOCIALIST REPUBLIC OF SRI LANKA
ROAD DEVELOPMENT AUTHORITY


DETAIL DIMENSION OF PIERS (1)
(P1~P3, P5, P6)
1 — 1
2 — 2
3 — 3
4 — 4
5 — 5
KEY PLAN
DEMOCRATIC SOCIALIST REPUBLIC OF SRI-LANKA
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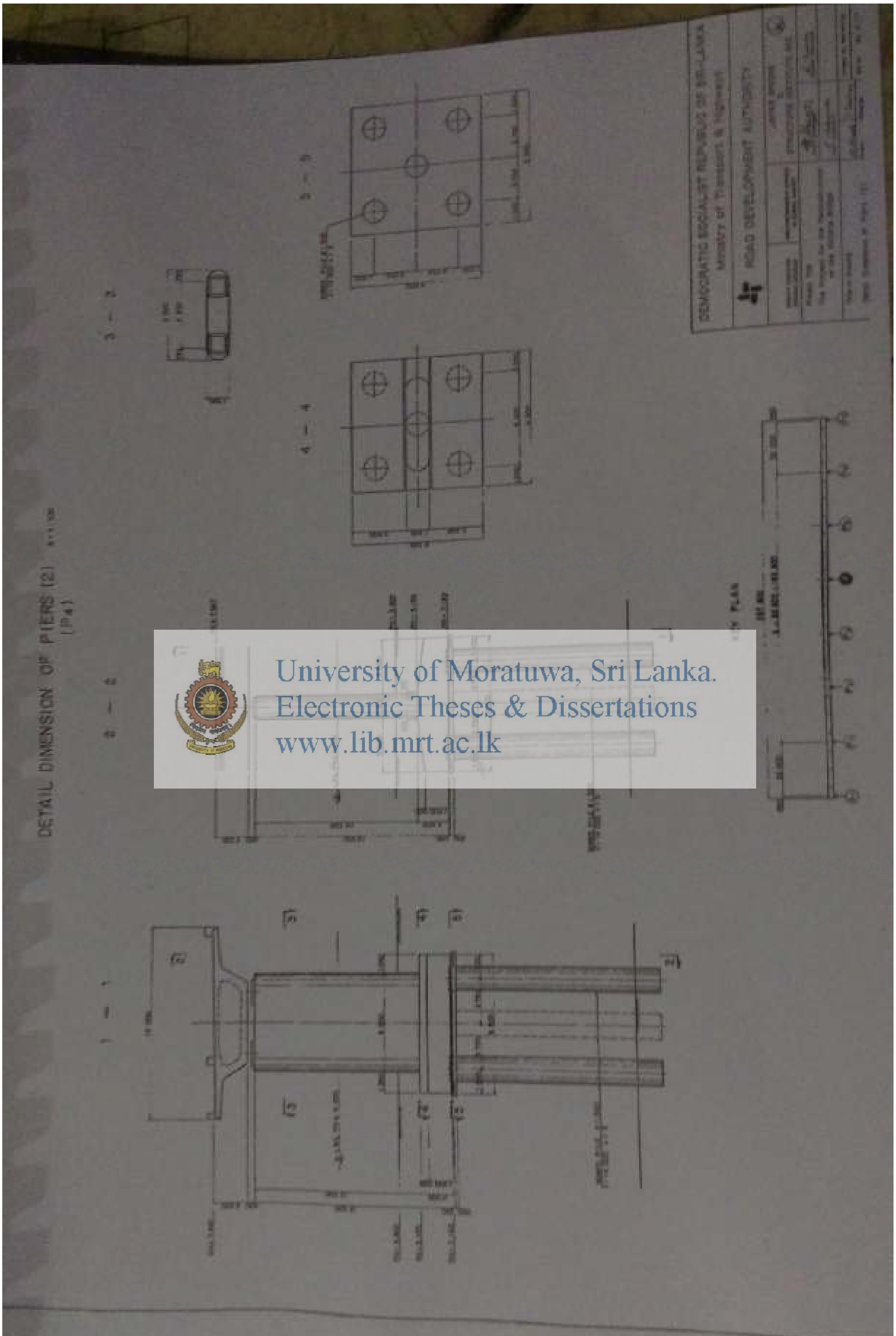
DETAIL DIMENSION OF PIERS (2)
(P4)
1 — 1
2 — 2
3 — 3
4 — 4
5 — 5
DEMOCRATIC SOCIALIST REPUBLIC OF SRI LANKA
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