DEVELOPMENT OF EARTHQUAKE RESISTANT DESIGN GUIDELINES FOR BRIDGES IN SRI LANKA

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Degree of Master of Science in Structural Engineering Design

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DECLARATION

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Name of the supervisor: Prof. C.S. Lewangamage

…………………………….. …………………………

Signature of the supervisor Date

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Abstract

Sri Lanka is an island located on the Indo-Australian tectonic plate and the location of the country is far away from the plate boundary where inter-plate earthquakes are possible to occur. However, there are earthquake records during the recent past which have considerable magnitude. Moreover, intraplate earthquake risk is there which is possible to occur without prior warning. Therefore, Sri Lanka is no longer be considered safe from seismic threats.

With the increasing demand to improve the road network in the country, it is necessary to reconstruct existing bridge structures. However, there are no seismic design guidelines to use for bridge design procedure in Sri Lanka.

This study is therefore aims to formulate earthquake-resistant design guidelines for bridges in Sri Lanka. For the purpose of formulating the design guideline, bridge classification into three different important classes is proposed based on the relevant classifications in similar codes such as EN 1998-2:2005 , IS 1893-3:2014 and AS 5100.2-2004. Important factors and return periods are proposed based on the guideline given in EN 1998-1:2004. Further, Elastic response spectrums for rock or hard soil are selected based on the available response spectrum for Sri Lanka. However, there is no response spectrum defined for medium soil and soft soil. Therefore, it is proposed to use the response spectrum available in IS 1893-1:2002 for medium and soft soil since soil and rock types are more similar when both countries lie on the same tectonic plate.

This study proposed a seismic analysis approach for bridges using either EN 1998-2:2005 or AS 5100.2-2004. Moreover, suitable design parameters such as peak ground acceleration values are proposed to select according to the available national data.

There are three case studies carried out to illustrate the use of developed guidelines for seismic analysis of bridges in Sri Lanka. Three Case studies were carried out for bridges in the category of important class I, important class II and important class III by using the design codes of AS 5100.2-2004, IS 1893-3:2014, and EN 1998-2:2005. For case study 1, base shear values were calculated using the static lateral force method of analysis and three different design codes provided different results of 50kN, 51kN, and 36kN. For case studies 2 and 3, static analysis was carried out using selected three design codes, and response spectrum analysis was carried out according to EN 1998-2:2005 using the response spectrum defined for rock or hard soil of Sri Lanka. Base shear values of the static analysis results provide different values for three design codes. However, a comparison of the results of response spectrum analysis and fundamental mode method of analysis for Eurocode 8-part2 as the basis of analysis, provided similar fundamental period values and base shear values for both methods.
Results of the case studies illustrate that when the design basis is Eurocode 8-Part 2, it provides an average or lower result of base shear value. Since Eurocode 8 provides an opportunity to use it with national choices, it is more suitable to use EN 1998-2:2005 with a national annex for bridge design. Therefore, a developed guideline with national parameters can be used for the seismic design of bridges in Sri Lanka.

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List of Abbreviations

q	-	Behavior factor
ɳ	-	Damping correction factor with a reference value of ɳ = 1 for 5% viscous damping,
d_{es}	-	d_{Ed} for decks connected monolithically or through fixed bearing
h	-	Depth of cross section
DBE	-	Design Basis Earthquake
d_g	-	Design ground displacement in accordance with EC8-1 Cl. 3.2.2.4 $= 0{,}025.a_g.S.T_C.T_D$
d_E	-	Design seismic displacement in accordance with 2.3.6.1. EC8-2
L_s	-	Distance from the plastic hinge to point of zero moment
L_g	-	Distance beyond which the ground motions may be considered as completely uncorrelated.
d_T	-	Displacement due to thermal movements
d_{eg}	-	Effective displacement of the two parts due to spatial variations of the seismic ground displacement.
L_{eff}	-	Effective length of deck, taken as the distance from the deck joint in question to the nearest full connection of the deck to the substructure.
d_{es}	-	Effective seismic displacement of the support due to the deformation of the structures
Se(T)	-	Elastic response spectrum
EC	-	Eurocode
t	-	Exposure period
Ƴ	-	Importance Factor
d_G	-	Long term displacement due to permanent & quasi-permanent actions
T_B	-	Lower limit of the period of the constant spectral acceleration branch
MCE	-	Maximum Considered Earthquake
l_m	-	Minimum support length ensuring the safe transmission of the vertical reaction, but no less than 400mm
T_n	-	Natural Period

PGA	-	Peak Ground Acceleration
a_g	-	Peak Ground acceleration value
P	-	Probability of exceedance
a_{gR}	-	Reference peak ground acceleration
T_{LR}	-	Reference return period
T_L	-	Return Period
R	-	Risk Factor
α_s	-	Shear Span ratio
S	-	Soil factor
$S_{a/g}$	-	Spectral Acceleration
SPT	-	Standard penetration test
M_s	-	Surface wave magnitude
ULS	-	Ultimate Limit State
T_c	-	Upper limit of the period of the constant spectral acceleration branch
T_D	-	Value defining the beginning of the constant displacement response range of the spectrum
T	-	Vibration period of a linear single-degree-of-freedom system

1.0 INTRODUCTION

1.1 Background

The geographical location of Sri Lanka lies within Indo-Australian tectonic plate and the location of the country is far away from the plate boundary where inter-plate earthquakes are possible to occur. Moreover, there is a belief of the general public that Sri Lanka is located in an area where no earthquakes will occur.

However, it was reported by Abayakoon[1] that 88 earthquakes have occurred around Sri Lanka since 1819. The Magnitude of these earthquakes was varied from 3.0-6.0 on the Richter scale. One of the recent event felt by most Sri Lankans was on 7th December 1993 which the epicenter was located 170km west of Colombo and the magnitude in Richter scale was 4.7. Other than this, The most recent earthquake recorded was on 12th September 2021 which has a magnitude of 4.1 on the Richter scale and the epicenter of the earthquake is 160km away from Hambanthota[2].

The belief of a particular area is seismically inactive has gone wrong many times and one good example is the intra-plate earthquake that hit New Castle, Australia in the year 1989. Considering all these records and recent research studies, structural engineers in Sri Lanka have started to incorporate seismic loads to the structural designs of buildings.

As buildings, Bridges are the most important road structures for road and railway transportation. There are more than 4600 bridges in national highways in Sri Lanka and bridge spans start from 3m and vary up to 180m or more according to the river width or requirement. The Collapse of a bridge in a national highway or in an expressway during a natural disaster situation will make the recovery operations further difficult. Most of the existing bridges in national highways were not designed to cater to seismic effects. Not only existing old bridges, but new bridges were also designed and constructed during the recent past which were not designed to resist seismic effects.

The major reason behind this is that there is no earthquake resistant design guideline available for bridge design in Sri Lanka. Therefore, there is a strong need of developing earthquake resistant design guidelines for bridges in Sri Lanka.

1.2 Objectives

The main objectives of this research work can be highlighted as follows,

1. Classification of bridges into different importance classes
2. Identification of design parameters appropriate to Sri Lanka for earthquake resistant design of bridges.
3. Development of earthquake resistant design guidelines for bridges that ensure satisfactory behavior during an earthquake.

1.3 Methodology

In order to achieve the specific objectives above, the following methodology was adopted.

For the classification of bridges into importance classes, available classifications in other seismic design codes were reviewed. These classifications were compared to provide a suitable bridge classification into importance classes according to the condition of bridges in Sri Lanka.

Return periods and importance factors for selected importance classes were calculated in this study according to the guideline provided in EN 1998-1:2004[3]. In order to select suitable peak ground acceleration values and response spectrum for different soil types, the available seismic studies for Sri Lanka were reviewed. A seismic analysis approach for bridges is proposed after selecting suitable design parameters through a literature review.

Three case studies were carried out to illustrate the use of developed guideline for earthquake resistant design of bridges in Sri Lanka. Csi Bridge 2016 v18.0.1 [4] was used for the finite element modal analysis during case studies. Finally, the results of the analysis under different codes were compared. Moreover, analysis results under the fundamental mode method and response spectrum analysis were also compared.

1.4 Main outcome

According to the results of three case studies carried out using the fundamental mode method, three different codes provided different base shear results. However, analysis carried out using Eurocode 8 (EN 1998-2:2004) [5] provided average or lower base shear result. Analysis carried out using Australian standard (AS 5100.2-2004) [6] or Indian standard (IS 1893-3:2014) [7] provided either average or highest base shear value. Also, response spectrum analysis was carried out using Eurocode 8-Part 2 for case study 2 and case study 3, and results were compared with the fundamental mode method of analysis. For case study 2 and case study 3, Base shear values obtained from the fundamental mode method of analysis and response spectrum analysis were almost equal. Moreover, fundamental period values obtained from the two methods were shown similar values for each case study. This illustrates the use of developed design guideline for earthquake resistant bridge design for Sri Lanka and selecting EN 1998-2:2005 with national parameters is more suitable for seismic analysis of bridges in Sri Lanka.

1.5 Outline of the thesis

1.5.1 Chapter 2 Literature review

An extensive literature review was carried out to find out the necessary details for this research. A literature review was done to obtain more details about Bridges in Sri Lanka, Earthquakes damages to bridges, seismic studies in Sri Lanka, peak ground acceleration, Response spectrum, Performance based design, Classifications of bridges to important classes in different codes and elastic response spectrums used in different codes.

1.5.2 Chapter 3 Selection of suitable design parameters for seismic analysis of bridges in Sri Lanka

The selection of suitable design parameters of this research is described in chapter 3. The First step is the classification of bridges to different important classes. According to the details provided in different codes, suitable important classes for the bridges in Sri Lanka will be defined. Identification of appropriate design parameters for earthquake resistant bridge design in Sri Lanka is described in this chapter.

1.5.3 Chapter 4 - Case Study

Case studies for bridges in different important classes are described in this chapter. The results of these case studies provide an opportunity to understand the significance of seismic effect for bridge design in Sri Lanka.

1.5.4 Chapter 5 – Conclusion and recommendations

In the Final Chapter, conclusions and recommendations are provided for this study. Also, it discusses the areas that need to be studied further.

2.0 LITERATURE REVIEW

2.1 Bridges in national highways

A and B class roads and expressways have more than 4600 bridges functioning in Sri Lanka. Typically, a roadway structure, which has a clear span of more than 3m is considered a bridge. Mahawewa Bridge in outer circular highway is considered as the longest bridge in Sri Lanka which has an overall length of 3100m. Other Large bridges can be named as Awiththawa Bridge across Benthara River which has a length of 695m and Kinniya Bridge has a length of 396m [8].

Primarily, the Bridge design procedure in Sri Lanka does not consider seismic effects. However, it is a precautionary measure to analyze the existing bridges and design the new bridges as earthquake resistant.

2.2 Earthquakes

When rocks are suddenly disturbed, a large amount of energy is released and the resulting vibrations spread out in all directions from the source of the disturbance. An earthquake is the passage of these vibrations. The place of the origin of the earthquake in the interior of the earth is the focus (hypocenter) and the point vertically above the focus on the surface of the earth is the epicenter [9].

Oceanographic and geological studies indicate that the earth crust comprises several stable rock slabs known as crustal plates, bordered by rifts mainly in the ocean, along which the plates are in continuous slow motion relative to one another. The convection currents in the magma are responsible for plate movements. It is along the plate boundaries that most of the earthquakes occur. There are 20 crustal plates on the surface of the earth, out of which the following eight are very large.

1. Indian Plate
2. Australian Plate
3. Eurasian Plate
4. African Plate
5. North American Plate
6. Pacific Plate

7. Antarctic Plate
8. South American Plate

Earthquakes can be classified as following two major types,

Inter-Plate earthquakes,
Intra-plate earthquakes.
Along active boundaries of tectonic plates inter plate earthquakes can occur. There are three types of plate boundaries as convergent, divergent and transform plate boundaries where tectonic plates move towards each other, move away from each other and plates slide past each other [9].

Intra-plate earthquakes occur within the tectonic plates due to internal rupture owing to the presence of faults in the rock strata.

2.3 Bridge Failures due to seismic effects

There are several ways of bridge failures due to seismic effects. Superstructures or substructures can be damaged and structural damage caused by liquefaction of soil are the main reasons for bridge failure due to seismic effects.

2.3.1 Unseating of Bridge Girders

During an earthquake, a bridge girder may become unseated. Because of the earthquake, support distance between abutment or piers may become higher than bridge girder length. That will cause to unseating of the superstructure (see figure 2.1). During an earthquake, many superstructures fell off their seats due to the construction of the bridge with inadequate seating length, weaker shear keys and not having an end diaphragm between girders.

Figure 2.1. Unseating of superstructure

2.3.2 Damage to the substructure

During an earthquake, Substructures damage occurs (see figure 2.2) instances where superstructure is monolithically cast to the substructure. Short stiff substructures carry shear and tall flexible substructures carry bending as main forces. Substructure fails in flexure and shear due to inadequate detailing for ductility.

Figure 2.2. Substructure failure

2.3.3 Structural damage due to ground failure, Liquefaction

Foundation damages mainly happens when the foundation is supported in weak soil or when liquefaction occurs. In stiff soil, foundation movement is very little. Therefore, the damage will be minimal compared to weak soil.

2.4 Peak ground acceleration and response spectrum

2.4.1 Peak ground acceleration

For an infinitely rigid structure(Natural Period, Tn = 0), the mass does not experience any motion relative to the base, and therefore, the acceleration response of the mass will be the same as the acceleration of the ground. The response acceleration at Tn=0 is therefore, termed as Peak Ground Acceleration [9].

2.4.2 Response spectrum

The response spectrum is a standard method of representation of the response of structures to ground acceleration. It is the summarization of response histories in terms of the peak responses of all single degree of freedom systems with different natural periods [9]. The maximum response parameter, generally, the acceleration, can be obtained by the response history for the dynamic systems having different natural periods. From the data so obtained, a graph of maximum response versus natural period for given damping, called the response spectrum.

2.5 Seismic hazard assessments for Sri Lanka

Seismic hazard for Sri Lanka was studied by many researchers. As proposed by Peiris L.M.N. [10], the peak ground acceleration (PGA) value at a rock site for Colombo is 0.026g for 475 years return period. Uduweriya et al. [11] presented results on seismic hazard assessment for the Colombo area. He proposed a value of 0.1g as the PGA value at a rock site for Colombo for 475 years return period.

The seismic hazard map of a country will divide the country into several zones according to PGA values. During the past, Structural engineers in Sri Lanka have faced the issue of designing bridges or buildings against earthquake loading due to the unavailability of PGA values.

However, in the recent past, there were some research work regarding seismic hazards.

According to research done by Senevirathne H.N. et al. [12], they have demonstrated that seismicity around Sri Lanka cannot be ignored and for the design works, Seismic hazard should be incorporated. Further, they have found that within the Mannar rift zone there is a possibility of generating a seismic event that has a magnitude of 4.7 for 475 year return period.

Weerasinghe D.R. et al. [13] perform a study to obtain peak ground acceleration values for some of the major cities in Sri Lanka using the deterministic seismic hazard assessment method (DSHA). They have predicted that peak ground acceleration (PGA) value of 0.13g for Colombo city after their study.

Uduweriya S.B. et al. [14] carried out a study using the probabilistic seismic hazard assessment (PSHA) method to find out seismic hazards around Sri Lanka. As per the results of the study, there were higher PGA values predicted for cities such as Colombo, Jaffna, Mannar and Puttalam.

Dananjaya R.M.S.et al. [15] have carried out a research study to find out response spectra for Sri Lankan cities using the finite difference method. They have presented peak ground acceleration values for 58 cities of Sri Lanka and Colombo is having a PGA value of 0.083g according to that study.

Senevirathna H.N.et al.[16] have proposed a seismic hazard Zonation map for Sri Lanka based on previous studies of deterministic and probabilistic seismic hazard assessment and based on the studies carried out using finite difference method. There are two seismic zones defined here as Zone 1 and Zone 2 as shown in figure 2.3. These zones are having design PGA values of 0.1g and 0.05g.

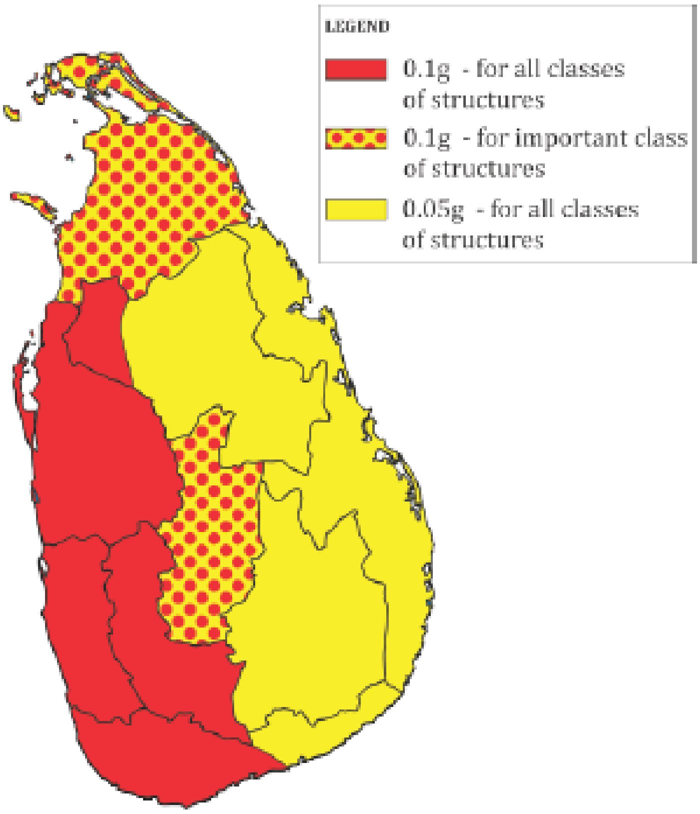


Figure 2.3. Seismic hazard zonation map for Sri Lanka. [16]

No.	City	PGA (g)
1	Galle	0.080
2	Kottawa	0.063
3	Akuressa	0.059
4	Pasgoda	0.065
5	Katuwana	0.068
6	Panamure	0.062
7	Angunakolapalassa	0.044
8	Weheragala	0.036
9	Andiawela	0.029
10	Arugam Bay	0.023
11	Kaluthara	0.082
12	Egal Oya	0.060
13	Idangoda	0.053
14	Kuruwita	0.047
15	Thummodara	0.050
16	Nallathanniya	0.058
17	Maskeliya	0.060
18	Hatton	0.051
19	Lindula	0.047
20	Nanu Oya	0.040
21	Nuwara Eliya	0.045
22	Kandapola	0.042
23	Ragala	0.044
24	Bibile	0.031
25	Kalmunai	0.024
26	Colombo	0.083
27	Ussapitiya	0.060
28	Kandy	0.048
29	Narampanawa	0.044
30	Knuckles1	0.040
31	Meemure	0.041
32	Knuckles	0.037
33	Baticaloa	0.021
34	Chilaw	0.083
35	Ella	0.068
36	Migaswewa	0.058
37	Thalpathwewa	0.057
38	Alutwegedara	0.056
39	Bulanwewa	0.048
40	Dambulla	0.045
41	Kandalama	0.043
42	Kaduruwela	0.034
43	Tambala	0.033
44	Vakarai	0.025
45	Puttalam	0.095
46	Balagollagama	0.080
47	Saliyawewa	0.074
48	Anuradhapura	0.050
48	Mihinthale	0.047
50	Tammanewa	0.044
51	Amunukola	0.038
58	Trincomalee	0.027
53	Ambagahawewa	0.038
54	Katupotana	0.034
55	Nagollewa	0.032
56	Palampoddaru	0.029
57	Ganeshapuram	0.027

Figure 2.4. Peak ground acceleration values at selected cities in Sri Lanka. [15]

Peak ground acceleration values proposed by Dananjaya R.M.S. et al. [15] have covered the highest no of cities (see figure 2.4) in the country than any other study. These PGA values are for a 475-year return period. Also, the aforementioned study considered the effect of ground elevation profile for the generation of seismic waves. Therefore, the values proposed above can be considered for seismic analysis in Sri Lanka. However, these values can be replaced whenever more detailed studies are available.

2.6 Performance based seismic design of bridges

Bridge performance during seismic action should be in the way that bridge should perform adequately and economically reasonable way. The seismic performance of a bridge relates to its behavior under different seismic actions. Most probably, a bridge should be able to use after an earthquake which may be severe, frequent action or occasionally occurs.

In performance based design, performance objectives of clients and designers are incorporated. Simply, an acceptable level of damage to a specified earthquake becomes performance criteria.

2.7 Codes of practices/Guidelines

Each code gives different approaches to seismic design. In this research, the following codes/guidelines were considered and referred thoroughly to obtain guidance to form an earthquake resistant design guidelines for bridges in Sri Lanka.

1. Euro code 8 Design of structures for earthquake resistance, Part 2 Bridges[5]
2. AS 5100.2-2004 Australian Standard Bridge design, Part 2 Design loads.[6]
3. Transit Bridge Manual, Second edition 2003, New Zealand.[17]
4. IS 1893(Part 3):2014 Indian Standard, Criteria for earthquake resistant design of structures.[7]

2.8 Seismic performance requirement in different codes

2.8.1 Eurocode 8 part 2-2005

Seismic design done under this code should consider two requirements, namely no-collapse requirement and minimization of damage requirement. Aforesaid two requirements are under ultimate and serviceability limit states.

Here, minimization of damage condition is considered under serviceability limit state and that also divided to two states as immediate use and operational limit states. In operational limit state bridge satisfies the condition of practically no damage. If there is any repair that can be attended later without obstruction to normal traffic use. In immediate use limit state bridge may have slight damages such as concrete cracking or yielding of reinforcement. Here normal traffic use can be interrupted and slight damages can be repaired in the future without much cost [18].

No collapse requirement under ultimate limit state refers to structural collapse of bridge or stages close to collapse. ULS also divided into two limit states as life safety and near collapse limit states. In life safety limit states, that refers to a condition of the bridge that is significantly damaged and does not suitable for normal traffic use. However, there can be possibilities to use temporary or emergency situations. In this condition, repair is not economical and demolition of bridge is more suitable. The bridge can be reached to near collapse state when it is heavily damaged. That can be very unsafe to use a bridge even for emergency use.

2.8.2 IS 1893(Part 3):2014 Indian Standard

In IS 1893(part 3):2014 code for bridges and retaining walls also considers limit state design in its design philosophy. According to the code, performance requirements for serviceability limit state and the ultimate limit state are as follows.

For serviceability limit state, bridge design should consider the design basis earthquake (DBE) [7]. The bridge may get slight damage during this type of earthquake and there is no need for traffic reduction. These damages also can be attended to later without attending immediately.

For the ultimate limit state, bridge design should consider maximum considered earthquake (MCE) [7]. There may be significant damages to parts of the bridge. Therefore, a bridge is not suitable for normal use but can be used for emergency traffic under this condition.

2.8.3 Performance criteria in Transit bridge manual, 2003 second edition, New Zealand

Similar to other codes, the Transit bridge manual states that when a seismic event is a design return period event, a bridge can be allowed for emergency traffic. Also, when a seismic event is lesser than the design value, damages to the bridge are very lesser, and the bridge can be allowed for normal traffic. This gives the same idea as Eurocode 8’s immediate use or operational limit state.

The Transit bridge manual further states that when a seismic event is greater than the design value, a bridge may have considerable damage. However, the bridge should not be collapsed. This is similar to the no collapse requirement given in Eurocode 8-Part 2.

2.9 Importance classes in different codes

Indian standard IS 1893 part 3:2014 has given different importance factors to different seismic classes as shown in Table 2.1

Table 2.1. Importance factors and seismic classes as given in IS 1893-3:2014. [7]

SI No.	Seismic Class	Illustrative Examples of Bridges	Importance Factor "I"
1	Normal Bridges	All bridges except those mentioned in other clauses	1
ii	Important Bridges	River bridges and flyovers inside Cities	1.2
		Bridges on national and state highways	1.2
		Bridges serving traffic near ports and other centers of economic activities	1.2
		Bridges crossing railway line	1.2
iii	Large Critical Bridges in All Seismic Zones	Long bridges more than 1km length across perennial rivers and creeks	1.5
		Bridges for which alternative routes are not available	1.5

Australian standard AS 5100.2-2004 [6] code also classifies bridges into three different classes. However, it has less description than the Indian standard. Bridge classification according to importance classes in the Australian standard is given in Table 2.2.

Table 2.2. Importance factors and Bridge classification as given in AS 5100-2. [6]

Bridge Classification	Example	Importance Factor
Type i	Bridges not type ii or Type iii	1.0
Type ii	Bridges that are designed to carry large volume of traffic or bridges over other roadways, railways or buildings	1.0
Type iii	Bridges and associated structures that are essential to post earthquake recovery	1.25

However, the Transit Bridge manual, second edition -2003 [17] gives Risk factors of different important categories as shown in Table 2.3. These important categories are according to the average no of vehicles per day at the time of design.

Table 2.3. Importance category and risk factors given in Transit Bridge manual-2003. [17]

Importance Category	R
1 Bridges carrying more than 2500 vehicles/day. Bridges carrying or crossing motorways or railways	1.3
2 Bridges carrying between 250 and 2500 vehicles/day	1.15
3 Bridges carrying less than 250 vehicles/day. Non-permanent bridges	1.0

In Eurocode 8- part 2 clause 2.1.4(P), there is guidance for importance classes of bridges. In Eurocode 8-part 2, bridges have different classifications based on their importance. Importance classes and factors in Eurocode 8(part 2) are given in Table 2.4.

Table 2.4. Importance factors and classes given in EC8-2. [5]

Important Class	Description	Importance factor
i	Bridge is not critical for communications, adoption of either the reference probability of exceedance, PNCR, in 50 years for the design seismic action, or the standard bridge design life of 50 years is not economically justified	0.85
ii	Road and railway bridges not in class I and class III	1.0
iii	Bridges of critical importance for maintaining communications, especially in the immediate post-earthquake period, bridge the failure of which is associated with a large no of probable fatalities and major bridges where a design life greater than normal is required	1.3

2.10 Design Seismic action

Seismic actions are different according to their importance. These different importance classes have importance factors. When peak ground acceleration values available for reference seismic action, peak ground acceleration values for different return periods can be obtained using importance factors. In Sri Lanka, PGA values are available mostly for 475 year return period.

Therefore, to obtain PGA values for different return periods, we have to use the method proposed in Eurocode 1998-1 clause 2.1(4P).

Importance factors (ϒ) for different return periods can be determined using the following relationship.

$\Upsilon \sim (T_{LR}/T_L)^{-1/k}$ …………………………………………………………… (2.1)

Where T_{LR} and T_L are the reference return period and the return period in which the same probability of exceedance as in the T_{LR} years is achieved [2].

Using values of importance factors for different return periods and the PGA value of them can be determined using the following relationship.

$a_g = a_{gR}.\Upsilon_1$ ……………………………………………………………………(2.2)

Where a_g is the PGA value for a required return period (Eg. 1000 years, 1500 years) and a_{gR} is the reference PGA value on a rock site from the seismic zonation map.

2.11 Elastic response spectrum

Sri Lanka does not have extensive research findings on earthquakes. Seismic studies in Sri Lanka are in the preliminary stage. Therefore, there is no response spectrum developed for different soil classes in Sri Lanka. However, other countries in Indo-Australian tectonic plate such as India and Australia have developed their codes for earthquake resistant design, and they have already developed response spectrums for different soil classes. A comparison of different soil classes given in different codes is given in Table 2.5 and Table 2.6.

Table 2.5. Ground types and corresponding SPT N values given in EC8-2. [5]

Ground Type	Description	SPT N Value
A	Rock or other rock-like geological formation, including at least 5 m of weaker material at the surface.	-
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of meters in thickness, characterized by a gradual increase of mechanical properties with depth.	>50
C	Deep deposits of dense or medium- dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters	15-50
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-fine cohesive soil.	<15

Table 2.6.Ground types and corresponding SPT N values given in IS 1893-1. [7]

Ground Type	Description	SPT N Value
Type i	Rock or Hard Soil: Well graded gravel and sand gravel mixtures with or without clay binder, and clayey sands poorly graded or sand clay mixtures	>30
Type ii	Medium soil: All soils with N between10 and 30 and poorly graded sands or gravelly sands with little or no fines	10-30
Type iii	Soft soils	<10

2.11.1 Horizontal elastic response spectrum

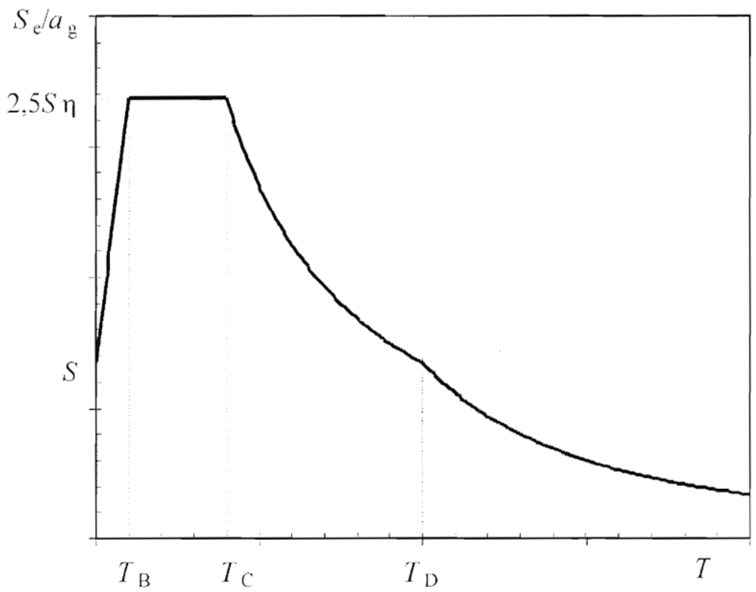


Figure 2.5. Shape of elastic response spectrum in Eurocode 8. [3]

As per Eurocode 8-part 1, two types of spectra can be used. Those are type 1 and type 2 spectra.

Type 2 spectra is recommended to use when earthquake are considered to have a surface-wave magnitude, Ms, not greater than 5.5. When Ms greater than 5.5, type 1 spectra is adopted [2].

Eurocode 8-part 1 table 3.2 and table 3.3 provides values for the parameters of different ground types of type 1 spectrum and type 2 spectrum. Table 3.2 of Eurocode 8-part1 is shown in Table 2.7.

Table 2.7.EN 1998-1 Table 3.2. [3]

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

Elastic response spectrum for 5% damping for the horizontal component of a seismic action(see Figure 2.6) is defined using the expressions given below in IS 1893-part 1 [19]

For rocky or hard soil sites

$$Sa/g = \begin{cases} 1+15T & 0.00 \le T \le 0.1 \\ 2.5 & 0.1 \le T \le 0.4 \\ 1.00/T & 0.4 \le T \le 4.0 \end{cases} \quad \text{……………………….. (2.3)}$$

For medium soil sites

$$Sa/g = \begin{cases} 1+15T & 0.00 \le T \le 0.1 \\ 2.5 & 0.1 \le T \le 0.55 \\ 1.36/T & 0.55 \le T \le 4.0 \end{cases} \quad \text{……………………… (2.4)}$$

For soft soil sites

$$Sa/g = \begin{cases} 1+15T & 0.00 \le T \le 0.1 \\ 2.5 & 0.1 \le T \le 0.67 \\ 1.67/T & 0.67 \le T \le 4.0 \end{cases} \quad \text{……………………. (2.5)}$$

Where S_a is spectral acceleration, g is the acceleration due to gravity and T is period.

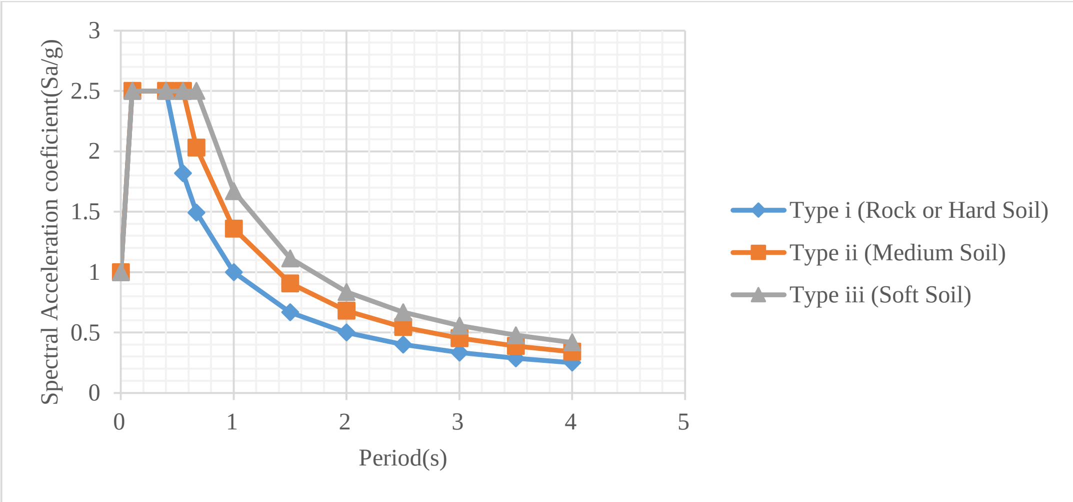


Figure 2.6. Elastic response spectra for 5% damping for rock and soil in IS 1893-1. [19]

Senevirathna H.N. et al. [16] have proposed a response spectrum for Sri Lanka with 5%damping for rock or hard soil sites as shown in Figure 2.7. The following expression used in defining the proposed response spectrum.

$$SA = \begin{cases} (1+10T)*PGA & 0.00 \leq T \leq 0.15 \\ 2.5*PGA & 0.15 \leq T \leq 0.5 \\ 1.25/T*PGA & 0.5 \leq T \leq 4.0 \end{cases} \qquad \text{............... (2.6)}$$

Where SA is spectral acceleration, PGA is peak ground acceleration and T is period.

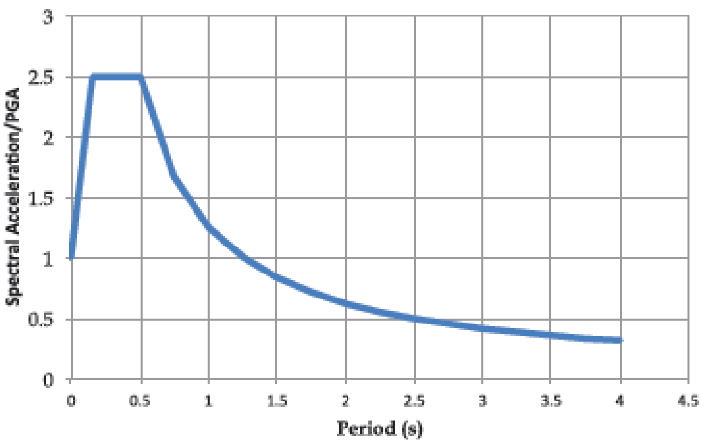


Figure 2.7. Proposed response spectrum for Sri Lanka. [16]

2.12 Evaluation of seismic capacity of existing bridges

New Bridges should be able to withstand an earthquake if earthquake design guidelines are applied at the design stage. However, there are more than 4600 existing bridges are functioning on national highways at the moment and those bridges were not designed to withstand seismic effects. Moreover, these bridges need to be regularly inspected and evaluated to check whether retrofitting is required.

Aluthapala U.L. et al. [20] have carried out a study on “Evaluation of seismic capacity on existing highway bridges in Sri Lanka”. They have proposed a methodology according to the Seismic Retrofitting Manual for highway bridges published by the Federal Highway Administration (Report No. FHWA-RD-94-052) which is used to rank the bridges to identify the priority of those to retrofit. Higher rank implies that detailed evaluation is required for retrofitting. This method is very useful to implement for the evaluation of the seismic capacity of existing bridges.

3.0 SELECTION OF SUITABLE DESIGN PARAMETERS FOR SEISMIC ANALYSIS OF BRIDGES IN SRI LANKA

3.1 Selection of suitable design code for seismic analysis of bridges

Sri Lanka does not have its own design code to perform seismic designs. Structural designs were done in the country using British standards for a long period of time and now the country is moving towards using Eurocodes for structural designs.

In this research, there are four different codes of practices examined which are used to perform seismic analysis. All these codes have used Importance classes, importance factors, and response spectrums for their seismic analysis.

Eurocode gives chance for designers to choose national parameters, which define local seismic characteristics through national annexes. Therefore, we can use Eurocode with our national annex as our design code.

Also, there should be consistent design guidelines or standards for the design of bridge structures in the country. Since we have already started to use Eurocodes for structural designs, it requires to use Eurocode 8 for seismic analysis with other parts of the Eurocodes to make the design standard more consistent.

Therefore, it is more suitable to adopt Eurocode 8 as the seismic design standard for Sri Lanka. In the preceding section of this research, parameters which are needed to be determined nationally will be determined based on the available seismic data and seismic researches of the country. Special consideration will be given to classify bridges in Sri Lanka into different importance classes and to find out importance factors which will lead to find design seismic action.

Also, elastic response spectrums will be defined according to available different soil classes in the country. The linear elastic method of seismic analysis is the only considered method during this study.

3.2 Bridge classification in to different important classes

A bridge is a structure that has a clear span of more than 3m. We have bridges with spans starting from 3m to more than 180m in Sri Lanka. When designing for different seismic actions, different importance will be given according to span, location of the bridge and post-earthquake recovery. According to that, there will be different return periods and different importance factors incorporated in seismic design. To formulate a suitable importance class for bridges in Sri Lanka, importance class classification in Eurocode 8-part 2, IS 1893-3, AS 5100-2, and Transit Bridge manual considered. The proposed bridge classification is given in Table 3.1.

Table 3.1. Proposed bridge classification in to importance classes for Sri Lanka

Importance Class	Classification	Examples
I	Bridges of minor importance	All other bridges except in Class II and III
II	Bridges of significant importance carries large volume of traffic	Bridges in A and B class roads River bridges and flyovers in cities Bridges over roadways , railways and buildings
III	Bridges of critical importance	Large span bridges which have bridge span more than 50m. Bridges of critical importance for immediate post-earthquake recovery. Bridges in Expressways. Major bridges where large no of fatalities probable

3.3 Return period values

According to EC8-1 clause 2.1(1P) reference probability of exceedance for 50 years is 10% and the reference return period is 475 years [3]. Peak ground acceleration (PGA) values for Sri Lanka obtained for reference return period of 475 years by different researchers.

It is required to determine the return periods for different important classes. These return periods approximately determined using the following expression.

T_R= -T_L/ln(1-p) [18] …………………………………………………………..(3.1)

Where T_R = Return Period

T_L = Design Life, 50years

P = Probability

For Importance class II, It is considered "Life Safety" as the performance criteria. It is a rare seismic action which has a low probability of exceedance during the service life.

Therefore, Probability of Importance class II, P = 10%

T_R = -T_L/ln(1-p) ………………………………………………………..(3.2)

= -50/ln(1-0.1)

= 474

~ 475years

For Importance class III,

Performance criteria is Near Collapse which is a very rare seismic action which has low probability (2-5%) exceedance during the design service life.

Therefore, Probability of Importance Class III = 2%

T_R = -T_L/ln(1-p)

= -50/ln(1-0.02)

= 2474

~ 2500years

For Importance Class I,

For important class I, a seismic action has a higher chance of occurrence. This seismic action may cause minor damage to bridge components of bridge. Performance criteria is damage limitation criteria in serviceability limit state.

Mean return period can be calculated using Probability > 10%. Also, return period can be considered as twice the design service life.

Therefore, Return period for Importance Class I = 100 years

Proposed return periods with important classes is given in Table 3.2.

Table 3.2. Proposed return period values for different important classes for Sri Lanka

Important Class	Return Period(Yrs.)
I	100
II	475
III	2500

3.4 Importance factors for corresponding return period values

However, peak ground acceleration values do not available for the different probability of exceedance other than 475 years. Therefore, an approximate relationship is given in EN 1998-1 clause 2.1(4) can be used to find importance factors and those important factors can be used to find out peak ground acceleration values of different important classes.

$\Upsilon_1 \sim (T_{LR}/T_L)^{-1/k}$ ………………………………………………………………….. (3.3)

Where T_{LR} and T_L are the reference return period and the return period in which the same probability of exceedance as in the T_{LR} years is achieved and k is the exponent. This relationship can be plotted as shown in Figure 3.1 and that can be used to determine importance factors for required return periods.

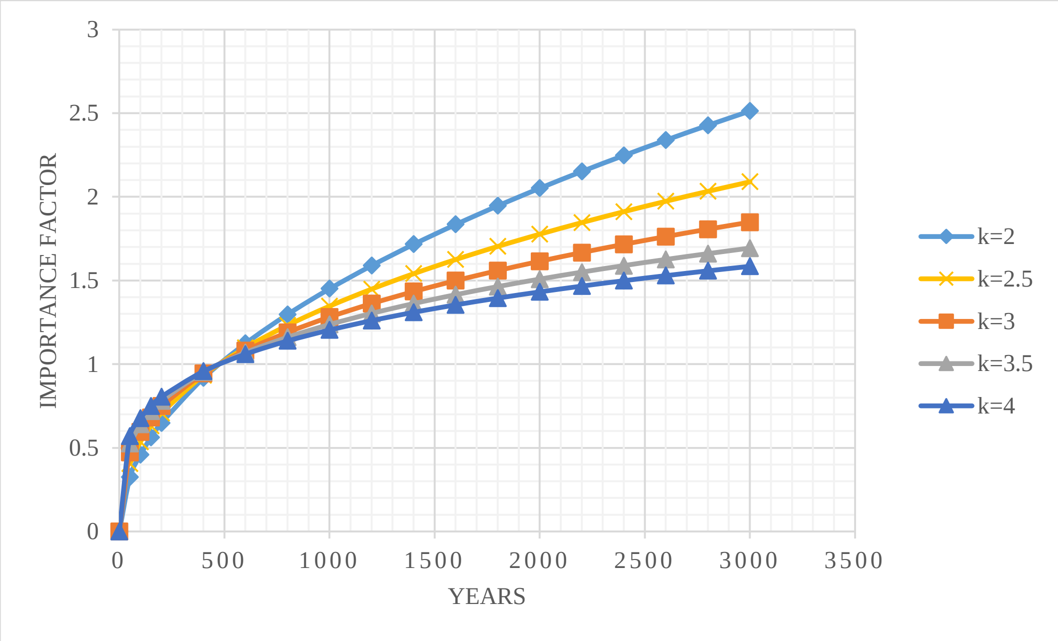


Figure 3.1. Importance factor and return period for different exponent values "k"

Here the exponent 'k' depends on seismicity and generally order of 3 [3]. Important factor values obtained from Figure 3.1 according to the required return period is given in Table 3.3.

Table 3.3. Proposed importance factors for different important classes for Sri Lanka

Importance Class	Return period	Importance factor (k=3)
I	100	0.6
II	475	1.0
III	2500	1.75

3.5 Proposed elastic response spectra for Sri Lanka

For Sri Lanka, there is an elastic response spectrum for rock or hard soils, which is proposed by Seneviratne H.N. et al. [16]. However, for other soil types, there is no elastic response spectrum proposed.

To propose an elastic response spectrum, there should be a soil classification for Sri Lanka according to different soil types. When considering soil types provided in Eurocode 8 and AS 1170.4, there are more detailed soil types starting from type A to type E according to SPT N values. However, the soil Classes provided in IS 1893-1 are more suitable for Sri Lanka since soil and rock types are considered more similar when both countries lie on the same tectonic plate. Proposed soil types and SPT N values are given in Table 3.4.

Table 3.4. Proposed soil types of the elastic response spectrum for Sri Lanka (As IS 1893-1:2002)

Soil Type	SPT N Value
Rock or Hard Soil	N>30
Medium Soil	30>N>10
Soft Soil	N<10

The next step is to develop expressions that define elastic response spectra for different soil types. Available study and proposed expression for the elastic response spectrum for Sri Lanka (Proposed by Senevirathna H.N. et al. [16]) will be used to represent rock or hard soils.

However, there is no proper study and there are no expressions for the other two soil types. Therefore, expressions given in IS 1893-1 are recommended for medium soil and soft soil.

For Rock or Hard Soils (As proposed by Senevirathna H.N. et al [16])

$$SA = \begin{cases} (1+10T)*PGA & 0.00 \le T \le 0.15 \\ 2.5*PGA & 0.15 \le T \le 0.5 \\ 1.25/T*PGA & 0.5 \le T \le 4.0 \end{cases} \quad \ldots\ldots\ldots\ldots\ldots (3.4)$$

For medium soil sites (As recommended in IS 1893-1)

$$Sa/g = \begin{cases} 1+15T & 0.00 \le T \le 0.1 \\ 2.5 & 0.1 \le T \le 0.55 \\ 1.36/T & 0.55 \le T \le 4.0 \end{cases} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.5)$$

For soft soil sites (As given in IS 1893-1)

$$Sa/g = \begin{cases} 1+15T & 0.00 \le T \le 0.1 \\ 2.5 & 0.1 \le T \le 0.67 \\ 1.67/T & 0.67 \le T \le 4.0 \end{cases} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.6)$$

Where S_a/SA is spectral acceleration, g is acceleration due to gravity and T is period. However, Eurocode 8 provides shape of response spectra as shown in Figure 3.2 which incorporates soil types and other parameters.

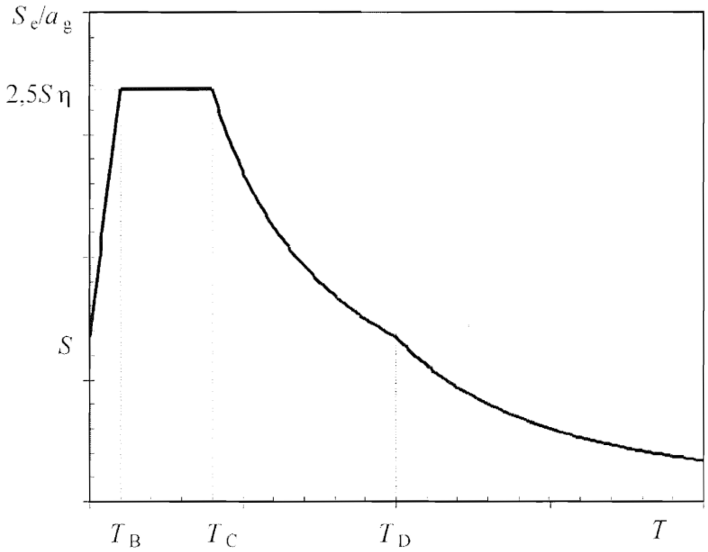


Figure 3.2. Shape of elastic response spectrum defined in EC8. [3]

Therefore, above equations will be modified to suit EC 8 format as follows.

For rock or hard soils

$$SA = \begin{cases} (1+10T)*PGA & 0.00 \le T \le 0.15 \\ 2.5*PGA & 0.15 \le T \le 0.5 \\ S/T*PGA & 0.5 \le T \le 4.0 \end{cases} \quad \ldots\ldots\ldots\ldots (3.7)$$

For medium soils,

$$Sa/g = \begin{cases} 1+15T & 0.00 \le T \le 0.1 \\ 2.5 & 0.1 \le T \le 0.55 \\ S/T & 0.55 \le T \le 4.0 \end{cases} \quad \ldots\ldots\ldots\ldots (3.8)$$

For soft soils

$$Sa/g = \begin{cases} 1+15T & 0.00 \le T \le 0.1 \\ 2.5 & 0.1 \le T \le 0.67 \\ S/T & 0.67 \le T \le 4.0 \end{cases} \quad \ldots\ldots\ldots\ldots (3.9)$$

Where S_a/SA is spectral acceleration, g is acceleration due to gravity and T is period.

Table 3.5. Soil types and parameters arranged to suit the format in EC8

Ground Type	S	T_B(s)	T_C(s)	T_D(s)
Rock/Hard Soil	1.25	0.15	0.5	4.0
Medium Soil	1.36	0.1	0.55	4.0
Soft Soil	1.67	0.1	0.67	4.0

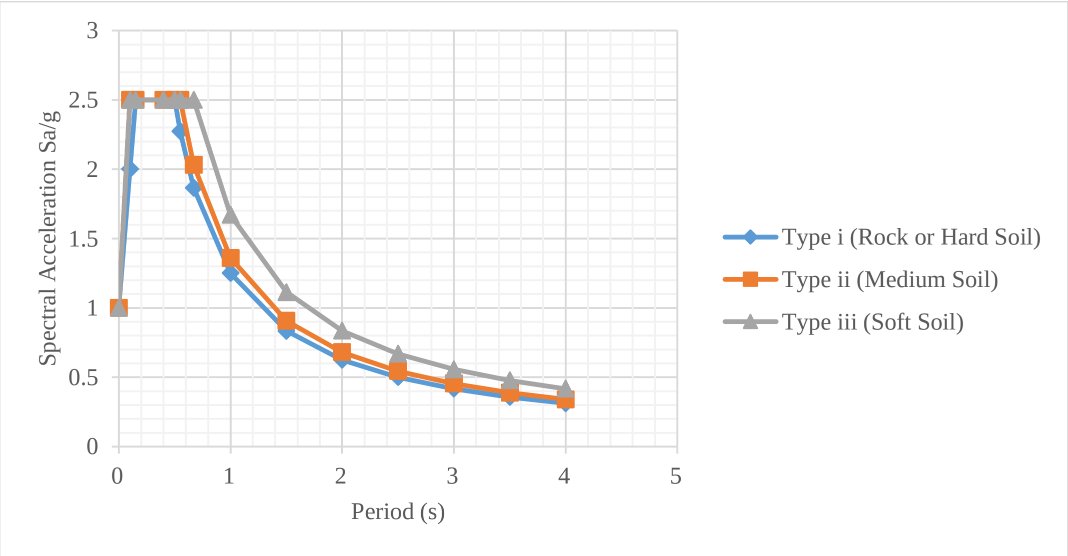


Figure 3.3. Proposed response spectrum for different soil types in Sri Lanka

Proposed response spectrums for different soil types are shown in Figure 3.3 and soil types and parameters arranged to suit the format in Eurocode 8 are shown in Table 3.5.

3.6 Site Dependent design spectrum for linear analysis

According to clause 3.2.4 of EN: 1998-2, ‘both ductile and limited ductile structures shall be designed by performing a linear analysis using a reduced response spectrum’ [5].

This is performed using a reduced response spectrum which is reduced with respect to its elastic response spectrum. This reduced response spectrum is called a design response spectrum. The method of obtaining a design spectrum from an elastic response spectrum is specified in EN: 1998-1-2004 clause 3.2.2.5[3].

Equation No. 3.13- 3.16 which is used to obtain design response spectrum in EC-8 part 1 is as follows [3].

$0<T<T_B$: $S_d(T) = a_g.S.[2/3+T/T_B.(2,5/q-2/3]$ ………………………………. (3.10)

$T_B<T<T_C$: $S_d(T) = a_g.S.2,5/q$ ……………………………………………….. (3.11)

$T_C<T<T_D$: $S_d(T) = a_g.S.(2,5/q).[T_C/T]$ ……………………………………… (3.12)

$T_D<T<4$: $S_d(T) = a_g.S.(2,5/q).[T_CT_D/T^2]$ ……………………………………. (3.13)

Where $S_d(T)$ is the design response spectrum, a_g is the design ground acceleration on Type A ground, S is soil factor, T is the vibration period of a linear single degree of freedom system, T_B is the lower limit of the period of the constant spectral acceleration branch, T_C is the upper limit of the period of the constant spectral acceleration branch, T_D is the value defining the beginning of the constant spectral acceleration branch and q is the behaviour factor value which can be selected from table 4.1 of EN:1998-2-2005[5] according the seismic behaviour of the bridge which is ductile or limited ductile.

3.6.1 Ductile behaviour

When designing bridges to ductile behaviour that is for providing methods to dissipate a reasonable amount of input energy during the seismic event of moderate or severe type. This can be done by the formation of intended plastic hinges or by using isolation devices. To be practicable, locations of these plastic hinges should be accessible to do inspection and repair works. It is preferable to use ductile behavior in bridge designs for locations of moderate to high seismicity.

3.6.2 Limited ductile behavior

In limited ductile behavior, structures have some hysteretic energy dissipation while deviation from the ideal elastic behavior. This kind of behavior corresponds to a value of behavior factor ≤1.5. Limited ductile behavior is used for the regions which have low seismicity. Behavior factor values of Eurocode 8-part 2 are given in Table 3.6.

Table 3.6. Maximum values of the behavior factor q (EC8-2). [5]

Type of Ductile Members	Seismic Behavior	
	Limited Ductile	Ductile
Reinforced Concrete piers:		
Vertical Piers in Bending	1,5	$3.5\lambda(\alpha_s)$
Inclined Struts in Bending	1.2	$2.1\lambda(\alpha_s)$
Steel Piers:		
Vertical Piers in Bending	1.5	3.5
Inclined Struts in Bending	1.2	2.0
Piers with normal Bracing	1.5	2.5
Piers with Eccentric Bracing	-	3.5
Abutment Rigidly connected to the deck		
In General	1.5	1.5
Locked-in structures	1.0	1.0
Arches	1.2	2.0

$\alpha_s = L_s/h$ is the shear span ratio of the pier. Where, L_s is the distance from the plastic hinge to the point of zero moment and h is the depth of the cross section in the direction of flexure of the plastic hinge [5].

For $\alpha_s \geq 3$ $\lambda(\alpha_s) = 1.0$

$3 > \alpha_s \geq 1$ $\lambda(\alpha_s) = \sqrt{\alpha_{s/3}}$

Equations of the response spectrum for different soil types in Sri Lanka is modified to obtain the design response spectrum as given below

For rock or hard soils

$$SA = \begin{cases} (1+10T)*PGA/q & 0.00 \leq T \leq 0.15 \\ 2.5*PGA/q & 0.15 \leq T \leq 0.5 \\ S/T*PGA/q & 0.5 \leq T \leq 4.0 \end{cases} \quad \ldots\ldots\ldots\ldots\ldots (3.14)$$

For medium soils

$$Sa/g = \begin{cases} (1+15T)/q & 0.00 \leq T \leq 0.1 \\ 2.5/q & 0.1 \leq T \leq 0.55 \\ S/Tq & 0.55 \leq T \leq 4.0 \end{cases} \quad \ldots\ldots\ldots\ldots\ldots (3.15)$$

For soft soils

$$Sa/g = \begin{cases} (1+15T)/q & 0.00 \leq T \leq 0.1 \\ 2.5/q & 0.1 \leq T \leq 0.67 \\ S/Tq & 0.67 \leq T \leq 4.0 \end{cases} \quad \ldots\ldots\ldots\ldots\ldots (3.16)$$

Where S_a and SA are spectral acceleration, g is acceleration due to gravity, T is period and q is behavior factor.

Assume a condition of reinforced concrete vertical pier in bending and seismic behavior is ductile. When $\lambda(\alpha_s)$ =1.0, behavior factor q= 3.5 according to table 4.1 of Eurocode 8-part 2. Using this value, design response spectrum for different soil are obtained as given in Figure 3.4, Figure 3.5 and Figure 3.6.

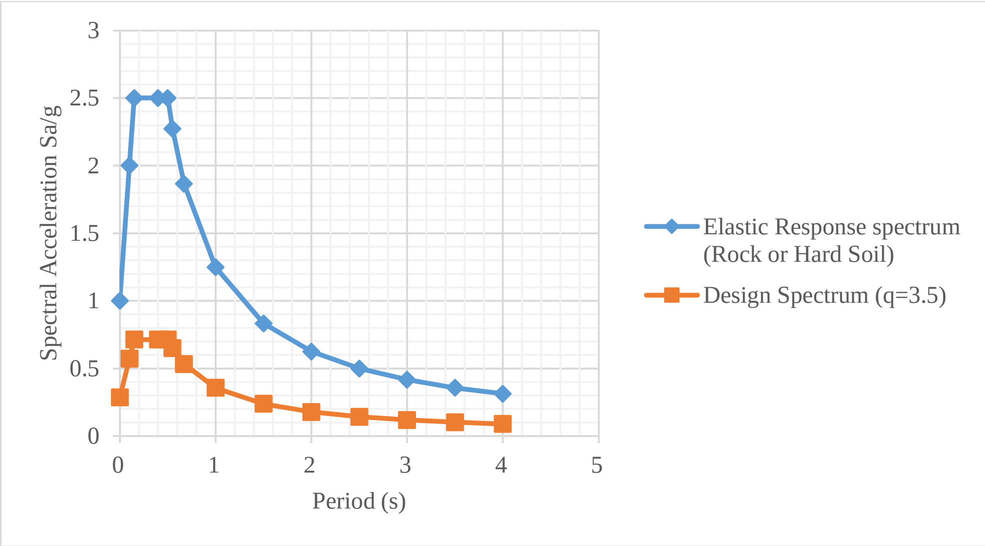


Figure 3.4. Elastic and design response spectrum for rock or hard soil types in Sri Lanka

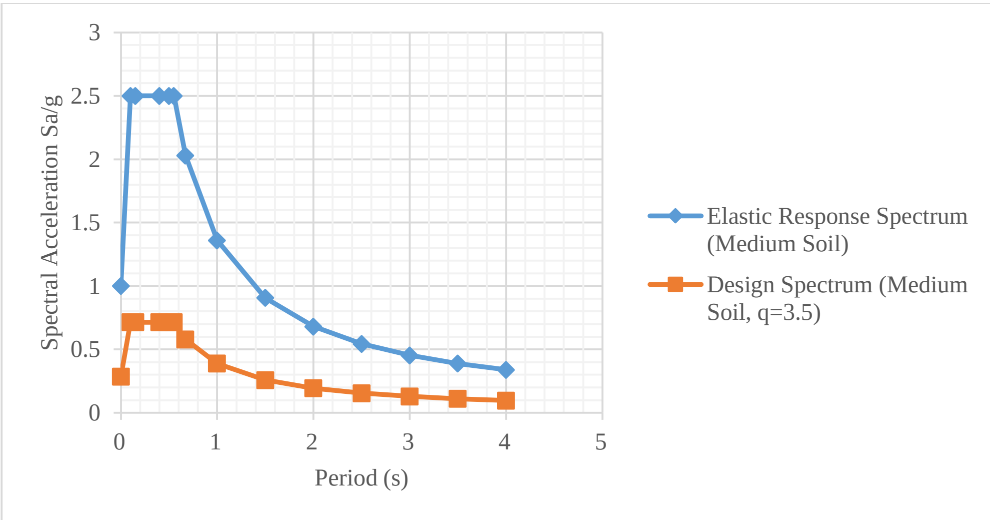


Figure 3.5. Elastic and design response spectrum for medium soil types in Sri Lanka

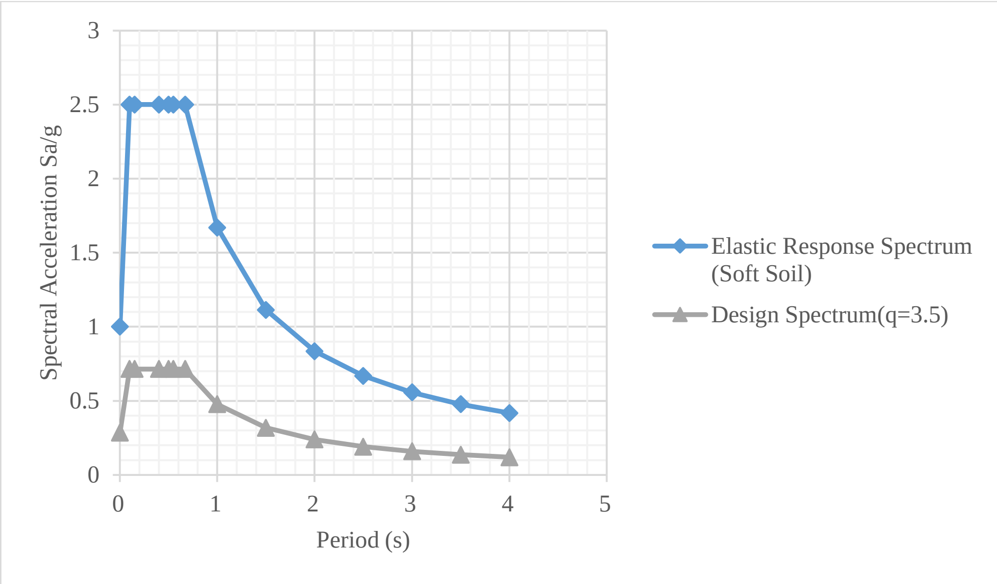


Figure 3.6. Elastic and design response spectrum for soft soil types in Sri Lanka

3.7 Use of AS 5100-2-2004 for seismic analysis

Although Eurocode 8 part2 is proposed to use in Sri Lanka, designers might require to use alternative design code due to complex nature of Eurocode. For those designers, it is a good alternative to use AS 5100.2-2004 with AS 1170-4 for the seismic design of bridges in Sri Lanka.

3.7.1 Bridge Classification

Bridge classification used in AS 5100.2-2004 [6] will be replaced with the proposed bridge classification for important classes for Sri Lanka.

3.7.2 Acceleration coefficient (a) and Site Factors (s)

Acceleration coefficient values will be used as given in the earthquake hazard zonation map for Sri Lanka or acceleration values specified for different areas of the country by Dananjaya R.M.S. et al. [15] Site factors (s) are not specified in AS-5100-2. Site factors are specified in AS 1170-4-1993[21]. However, site factor values are not specified in the latest edition of AS 1170-4-2007. Therefore, Table 2.4(a) given in AS 1170-4-1993[21] is extracted and shown in Table 3.7.

Table 3.7. Site factors (s) for general structures as given in AS 1170-4-1993. [21]

Soil Profile	Site Factor (S)
A Profile of rock materials with rock strength Class L (low) or better	0.67
A Soil profile with either rock materials Class EL(Extreme Low or Very Low(very low) characterized by shear wave velocities greater than 760 m/s Or Not more than 30m of Medium dense to very dense coarse sands & gravels , firm stiff or hard clays or controlled fill	1.0
A Soil profile with more than 30m of medium dense to very dense coarse sands & gravels , firm stiff or hard clays or controlled fill	1.25
A Soil profile with a total depth of 20m or more & containing 6 to 12m of very soft to soft clays , very loose or loose sands , silts or uncontrolled fill	1.5
A Soil profile with more than 12m of very soft to soft clays , very loose or loose sands silts or uncontrolled fill characterized by shear wave velocities less than 150 m/s	2.0

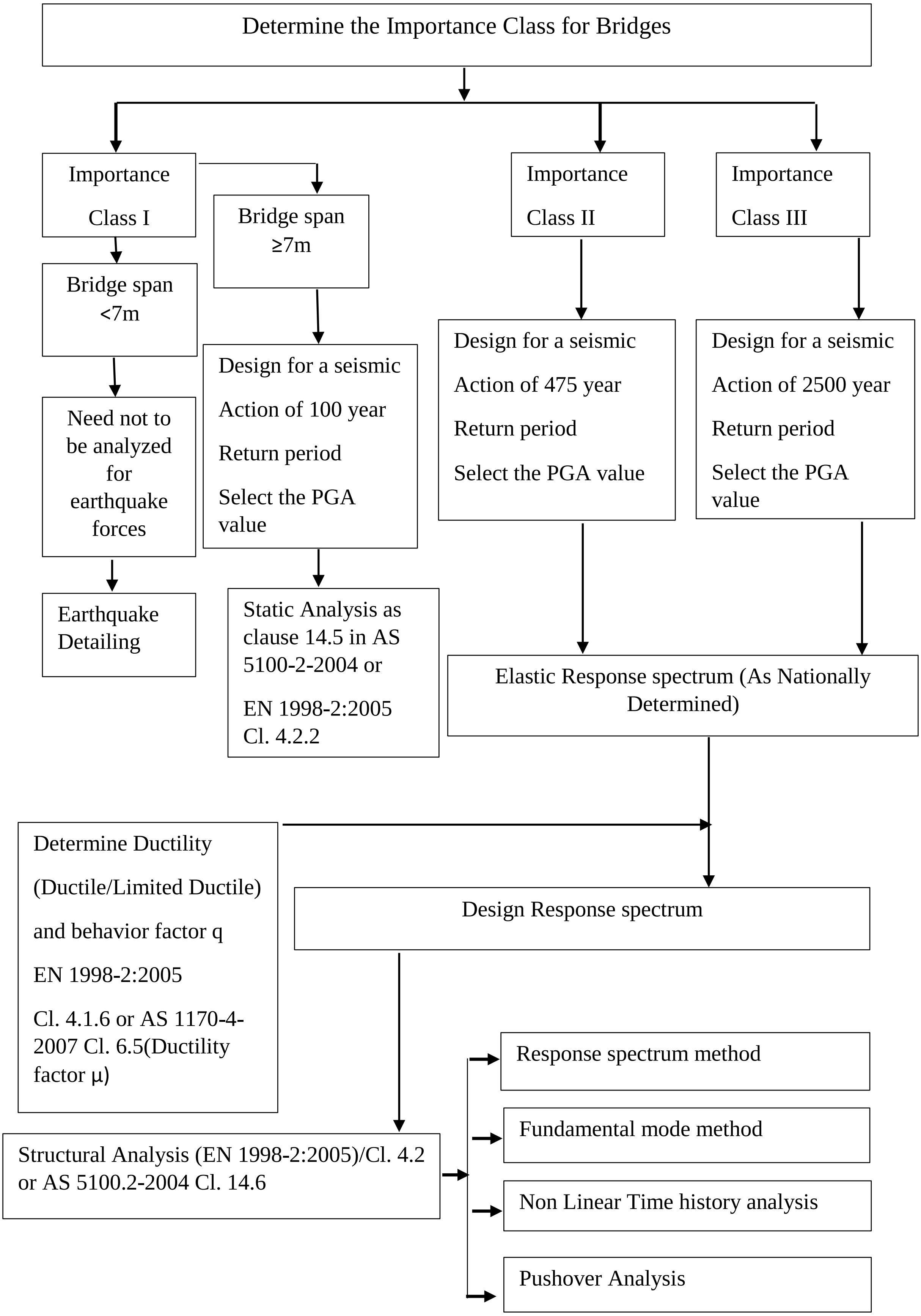
3.7.3 Bridge Earthquake Design Category

The method of analysis of earthquake effect and additional requirement specified in clause 14.4 of AS 5100-2-2004 [6] is depending on the bridge earthquake design category (BEDC). BEDC-1 given in AS 5100-2 specified that when bridge span less than or equal to 20m, need not to be analyzed for earthquake forces.

However, this will be modified to suit Sri Lanka as follows. “Bridge structures in BEDC-1, Where the bridge span is lesser than 7m, need not be analyzed for earthquake forces”.

The reason to select a 7m span for this category for Sri Lanka is that in C or D class roads, 7m is the minimum available beam length which use for single span bridge. Using 7m length pretension beams, two span bridges can be easily constructed with the inclusion of a pier. Earthquake effect to the pier cannot be omitted and therefore only BEDC-1 modified and other BEDC categories use as stated in the design code.

3.8 Proposed seismic analysis approach for bridges in Sri Lanka



3.9 Minimum Overlap Length

Bridge decks are supported on abutments and piers through a horizontally movable device such as a bearing. However, it should be prevented from dropping off during a seismic event. This unseating of the superstructure is an issue during earthquakes and that can be easily controlled. To prevent unseating of the superstructure, a minimum horizontal overlapping length of the underdeck and the top of the supporting abutment or pier must be provided. This is a very important improvement that should be incorporated into new bridge designs of Sri Lanka. Since we have suggested to use EC8-Part2 [5] for seismic analysis of bridges in Sri Lanka, guidelines and equations given in EC8-2[5] suggested to use in estimating minimum overlap length l_{ov}

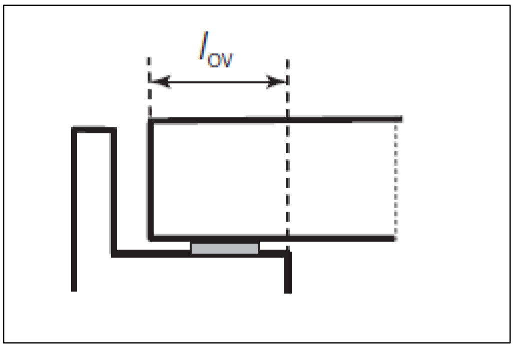


Figure 3.7. Minimum overlap length for seating of the deck on abutment

l_{ov} $= l_m + d_{eg} + d_{es}$ ……………………………………………….(3.17)

d_{eg} $= \varepsilon_e L_{eff} \leq 2d_g$ ………………………………………………….(3.18)

ε_e $= 2d_g/L_g$ …………………………………………………….. (3.19)

l_m = Minimum support length ensuring the safe transmission of the vertical reaction, but no less than 400mm

d_{eg} is the effective displacement of the two parts due to spatial variations of the seismic ground displacement.

d_g is the design ground displacement in accordance with EC8-1 Cl. 3.2.2.4

$= 0,025.a_g.S.T_C.T_D$

L_g is the distance beyond which the ground motions may be considered as Completely uncorrelated.

Eurocode 8 specified ground types, Type A, C and D are approximately similar to Soil types in Sri Lanka which are rock/hard soil, medium soil and soft soil. Therefore, EC8-2 table 3.1N modified to suit Sri Lanka.

Table 3.8 Distance beyond which ground motions may be considered uncorrelated

Ground Type	Rock/Hard Soil	Medium Soil	Soft Soil
L_g(m)	600	400	300

L_{eff} is the effective length of deck, taken as the distance from the deck joint in Question to the nearest full connection of the deck to the substructure.

d_{es} is the effective seismic displacement of the support due to the deformation of the structures

d_{es} = d_{Ed} for decks connected monolithically or through fixed bearing

d_{Ed} = $d_E + d_G + \Psi_2 d_T$

d_E is the design seismic displacement in accordance with 2.3.6.1. EC8-2

d_G Long term displacement due to permanent and quasi-permanent actions

d_T is the displacement due to thermal movements

4.0 CASE STUDIES

4.1 Case study 1-Bridge in importance class I

Consider a bridge in Mudduwa Batugedara road (see Figure 4.1) in Rathnapura. This road is a municipal road in Rathnapura town proximity. However, this road is not a national highway and it does not have critical or significant importance for earthquake recovery. Therefore, the bridge has minor importance according to the proposed bridge classification into importance classes for Sri Lanka as given in Table 3.1.

Figure 4.1.Bridge in Mudduwa Batugedara road in Rathnapura

This bridge is a two span bridge across "We gaga", with equal spans of 15.5m and a total length of the bridge is 31m. The deck has composite slab. The piers consist of two cylindrical columns with diameter of D= 1.5m. The bridge is simply supported on abutments and the deck slab is continuous. Sample calculations carried out using the static method of analysis for three different design codes are given below.

Reference	Description	Output
Table 3.1	Importance class of the bridge = Class I	
Table 3.2	Return period for Importance class 1 =100 year	
	Importance Factor =0.6	
	PGA Value for Rathnapura = 0.041	
	PGA value for Rathnapura = $a_{gR}.\Upsilon_1$	
	$a_g = a_{gR}.\Upsilon_1$	
	= 0.041 x 0.6	
	= 0.0246g	
	Total mass = M	
	M= Mass of deck +Mass of upper half of piers	
	Mass of upper half of piers=((10.4x2x1.5x25)+ (πx1.5^2)/4x8.35x25x2)	
	= 1517.4 kN	
	G_g=(25x((9.8x31x0.75)-(3.14x(0.225^2/4) x31x14))	
	= 5265 kN	
	Mass of Deck = 5265 kN	
	M = 5265+1517	
	= 6782 kN	
	K = $3EI/L^3$	
	I = $\Pi d^4/64$	
	I = 3.14 x1.5^4/64	
	= 0.248	
	EI = 33x10^{9}x0.248	
	= 8.19x10^9	

	$K = (3x8.18x10^9/16.7^3)x2$ $= 10.54x10^6$ $T = 2\pi(M/K)^{1/2}$ $=2\pi\ (6782/9.81)/10.54x10^3)^{1/2}$ $= 1.61$ s Analysis using AS 5100-2	
AS 1170-4:1993 Table 2.4a	Soil profile below founding level = Rock Site Factor (S) = 0.67 Acceleration coefficient a = 0.041	
AS 5100.2-2004, Table 14.3.1	a x S = 0.041*0.67 = 0.027 aS= 0.027< 0.1 and Bridge Classification is type I Bridge Earthquake design category = BEDC-1	
AS 5100.2-2004, Cl. 14.5.2	Static analysis performed as per clause 14.5 AS 5100-2-2004. $H_u = I\ [CS/R_f]G_g$ T = 1.61s	
AS 5100.2-2004, Cl. 14.5.4	$C = 1.25a/T^{2/3}$ $= 1.25*0.041/1.61^{2/3}$ = 0.037	
AS 5100.2-2004, Table 14.5.5	S = 0.67 R_f =2.0(Cl 14.7.1 AS5100.2.2004 H_u = 0.6x (0.037x0.67/2) x6782 = 50kN	Base shear 50kN

EN 1998-2:2005 Eqn. 4.12	Analysis using Eurocode 8-Part 2 Horizontal equivalent static force, F = $MS_d(T)$ Total mass = M M = 6782 kN T = 1.61 s For 0.5< T<4 S = (1.25/Tq)xPGA = ((1.25)/1.61x3.5)x 0.024x9.81 = 0.052 $S_d(T)$ = 0.052 F = 0.052 x 6782/9.81 = 36kN	Base shear 36kN
IS 1893-3:2004, Cl.9.2 IS 1893-3:2004, Cl.9.4.1	Case study as per IS 1893-3-2014 The seismic force resisted by Bridge components F = A_hW A_h = Z/2. I/R .S_a/g Zone factor = 0.1(For Zone II) Importance Factor = 0.6 (For Class I) Response reduction factor R= 2.5 (Table 3 IS 1893-3) T = 1.61 Sa/g for Rock = 1/Tn = 1/1.61 = 0.621 A_h = (0.1/2)x(0.6/2.5)x0.621 = 0.0075 W = 6782 F = 0.0075x6782 = 51kN	Base shear 51kN

Table 4.1. Base shear values calculated- Case study 1

	AS 5100-2	EC8-2	IS 1893-3
Base Shear (kN)	50kN	36kN	51kN

Base shear values given in Table 4.1 are results of static lateral force method analysis by using different codes. However, the base shear values of the pier are different for each code. AS 5100-2 code and IS 1893-3 gives similar result and EC8-2 gives lower result.

4.2 Case Study 2- Bridge in Importance Class II

Consider bridge no 101/6 in Colombo-Rathnapura-Wellawaya Batticaloa road (see Figure 4.2). This road is in A Class road in Rathnapura town. Since this road is a national highway, this bridge has significant importance for earthquake recovery. Therefore, the bridge has significant importance according to the proposed bridge classification into important classes for Sri Lanka according to Table 3.1.

Figure 4.2. Bridge No. 101/6 Colombo-Rathnapura-Wellawaya road

This bridge is a four span bridge, with four equal spans of 34.5m and bridge has total length of 138m. The deck consists of post tensioned beams, with 6 beams per span, and concrete deck. Piers consist of two cylindrical columns with diameter of D= 1.5m. According to Table 3.1 of this document, the importance class selected is class II. Sample calculation carried out according to the fundamental mode method is given below.

Reference	Description	Output
Table 3.1	Bridge classified in importance Class II	
	Return period for Importance class II =475 year	
Table 3.2	Importance Factor = 1	
	PGA Value for Rathnapura = 0.041	
	PGA value for Rathnapura = $a_{gR}.\Upsilon_1$	
	a_g = $a_{gR}.\Upsilon_1$	
	= 0.041 x 1	
	= 0.041g	
EN 1998-2:2005 Eqn. 4.12	Analysis according to EC8-2:2005	
	Horizontal equivalent static force, F = $MS_d(T)$	
	Total mass = M	
	M= Mass of the deck +Mass of the upper half of piers	
	Weight of Beams	
	Area of Beam = 0.881 m^2	
	No of beams in one span = 6	
	No of Spans in bridge = 4	
	Length of Beam = 34.5 m	
	Total beam weight = 0.881x34.5x4x6x25	
	= 18236.7 kN	
	Weight of Diaphragms	
	Length of mid diaphragms = 8.66m	
	Thickness of mid diaphragms = 0.18m	
	Height of diaphragms = 2.2 m	
	No of mid diaphragms in a span = 5	
	Total weight of diaphragms (4 spans)=	
	8.66x0.18x5x2.2x4x25	
	= 1714.68 kN	

	Weight of the slab between beams Total Width of the deck slab between beams = 2.95m Thickness of the deck slab = 0.14m Total wt. of the deck slab between beams = 2.95x0.14x34.5x4x25 (4span) = 1424.85kN Weight of the Deck Width of the deck = 9.8m Thickness of the deck concrete = 0.1m Total weight of the deck =9.8x0.1x34.5x4x25 = 3381 kN Weight of pier heads = 2.1x1.65x9.8x3x25 = 2546.77 Weight of pier upper half=π x 1.5^2/4(3.6+8.7+8.7) x25 = 927.3 M =18236.7+1714.7+1424.9+3381+2546.8+927.3 = 28231.4 kN Quasi permanent value of traffic load	
EN 1991-2:2003, Table 4.2	Total Carriageway width = 7.4m Notional lane width = 3m For Lane no 01 αq_k = 1x9x3 = 27 kN/m For Lane no 02 αq_k = 1x2.5x3 = 7.5 kN/m Residual Area = 1x2.5x1.4 = 3.5 kN/m Uniformly distributed traffic load of Load Model 1 = 38 kN/m	

EN 1998-2:2005, Cl. 4.1.2	In seismic situation quasi –permanent value of the traffic load is taken as 20% of UDL of Load model 1(LM1) Load due to traffic load = 0.2x38x34.5x4 = 1048.8 Total load applied on the bridge deck M= 28231+1049 = 29280 kN K = $3EI/L^3$ I = $\Pi d^4/64$ I = $3.14 \times 1.5^4/64$ = 0.248 E = 33000Mpa K_1 = $(3x33000x0.248x2)/5.25^3$ = 339.89MN/m K_2 = $(3x33000x0.248x2)/10.35^3$ = 44.36MN/m K_3 = $(3x33000x0.248x2)/10.35^3$ = 44.36MN/m K = K1+K2+K3 = 428.6 MN/m T = $2\pi(M/K)^{1/2}$ = $2\pi\ ((29280/9.81)/428600)^{1/2}$ = 0.524 s For 0.5< T<4 S =1.25/TqxPGA = (1.25/0.524x3.5)x0.041x9.81 = 0.274 $S_d(T)$ = 0.274 F = 0.274x 29280/9.81 = 817.81 kN Total seismic shear force in piers = 818kN	Base shear 818kN

	Shear force in pier 1 =817.81x(340/429) = 648.15kN Shear force in pier 2 and 3 = 817.81x (44/429) = 83.87kN Consider limited ductile behavior Behavior Factor q = 1.5 S =1.25/TqxPGA = (1.25/0.524x1.5)x0.041x9.81 = 0.639 $S_d(T)$ = 0.639 F = 0.639x 29280/9.81 = 1907 kN Case study according to AS 5100-2-2004 Soil profile below founding level = Rock	
AS 1170-4:1993, Table 2.4a	Site Factor (S) = 0.67 Acceleration coefficient a= 0.041 a x S = 0.041*0.67 = 0.0275	
AS 5100.2 2004 Table 14.3.1	aS= 0.027<0.1 & Bridge Classification is type II Bridge Earthquake design category = BEDC-I	
AS 5100.2 :2004 Cl. 14.5	H_u = I $[CS/R_f]G_g$ Fundamental period T = 0.524 C = $1.25a/T^{2/3}$ = $1.25*0.041/0.524^{2/3}$ = 0.079	
AS 5100.2 :2004 Cl 14.7.1	S = 0.67 For simply supported spans R_f =2.0 G_g = 28231 kN	

	H_u = 1x (0.079x0.67/2) x28231 = 747kN Case study as per IS 1893-3-2014 The seismic force resisted by Bridge components F = A_hW A_h = Z/2. I/R .S_a/g Zone factor = 0.1(For Zone II) Importance Factor = 1 (For Class II) Response reduction factor R = 2.5 (Table 3 IS 1893-3) T = 0.524 Sa/g for Rock = 1/Tn = 1/0.524 = 1.91 A_h = (0.1/2)x(1/2.5)x1.91 = 0.0382 W = 28231 F = 0.0382x28231 = 1078kN	Base shear 747kN
IS 1893-3:2014 Cl.9.2 IS 1893-3:2014 Cl.9.4.1		Base shear 1078kN

Table 4.2. Base shear values calculated- Case Study 2

	AS 5100-2	EC8-2	IS 1893-3
Base Shear (kN)	747 kN	818 kN	1078 kN

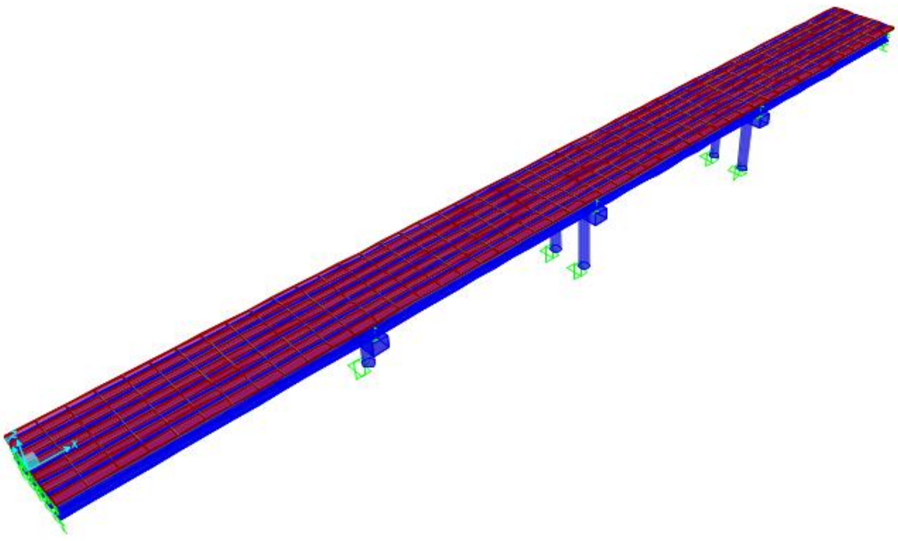


Figure 4.3. Finite element modal of Bridge No. 101/6

A finite element model for bridge no. 101/6 was developed using Csi Bridge 2016 v18.0.1. Here, bridge superstructure modeled using bridge wizard in the program and piers, piles modeled using frame elements. Also, soil modeled using soil springs.

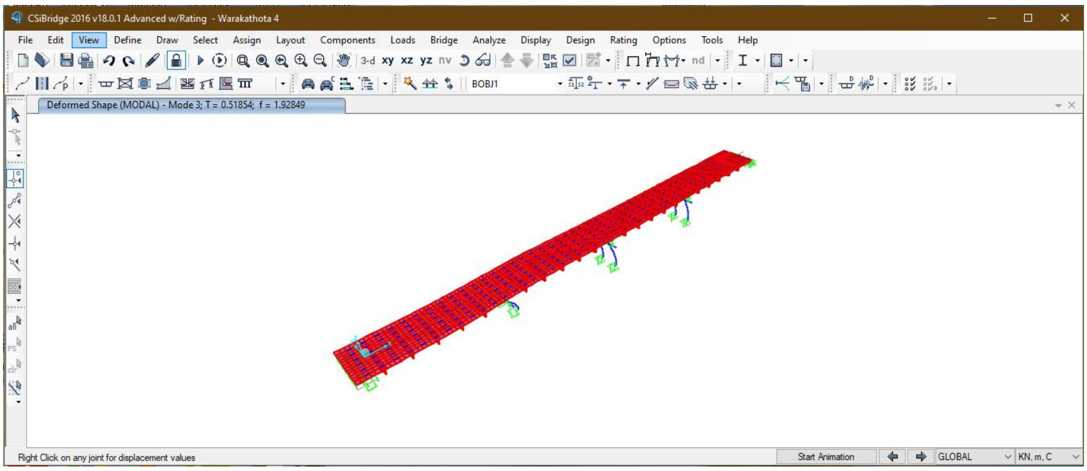


Figure 4.4. Mode No. 03-Highest mass participated mode

Response spectrum analysis was done using the response spectrum defined for Sri Lanka. No of modes used for the analysis was 20 and the most important modes are presented in Table 4.3.

Table 4.3. Most important modes of case study 2

Mode No	Period: s	Modal Mass contribution %		
		X Direction	Y Direction	Z Direction
1	0.965	0	7.05	0
2	0.742	0	71.7	0
3	0.519	97.3	0	0
4	0.442	0	10.8	0
6	0.237	0.1	0	0
7	0.207	0.05	0	9.52
8	0.202	0	0.01	0
13	0.133	0	0.3	0
14	0.131	0	0	57.2
15	0.129	0	5.02	0

Table 4.4. Comparison of fundamental mode and response spectrum analysis

	Fundamental mode analysis	Response spectrum analysis
Fundamental period	0.524s	0.519s
Base Shear(q=3.5) (Ductile behavior)	818kN	769kN
Base Shear(q=1.5) (Limited ductile)	1907 kN	1792 kN

4.3 Case Study 3- Bridge in Importance Class III

Consider a bridge at the Central Expressway project –Package 2, section from Mirigama to Kurunegala at 42+240(see Figure 4.5). This bridge is in an expressway and it is an important bridge for post-earthquake recovery. Therefore, the bridge has critical importance according to the proposed bridge classification into importance classes for Sri Lanka as given in Table 3.1.

Figure 4.5. Mahaoya Bridge at 42+240 in Central Expressway

According to Table 3.1, importance class is III. Sample calculation carried out using earthquake resistant design guidelines and relevant parameters are given below.

Reference	Description	Output
Table 3.1	Bridge classified in importance Class III	
Table 3.2	Return period for Importance class III=2500- year	
	Importance Factor = 1.75	
	PGA Value for Kurunegala = 0.041	
	PGA value for 2500yr return period = $a_{gR}.Y_1$	
	a_g = $a_{gR}.Y_1$	
	= 0.041 x 1.75	
	= 0.0718g	
	Analysis according to EC8:2:2005	
EN 1998-2:2005 Eqn 4.12	Horizontal equivalent static force, F = $MS_d(T)$	
	Total mass = M	
EN 1998-2:2005 Cl. 4.2.2.3	M = Mass of the deck +Mass of the upper half of piers	
	Weight of Beams	
	Volume of Beam = 33.6 m^3	
	No of beams in one span = 5	
	No of Spans in bridge = 6	
	Total beam weight = 33.6x5x2x6x25	
	= 50400 kN	
	Weight of Diaphragms	
	Length of diaphragms LHS side = 10.2 m	
	Length of diaphragms LHS side = 10.5 m	
	Thickness of diaphragms = 0.18m	

	Height of diaphragms = 1.8 m No of mid diaphragms in a span = 4 Total weight of diaphragms (6 spans, LHS and RHS) =10.2x0.18x1.8x4x6x25+ 10.5x0.18x1.8x4x6x25 = 4024.08 kN Weight of slab between beams Width of the deck slab between beams LHS side=5.2 m Width of the deck slab between beams LHS side=5.5 m Thickness of the deck slab between beams = 0.2m Total weight of the deck slab between beams (6 spans) =5.2x0.2x30x6x25+5.5x0.2x30x6x25 = 9630 kN Weight of Deck Width of the deck LHS side = 11.2 m Width of the deck RHS side = 11.5 m Thickness of the deck concrete = 0.1m Total weight of deck =11.2x0.1x30x6x25+11.5x0.1x30x6x25 = 10215 kN Weight of pier heads = (11.2-3) x2.4x1.6x5x25+ (11.5-3) x2.4x1.6x5x25 = 8016 kN	

EN 1991-2:2003, Table 4.2	Weight of pier upper half= πx1.5^2/4(7.45+9.81+10.31+10.45+11.95+7.64+9.78+10.77+10.91+12.4)x25 = 4482.4 kN M =50400+4024+9630+10215+8016+4482 = 86767 kN Quasi permanent value of traffic load Total Carriageway width, LHS = 11.2 m Total Carriageway width, RHS = 11.5 m Consider LHS side of bridge Notional lane width = 3m For Lane, no 01 αq_k = 1x9x3 = 27 kN/m For Lane, no 02 αq_k = 1x2.5x3 = 7.5 kN/m For Lane, No 03 αq_k = 1x2.5x3 = 7.5 kN/m Residual Area = 1x2.5x2.2 = 5.5 kN/m Uniformly distributed traffic load of Load Model 1 = 47.5 kN/m	

EN 1991-2:2003, Table 4.2	Consider RHS side of bridge Notional lane width = 3m For Lane, no 01 αq_k = 1x9x3 = 27 kN/m For Lane, no 02 αq_k = 1x2.5x3 = 7.5 kN/m For Lane, No 03 αq_k = 1x2.5x3 = 7.5 kN/m Residual Area = 1x2.5x2.5 = 6.25 kN/m Uniformly distributed traffic load of Load Model 1 = 48.25 kN/m In seismic situation, traffic load is taken as 20% of UDL of Load model 1(LM1) Load due to traffic load (LHS & RHS Side) = 0.2x47.5x30x6+0.2x48.25x30x6 = 3447 Total load applied on the bridge deck M = 86767+3447 = 90214 kN K = $3EI/L^3$	

	$I = \Pi d^4/64$ $I = 3.14 \times 1.5^4/64$ $= 0.248$ $E = 33000$Mpa Consider Piers at LHS side $K_1 = (3x33000x0.248x2)/7.45^3$ $= 118.95$MN/m $K_2 = (3x33000x0.248x2)/9.82^3$ $= 51.93$MN/m $K_3 = (3x33000x0.248x2)/10.32^3$ $= 44.75$MN/m $K_4 = (3x33000x0.248x2)/10.45^3$ $= 43.09$MN/m $K_5 = (3x33000x0.248x2)/11.95^3$ $= 28.82$MN/m Consider Piers at RHS side $K_6 = (3x33000x0.248x2)/7.63^3$ $= 110.72$MN/m $K_7 = (3x33000x0.248x2)/9.78^3$ $= 52.57$MN/m $K_8 = (3x33000x0.248x2)/10.77^3$ $= 39.37$MN/m $K_9 = (3x33000x0.248x2)/10.91^3$ $= 37.87$MN/m	

	K_{10} = $(3x33000x0.248x2)/12.41^3$	
	= 25.73MN/m	
	K = K1+K2+K3+K4+K5+K6+ K7+K8+K9+K10	
	= 553.8 MN/m	
	T = $2\pi(M/K)^{1/2}$	
	= $2\pi\ ((90214/9.81)/553800)^{1/2}$	
	= 0.809 s	
	For 0.5< T<4 S =1.25/TqxPGA	
	= (1.25/0.809x3.5)x0.0718x9.81	
	= 0.311	
	$S_d(T)$ = 0.311	Base
	F = 0.311x 90214/9.81	Shear
	Total seismic shear force in piers = 2860kN	2860kN
	Consider Limited ductile behavior	
	Behavior factor q = 1.5	
	For 0.5< T<4 S =1.25/TqxPGA	
	= (1.25/0.809x1.5)x0.0718x9.81	
	= 0.726	
	$S_d(T)$ = 0.726	
	F = 0.726x 90214/9.81	
	= 6676 kN	

	Case study according to AS 5100-2-2004	
AS 1170-4:1993 Table 2.4a	Soil profile below founding level = Rock Site Factor (S) = 0.67 Acceleration coefficient a = 0.041 a x S = 0.041*0.67 = 0.027	
AS 5100.2-2004 Table 14.3.1	aS= 0.027<0.1 & Bridge Classification is type III Bridge Earthquake design category = BEDC-2	
AS 5100.2-2004 Cl.14.5	H_u = I $[CS/R_f]G_g$ Fundamental period T = 0.809 C = $1.25a/T^{2/3}$ = $1.25*0.041/0.809^{2/3}$ = 0.059 S = 0.67 For simply supported spans R_f =2.0(Cl 14.7.1 AS5100.2.2004) I = 1.75 G_g = 86767 kN H_u = 1.75x (0.059x0.67/2) x86767 = 3001 kN	Base Shear 3001kN

IS 1893-3:2014 Cl.9.2 IS 1893-3:2014 Cl. 9.4.1	Case study as per IS 1893-3-2014 The seismic force resisted by Bridge components $F = A_h W$ $A_h = Z/2.\ I/R\ .S_a/g$ Zone factor = 0.1(For Zone II) Importance Factor = 1.75 (For Class III) Response reduction factor R = 2.5 (Table 3 IS 1893-3) T = 0.809 Sa/g for Rock = 1/Tn = 1/0.809 = 1.236 A_h = (0.1/2)x(1.75/2.5)x1.236 = 0.0433 W = 86767 F = 0.0433x86767 = 3757 kN	Base Shear 3757kN

Table 4.5. Base shear values calculated- Case Study 3

	AS 5100-2	EC8-2	IS 1893-3
Base Shear (kN	3001 kN	2860 kN	3757 kN

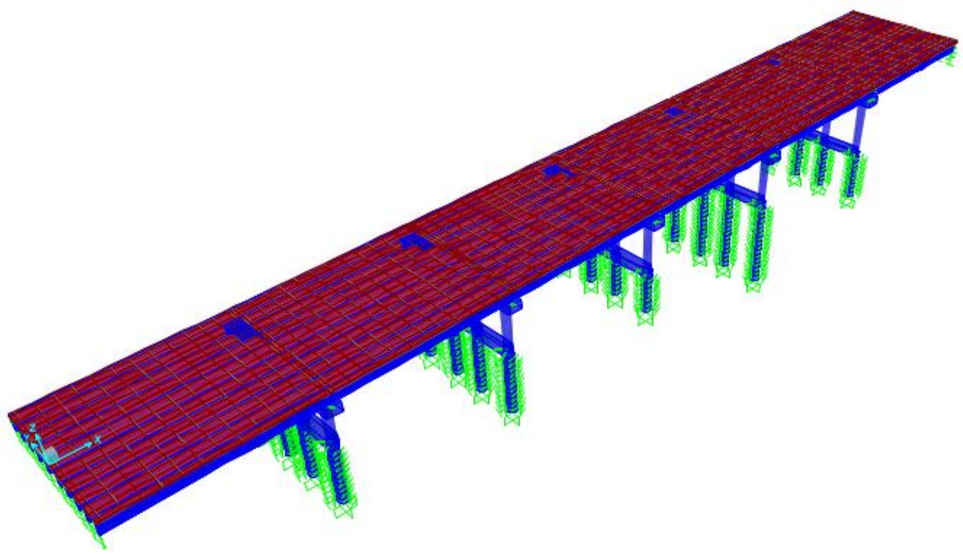


Figure 4.6.Finite element model for Mahaoya Bridge- Case Study 3

A finite element model for bridge at 42+240 in Central expressway was developed using Csi Bridge 2016. Here bridge superstructure modeled using bridge wizard in the program and piers, piles modeled using frame elements. Also soil was modeled using soil springs.

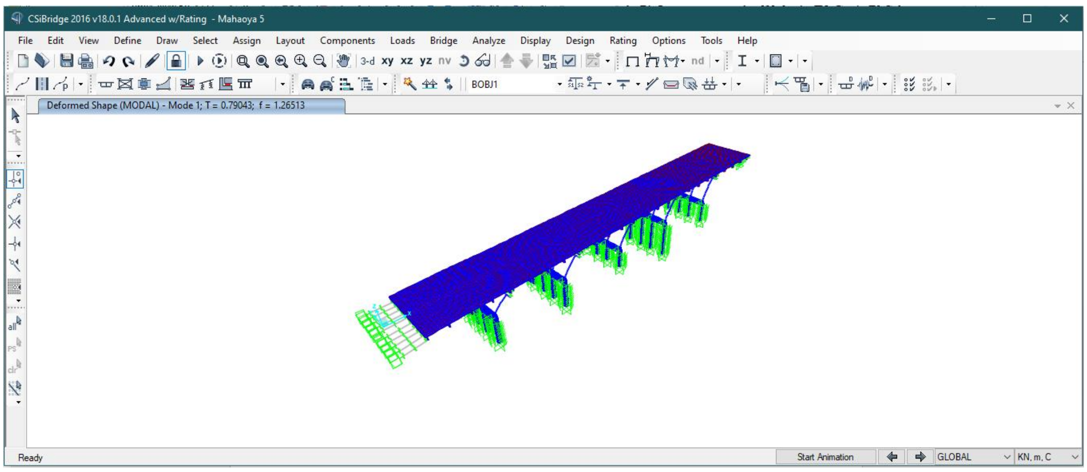


Figure 4.7. Mode No. 01-Highest mass participated mode of case study 3

Response spectrum analysis was done using the response spectrum defined for Sri Lanka. No of modes used for the analysis was 40 and the most important modes are presented in Table 4.6.

Table 4.6. Most important modes-Case Study 3

Mode No	Period: s	Modal Mass contribution %		
		X Direction	Y Direction	Z Direction
1	0.790	82.75	0	0
2	0.575	0	26.84	0
3	0.374	0	50.57	0
4	0.312	0	5.04	0
7	0.206	0	0	2.44
12	0.152	0	0	5.37
16	0.120	0	0	46.43

Table 4.7. Comparison of fundamental mode and response spectrum analysis-Case Study 3

	Fundamental mode analysis	Response spectrum analysis
Fundamental period	0.809s	0.790s
Base Shear(q=3.5) (Ductile behavior)	2860kN	2723kN
Base Shear (q=1.5) (Limited ductile)	6676 kN	6351 kN

5.0 CONCLUSIONS AND RECOMMENDATIONS

Earthquake effects for designs are not often adopted in Sri Lanka according to the belief of the country is free of earthquakes which located away from Indo-Australian tectonic plate boundary. However, the recent seismic events around the country which have considerable magnitudes and possibility of occurring intra-plate type earthquake should be considered when designing bridge structures. Therefore, it is advisable to adopt earthquake resistant design guidelines for bridges in Sri Lanka.

In the proposed earthquake resistant design guidelines for bridges, a seismic analysis approach for bridges is proposed based on Eurocode 8(EN 1998-2:2005). This code provides opportunities for designers to choose national parameters according to local seismic characteristics through a national annex. Also, Australian standards of AS-1170-4 and AS 5100.2-2004 are proposed to use for seismic analysis of bridges according to the choice of designer.

It is proposed to use three importance classes which classify bridges according to their location, span, use and post-earthquake recovery etc. Return periods for the proposed importance classes were determined as 100 year, 475 year and 2500 years and importance factors were determined as 0.6,1 and 1.75 respectively.

Further, it is proposed to use the response spectrum proposed by Senevirathna H.N. et al. [16] for Sri Lanka for rock or hard soil. However, there is no response spectrum available in Sri Lanka for medium soil and soft soil type. Therefore, it is proposed to use response spectrum given in IS 1893-1:2002 for medium and soft soil types.

Peak ground acceleration values proposed by Dhanajaya R.M.S.et al. [15] provided PGA values for 58 cities in the country. Those PGA values are proposed to use for seismic analysis of bridges in Sri Lanka.

According to the three case studies carried out using the fundamental mode method, Eurocode 8(EN 1998-2:2005) ,Indian standard (IS 1893-3:2014) and Australian standard (AS 5100.2-2004) provided different base shear value results.

Also, response spectrum analysis was carried out using Eurocode 8(EN 1998-2:2005) for case study 2 and case study 3 and fundamental period values and base shear values compared with the results obtained from fundamental mode method using the same design code. Results were approximately equal for both analyses.

Results of the case studies have shown the use of proposed earthquake resistant design guidelines for bridges in Sri Lanka and using of Eurocode 8-Part 2 with selected national parameters is more suitable.

It is recommended to use specified minimum overlap length in capping beam of bridges and earthquake detailing rules to minimize damage to bridges due to earthquakes in future.

Also, parameters proposed in this study are recommended to replace whenever more accurate and detailed studies are available in the future.

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APPENDIX -A

Method of bridge model preparation using Csi Bridge 2016

Steps of generating finite element model for bridge

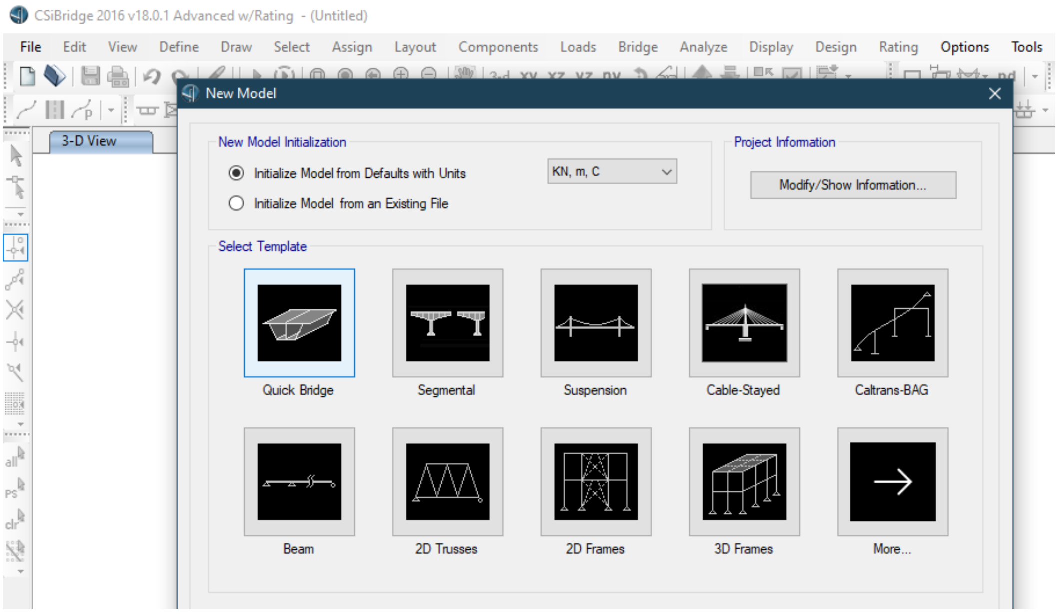


Figure A.1. Quick Bridge option in CSi Bridge

For generation of bridge model, CSI bridge ver.2016 will be opened and there from menu File ⟶ new modal ⟶ quick bridge have to be selected.

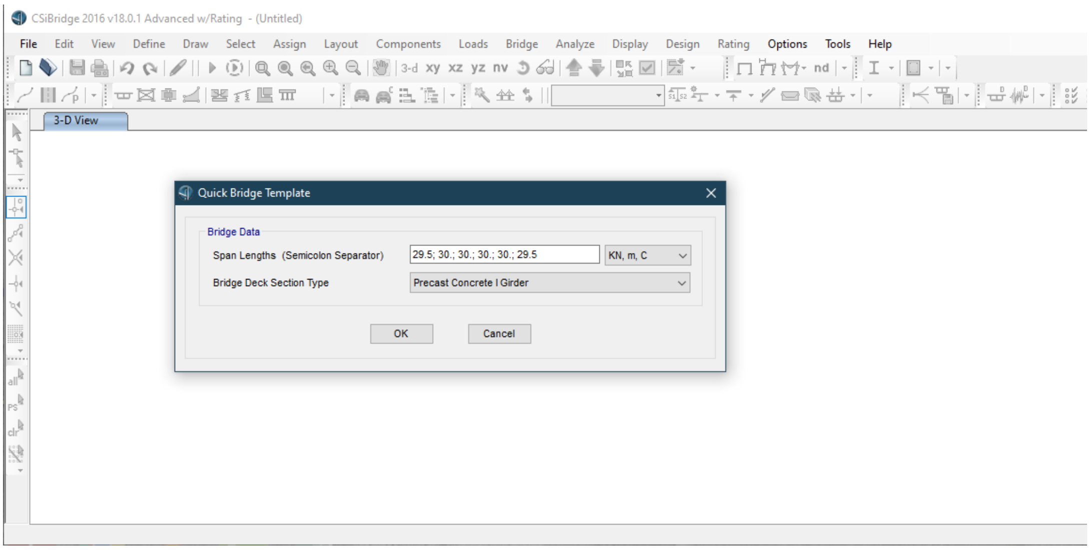


Figure A.2. Span length and deck type definition

Span lengths defined according to the no of spans and deck type selected from the available list under the quick bridge option. After that an initial model will be developed as figure A.3

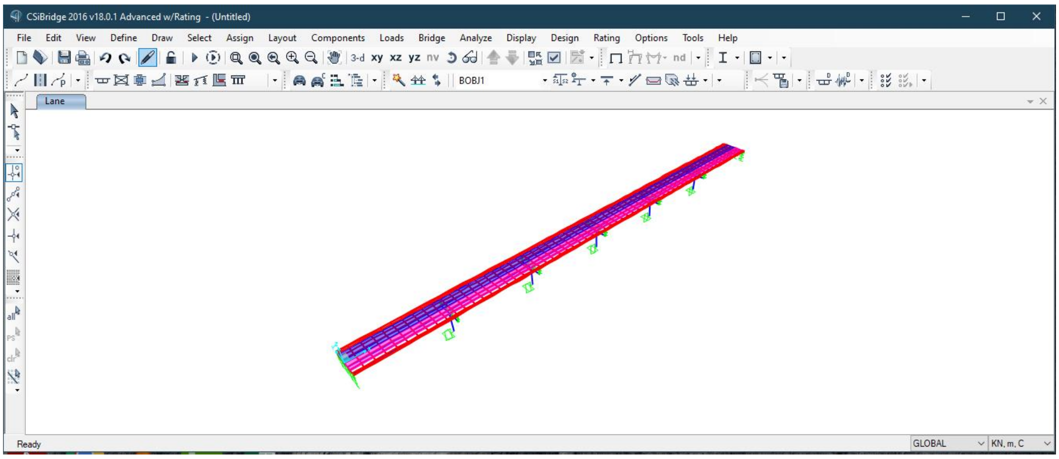


Figure A.3. Initial model generated in CSi Bridge

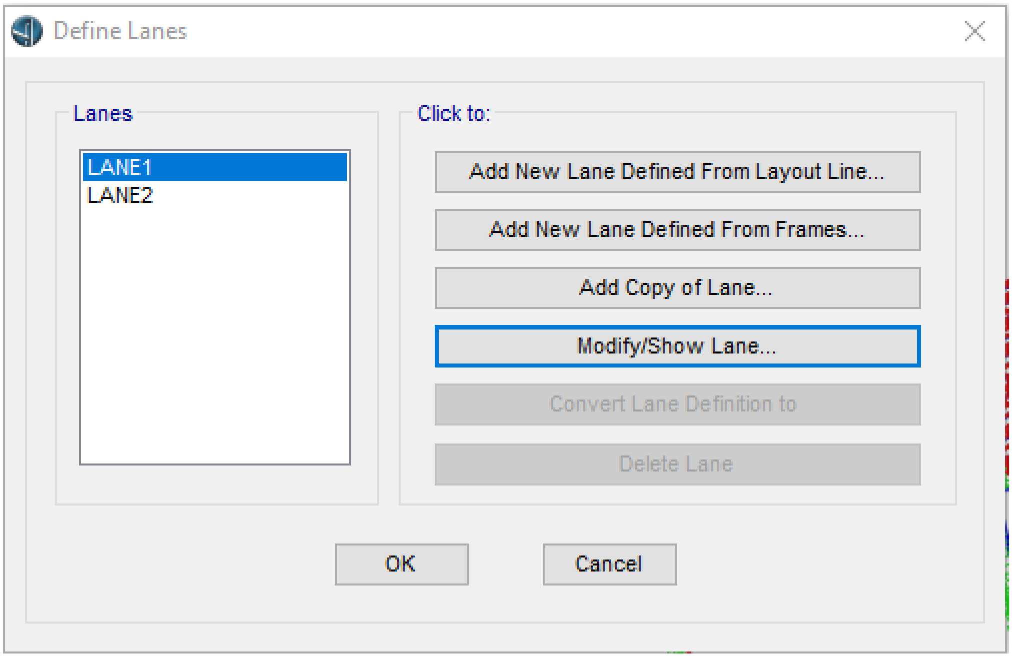


Figure A.4. Define lanes

Once the initial model is generated, lanes are auto generated. However, these lanes need to be modified as required for analyzing the bridge. For that from the Layout tab of main menu Lanes → Define lanes → Modify/ show lanes should be selected

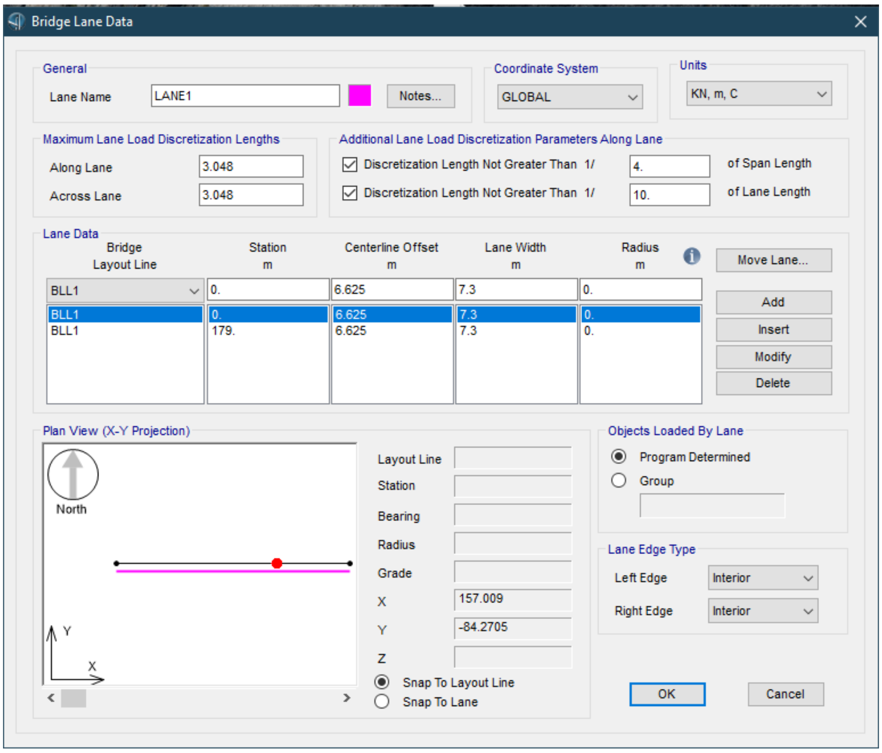


Figure A.5. Modify/show bridge lane data in CSi Bridge

Once the lanes are defined in the initial model, other most important steps are material definition, section definition and bridge components definition (deck, diaphragm, abutment and bearings). All these items are included in the components tab of the main menu in the Csi bridge program

Under material properties, required material will be defined. Thereafter, sections required for analysis such as pier section, piles, bent cap and girder section will be defined. When defining of section properties are completed, the main components of the bridge such as pier and abutment will be defined. For pier, when no of columns and height and connection detail is given, program will auto generate the model.

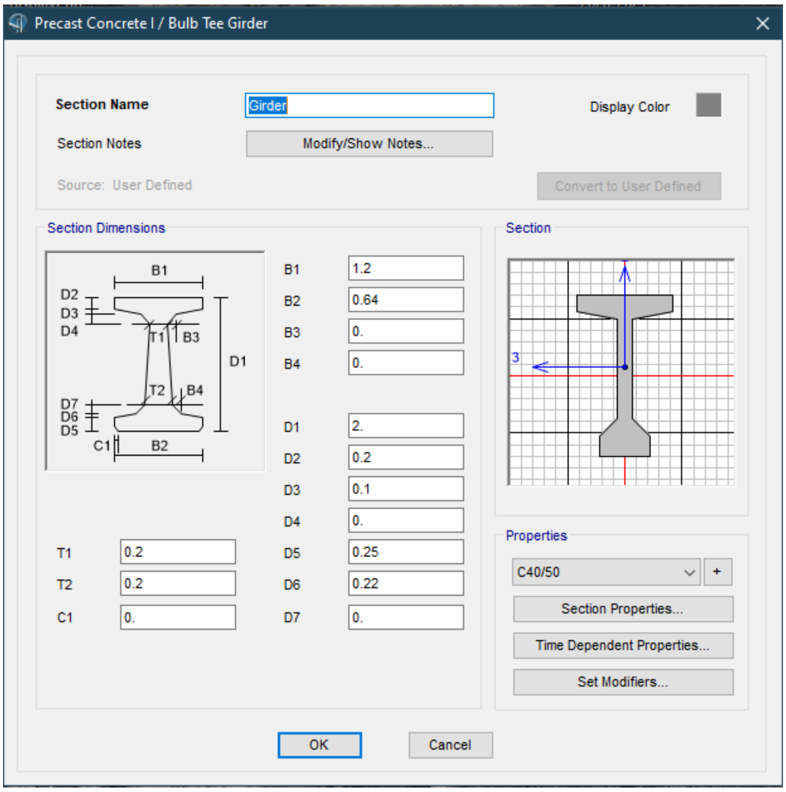


Figure A.6. Section properties-Precast girder

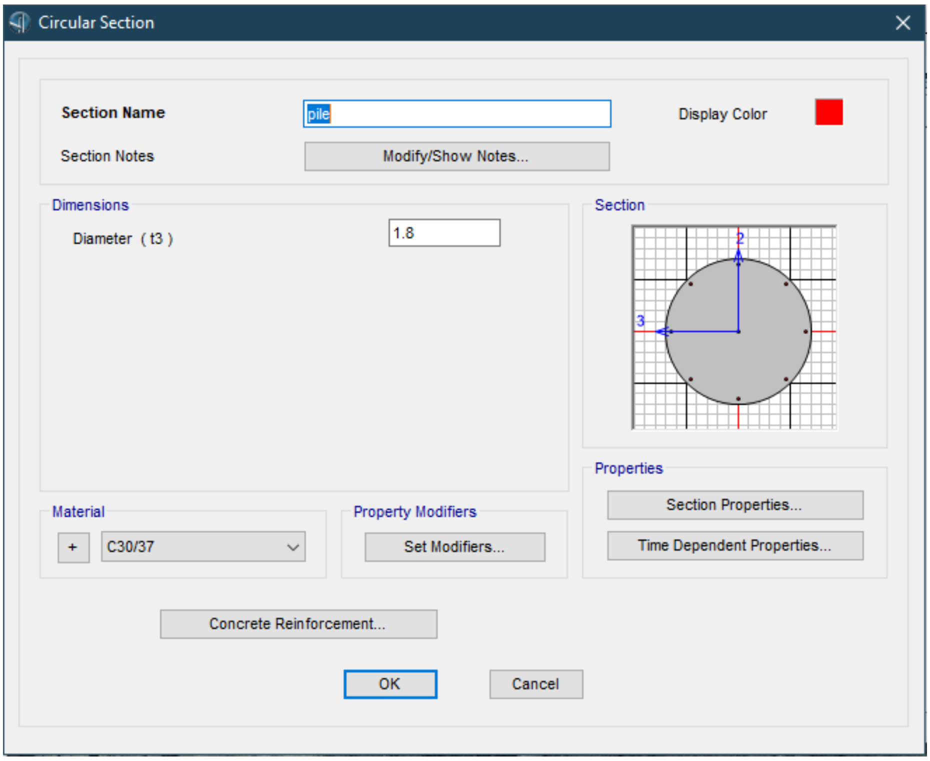


Figure A.7. Section properties-Pile

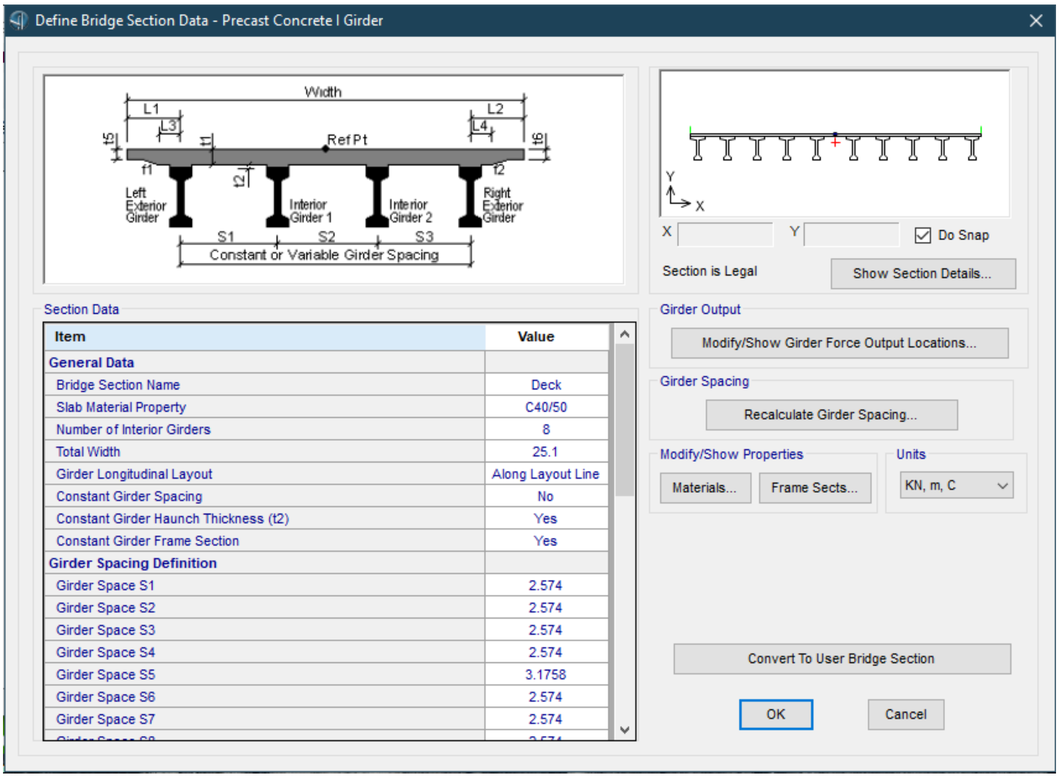


Figure A.8. Bridge section data

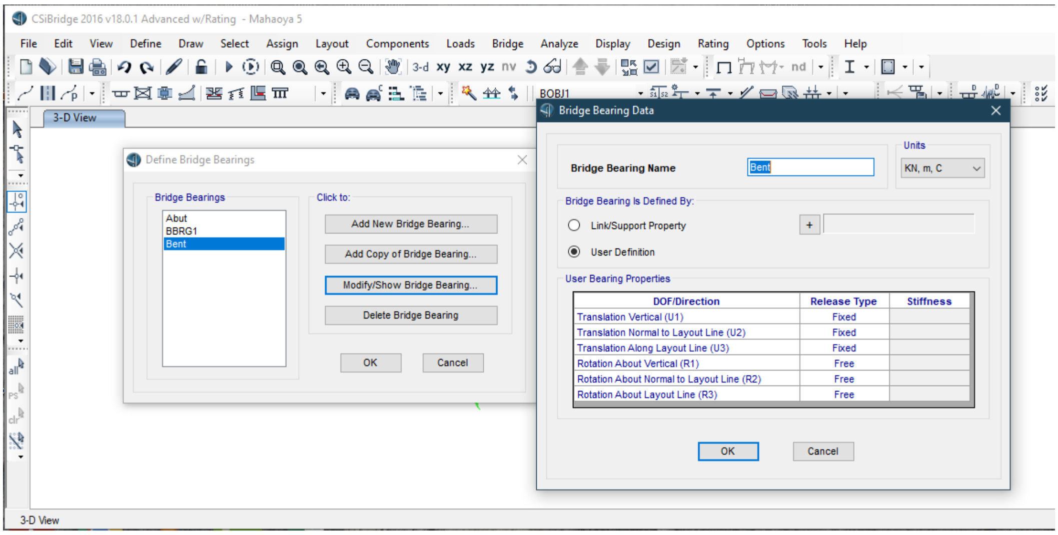


Figure A.9. Bridge bearing data-Abutment

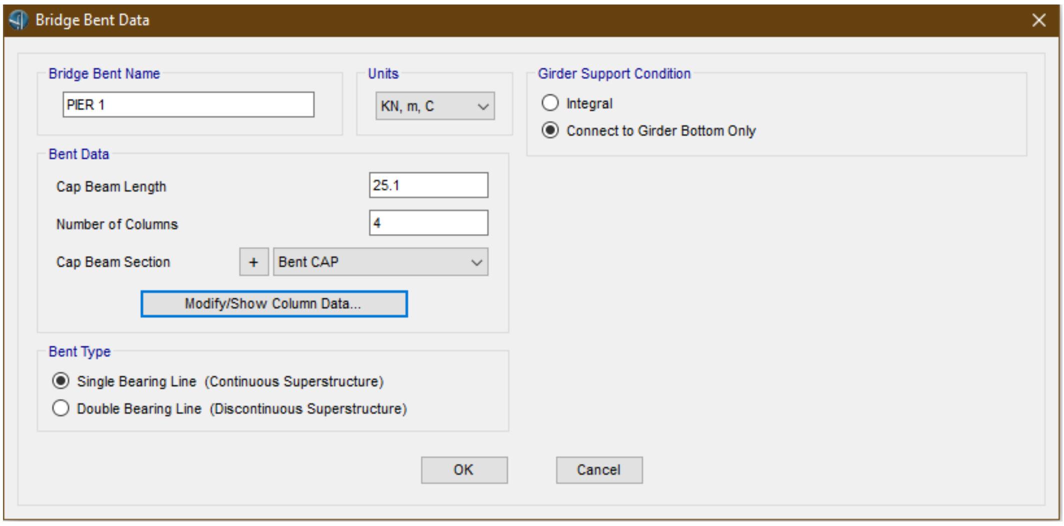


Figure A.10. Details of Bridge Bent

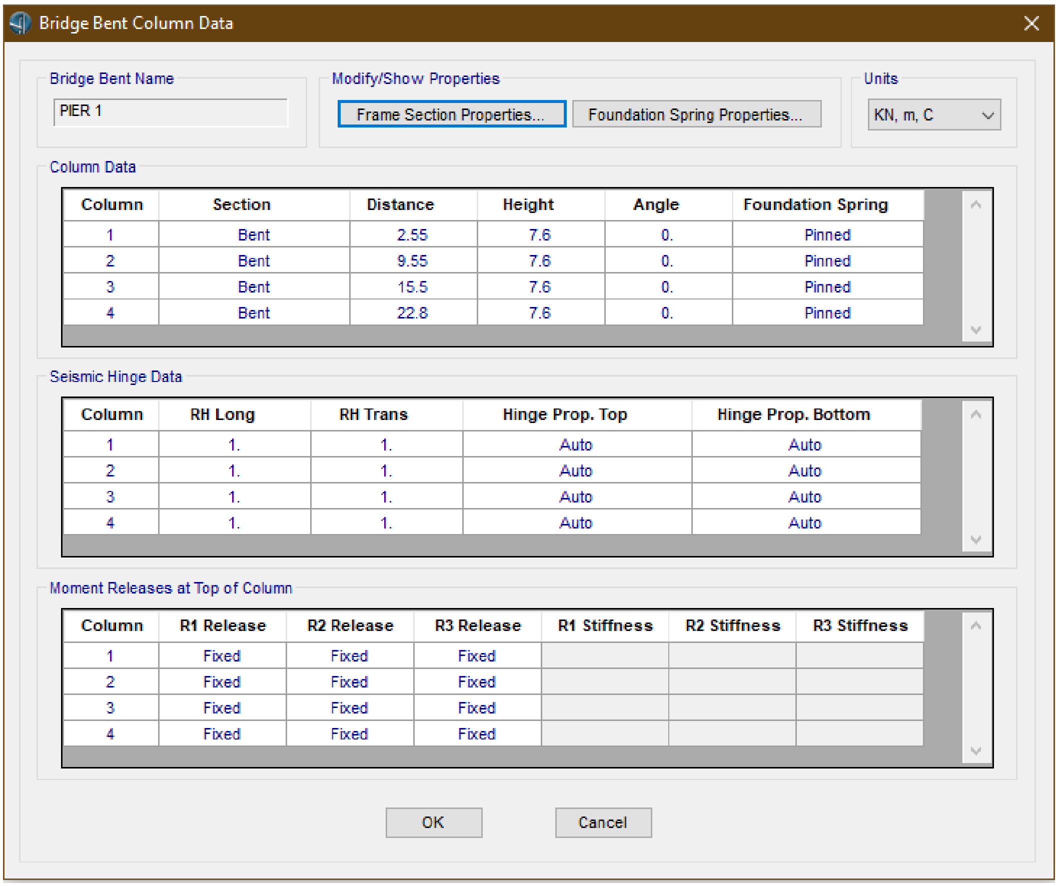


Figure A.11. Bridge bent column data

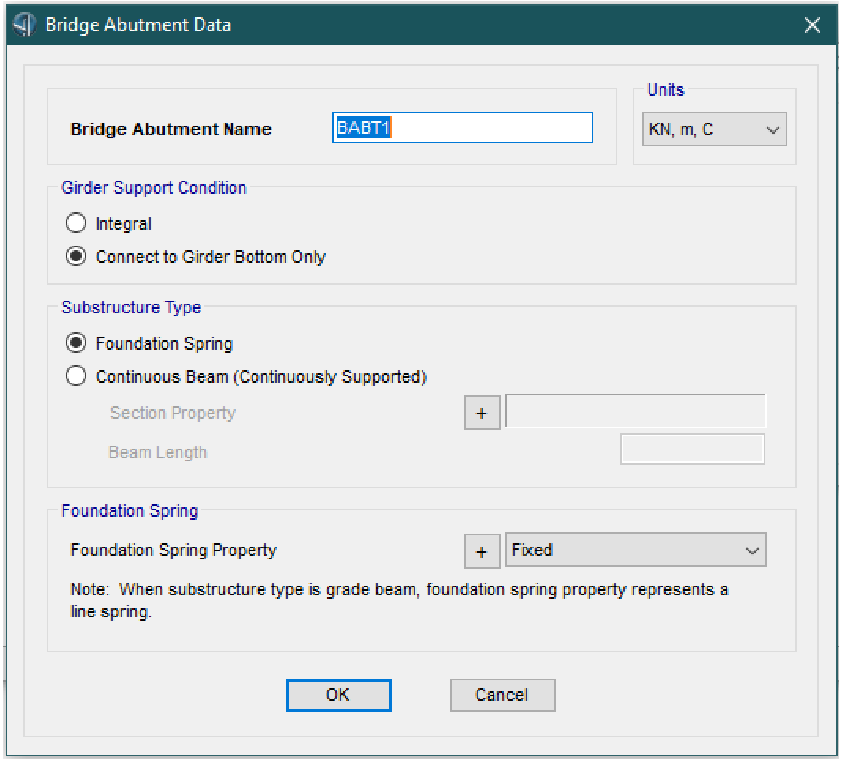
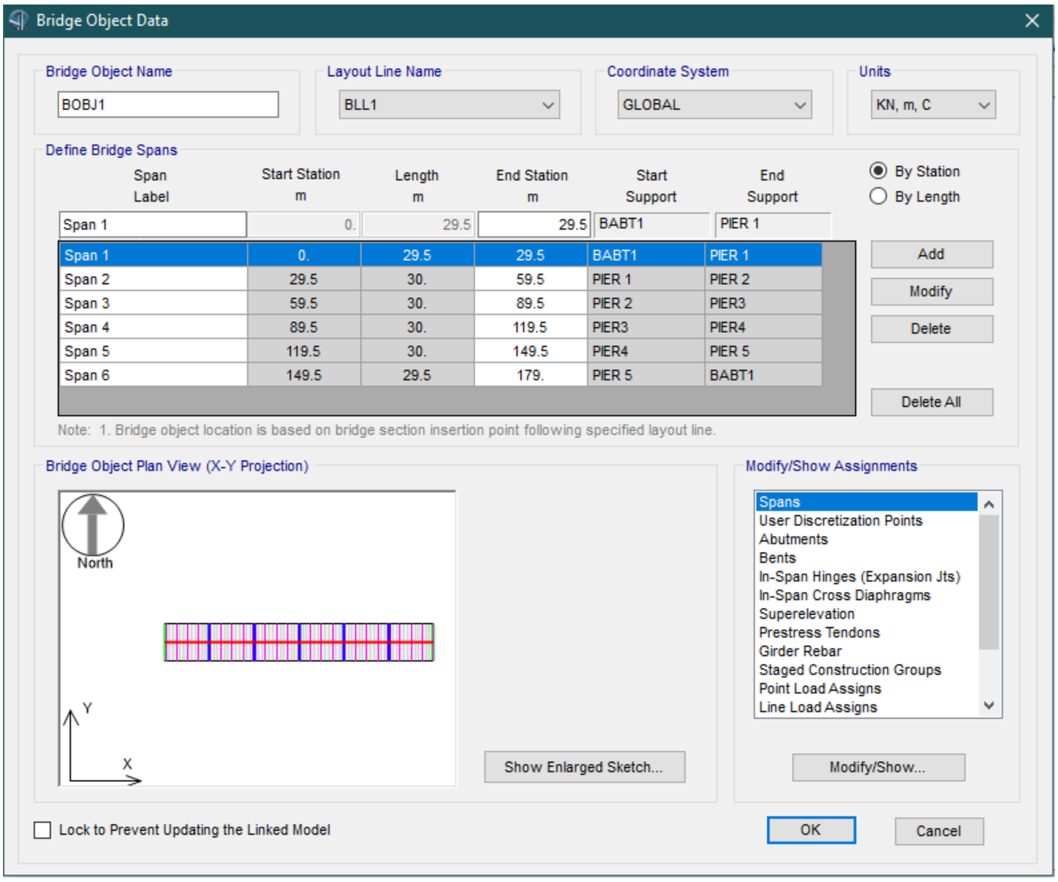


Figure A.12. Bridge abutment data

In bridge model, superstructure and piers will be generated as actual sections. However, abutments will not be generated as actual sections. Abutment will be represented as either spring or continuous beam. For seismic analysis, piers are the most critical elements. Therefore, abutments were not drawn in the model. If it is really needed to include abutments in the model, it is required to define it using area elements and add that to the model manually.

Soil properties along the pile depth are included to the model using soil springs. Assign ⟶ Joint ⟶ Springs steps used and spring properties assigned to the soil springs.



Span Label	Start Station m	Length m	End Station m	Start Support	End Support
Span 1	0.	29.5	29.5	BABT1	PIER 1
Span 1	0.	29.5	29.5	BABT1	PIER 1
Span 2	29.5	30.	59.5	PIER 1	PIER 2
Span 3	59.5	30.	89.5	PIER 2	PIER3
Span 4	89.5	30.	119.5	PIER3	PIER4
Span 5	119.5	30.	149.5	PIER4	PIER 5
Span 6	149.5	29.5	179.	PIER 5	BABT1

Figure A.13. Bridge object data

Once the bridge components defined, bridge object data need to be defined to connect the components to required bridge object. This will be accessed in the program using the Bridge → Bridge object in main menu. In the bridge object data window, it is required to define the details which required to generate complete modal. As shown in figure A.13. , modify/show assignments list have various items such as abutments, bents, diaphragms and those items need to be clicked individually and define required details such as the way of connecting to bearing etc.

When all the items are defined properly, bridge object should be updated as given in the bridge menu.

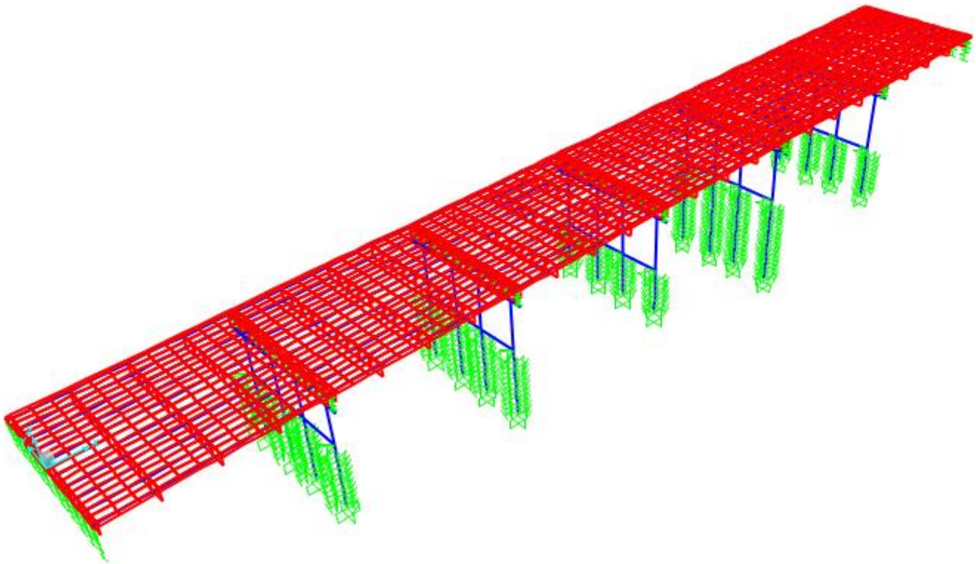


Figure A.14. Finite element 2D model of bridge

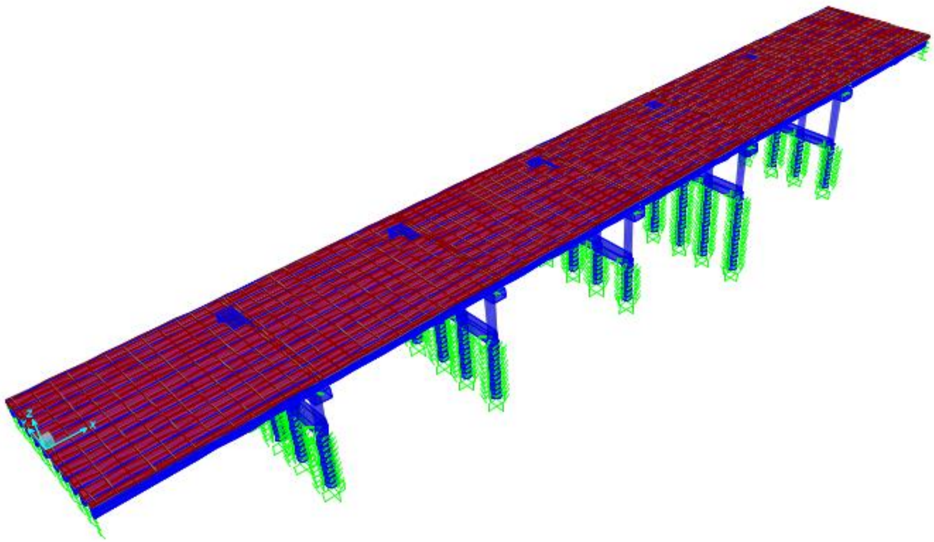


Figure A.15. Finite element 3D model of the bridge

The next step of the model generation is to define the response spectrum function. That was done using a tab named Loads in the main menu.

Loads ⟶ Functions ⟶ Response spectrum

Response spectrum proposed by Senevirathna H.N. et al. [16] for Sri Lanka was used for analysis.

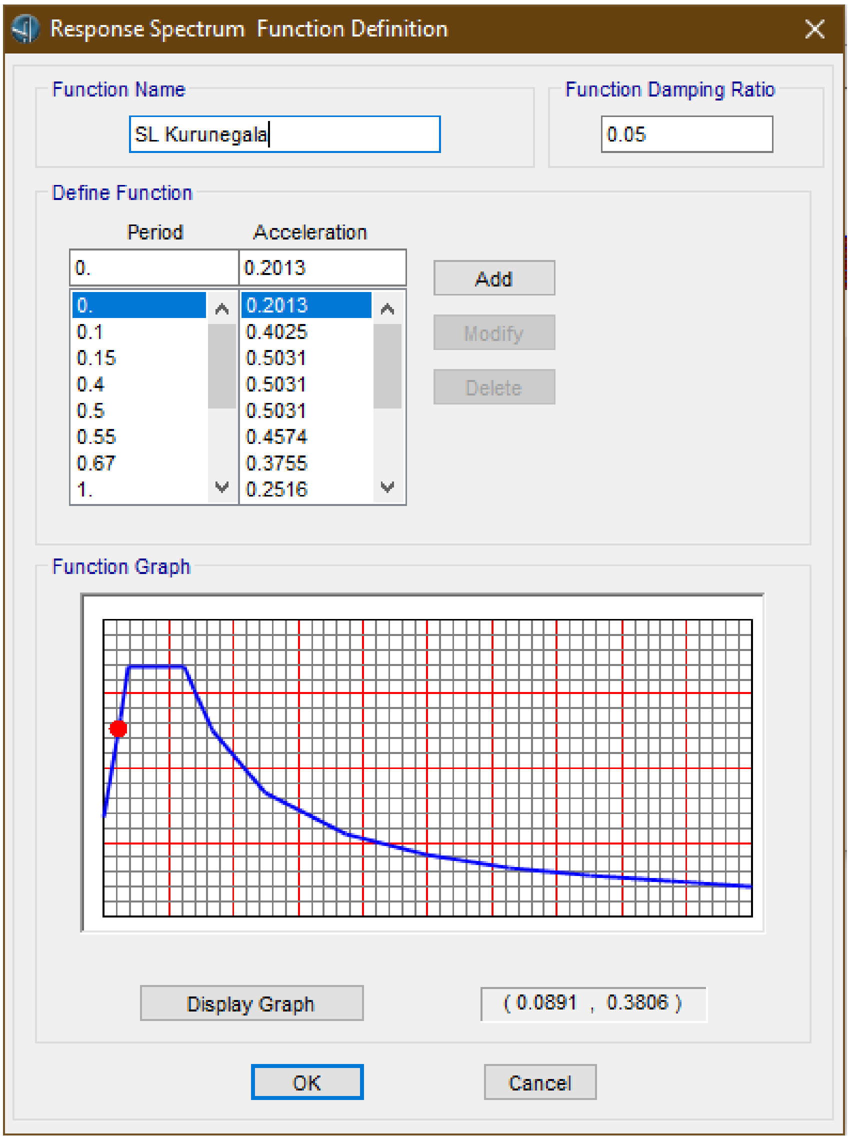


Figure A.16. Response spectrum used in analysis

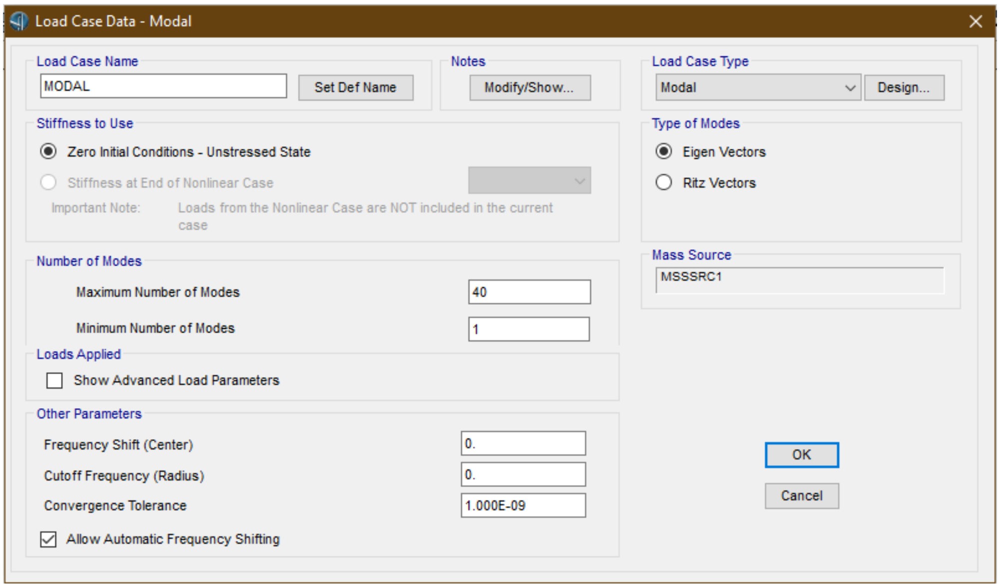


Figure A.17. Load case-Modal

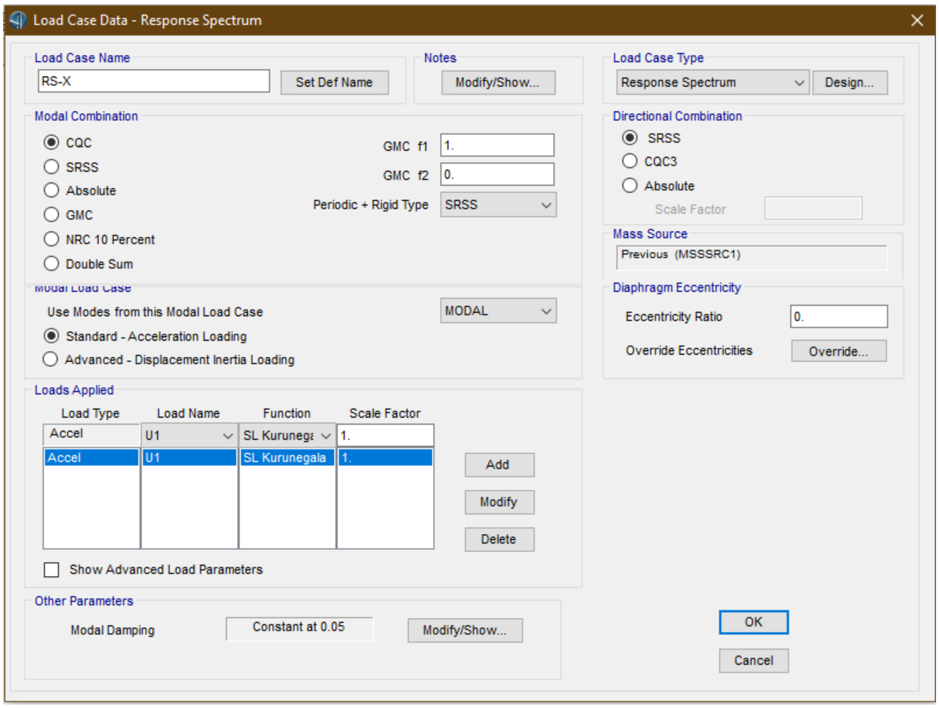


Figure A.18. Load case data-Response spectrum

After completion of bridge object, defining response spectrum and defining required load cases and load combinations, model need to be run and complete analysis to obtain results.

APPENDIX B

Results of Case studies

Results of Case Study 2

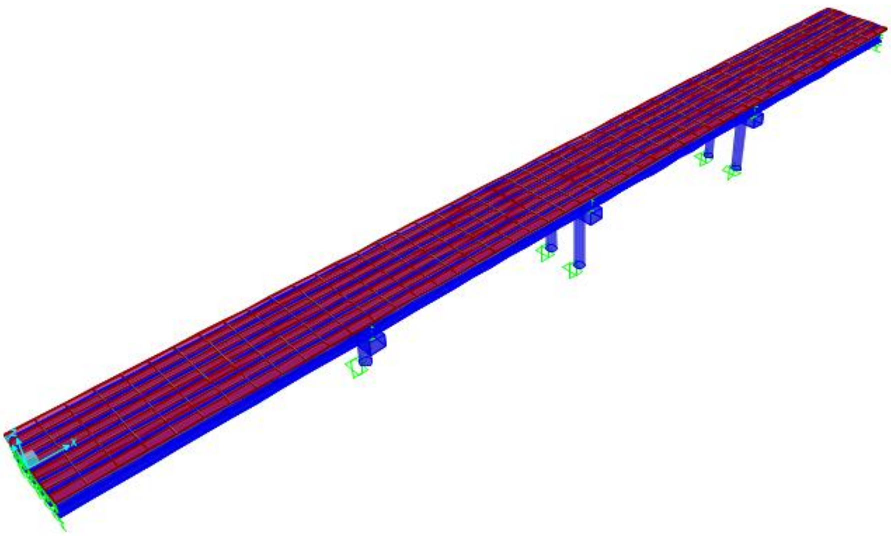
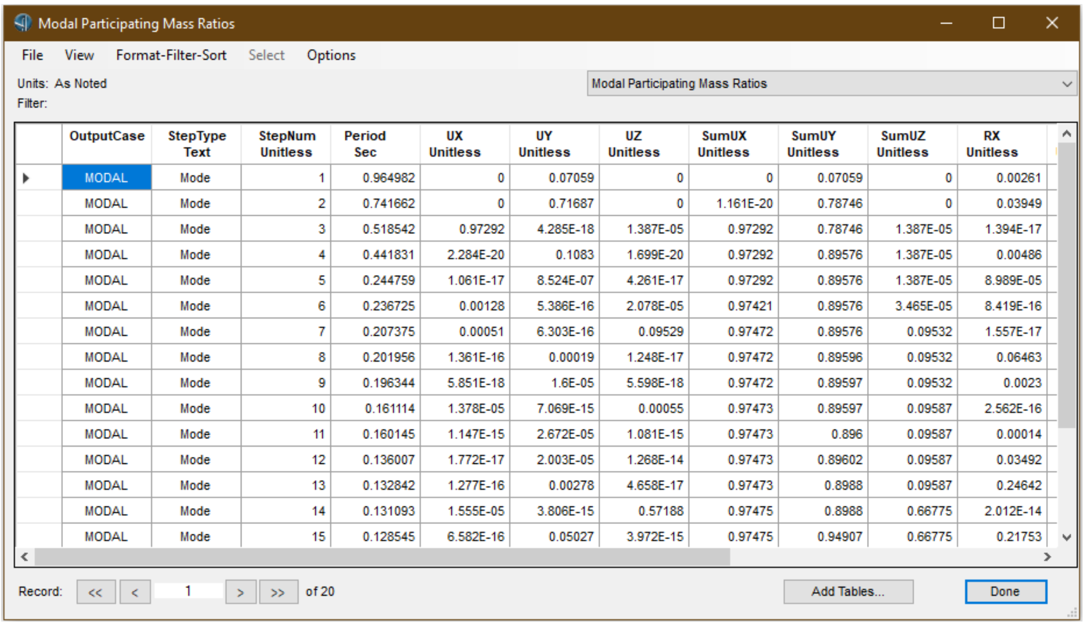


Figure B.1. Finite element model of bridge-Case study 2



Modal Participating Mass Ratios

File View Format-Filter-Sort Select Options

Units: As Noted

Filter:

OutputCase	StepType Text	StepNum Unitless	Period Sec	UX Unitless	UY Unitless	UZ Unitless	SumUX Unitless	SumUY Unitless	SumUZ Unitless	RX Unitless
MODAL	Mode	1	0.964982	0	0.07059	0	0	0.07059	0	0.00261
MODAL	Mode	2	0.741662	0	0.71687	0	1.161E-20	0.78746	0	0.03949
MODAL	Mode	3	0.518542	0.97292	4.285E-18	1.387E-05	0.97292	0.78746	1.387E-05	1.394E-17
MODAL	Mode	4	0.441831	2.284E-20	0.1083	1.699E-20	0.97292	0.89576	1.387E-05	0.00486
MODAL	Mode	5	0.244759	1.061E-17	8.524E-07	4.261E-17	0.97292	0.89576	1.387E-05	8.989E-05
MODAL	Mode	6	0.236725	0.00128	5.386E-16	2.078E-05	0.97421	0.89576	3.465E-05	8.419E-16
MODAL	Mode	7	0.207375	0.00051	6.303E-16	0.09529	0.97472	0.89576	0.09532	1.557E-17
MODAL	Mode	8	0.201956	1.361E-16	0.00019	1.248E-17	0.97472	0.89596	0.09532	0.06463
MODAL	Mode	9	0.196344	5.851E-18	1.6E-05	5.598E-18	0.97472	0.89597	0.09532	0.0023
MODAL	Mode	10	0.161114	1.378E-05	7.069E-15	0.00055	0.97473	0.89597	0.09587	2.562E-16
MODAL	Mode	11	0.160145	1.147E-15	2.672E-05	1.081E-15	0.97473	0.896	0.09587	0.00014
MODAL	Mode	12	0.136007	1.772E-17	2.003E-05	1.268E-14	0.97473	0.89602	0.09587	0.03492
MODAL	Mode	13	0.132842	1.277E-16	0.00278	4.658E-17	0.97473	0.8988	0.09587	0.24642
MODAL	Mode	14	0.131093	1.555E-05	3.806E-15	0.57188	0.97475	0.8988	0.66775	2.012E-14
MODAL	Mode	15	0.128545	6.582E-16	0.05027	3.972E-15	0.97475	0.94907	0.66775	0.21753

Record: << < 1 > >> of 20 Add Tables... Done

Figure B.2. Modal participating mass ratio

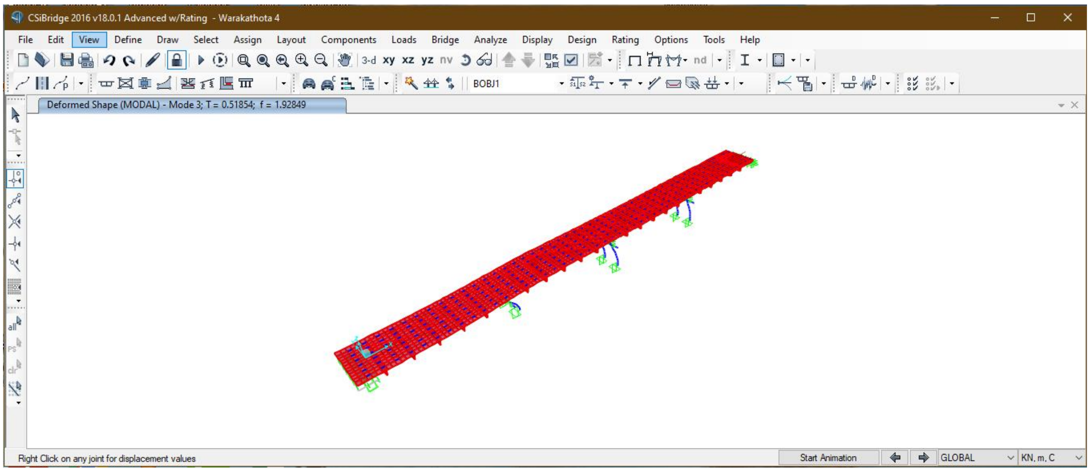


Figure B.3. Deformed shape Mode 3-Case study 2

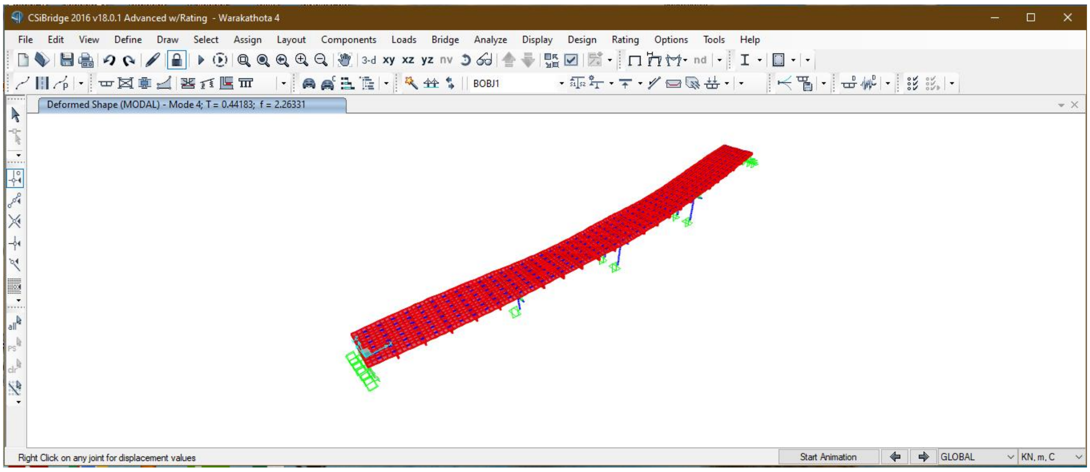


Figure B.4. Deformed shape mode 4- Case study 2

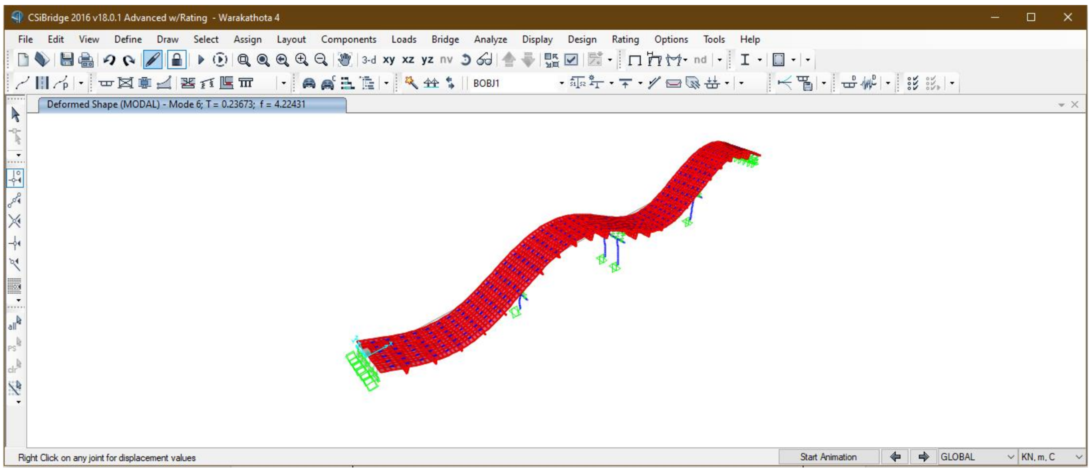


Figure B.5. Deformed shape mode 6-Case study 2

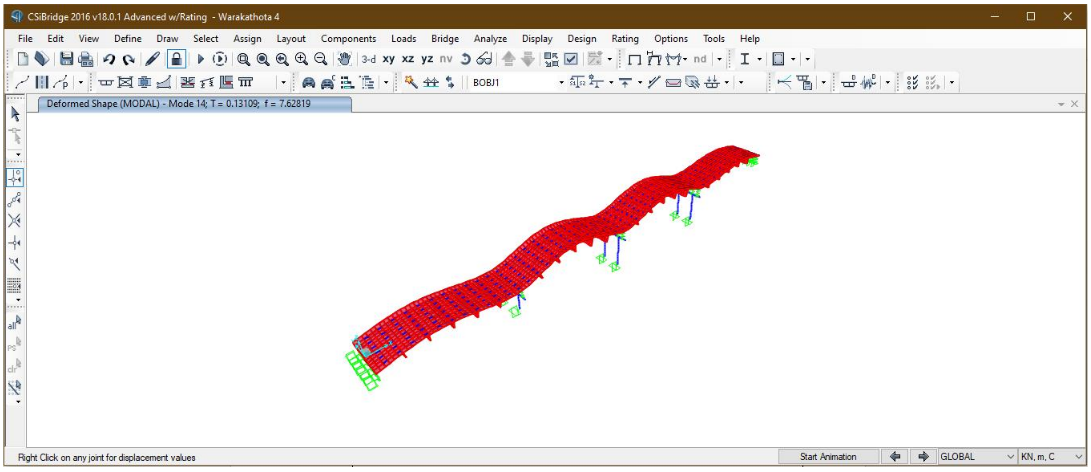
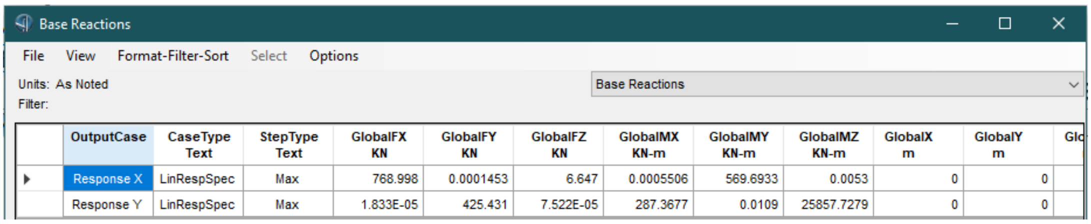


Figure B.6. Deformed shape mode 14-Case study 2

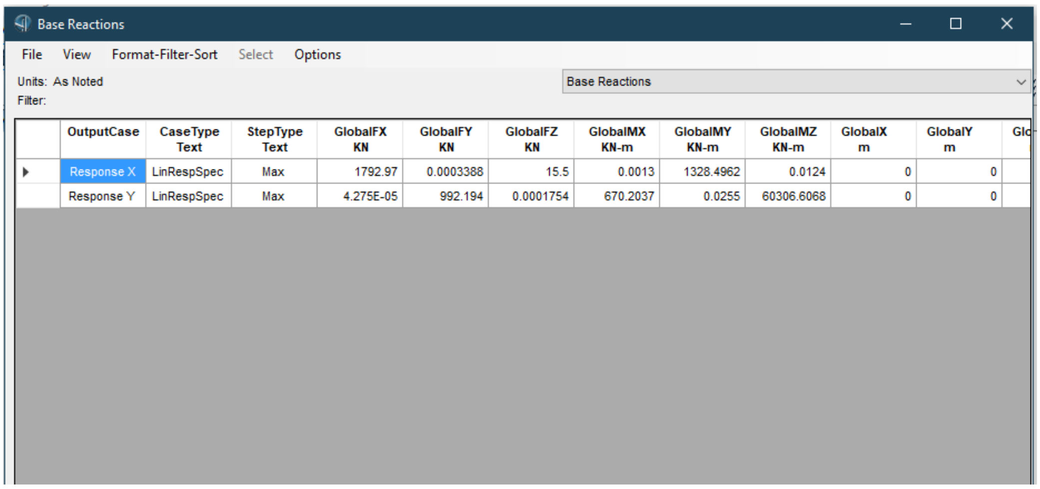


Base Reactions

Units: As Noted

OutputCase	CaseType Text	StepType Text	GlobalFX KN	GlobalFY KN	GlobalFZ KN	GlobalMX KN-m	GlobalMY KN-m	GlobalMZ KN-m	GlobalX m	GlobalY m	Glo
Response X	LinRespSpec	Max	768.998	0.0001453	6.647	0.0005506	569.6933	0.0053	0	0	
Response Y	LinRespSpec	Max	1.833E-05	425.431	7.522E-05	287.3677	0.0109	25857.7279	0	0	

Figure B.7. Base reactions for ductile behavior -case study 2



Base Reactions

Units: As Noted

OutputCase	CaseType Text	StepType Text	GlobalFX KN	GlobalFY KN	GlobalFZ KN	GlobalMX KN-m	GlobalMY KN-m	GlobalMZ KN-m	GlobalX m	GlobalY m	Glo
Response X	LinRespSpec	Max	1792.97	0.0003388	15.5	0.0013	1328.4962	0.0124	0	0	
Response Y	LinRespSpec	Max	4.275E-05	992.194	0.0001754	670.2037	0.0255	60306.6068	0	0	

Figure B.8. Base reactions for limited ductile behavior-case study 2

Results of Case Study 3

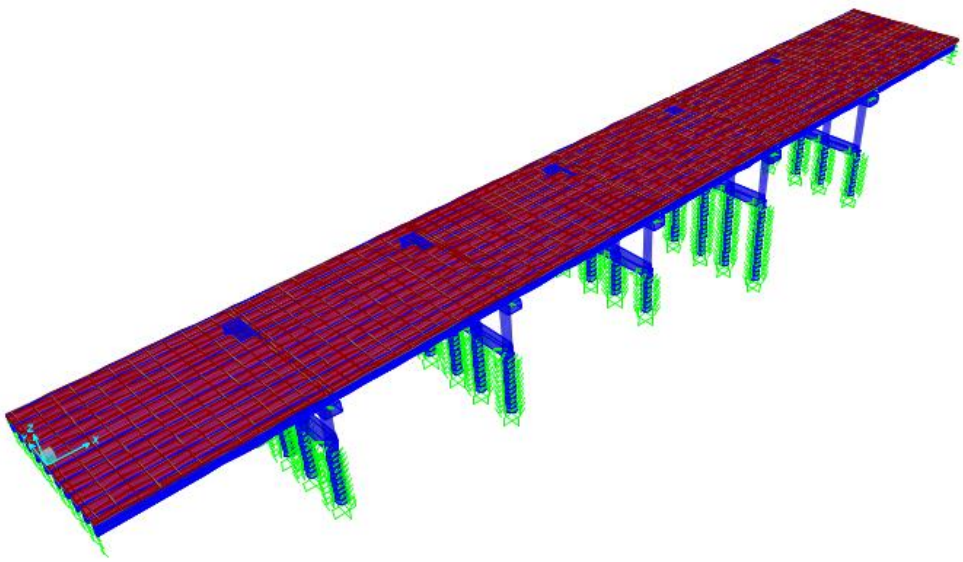
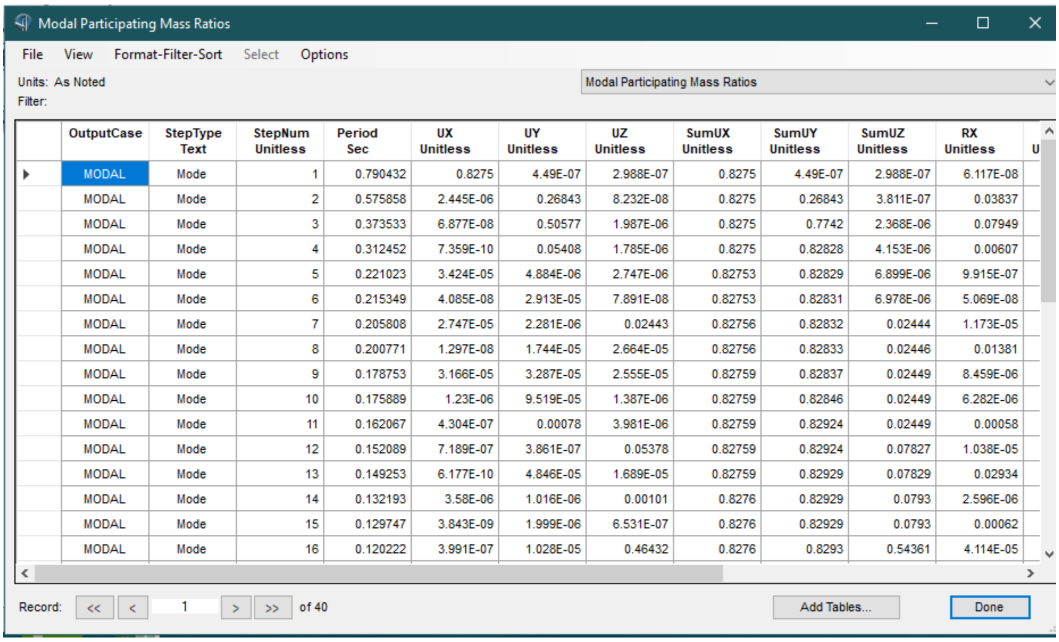


Figure B.9. Finite element 3D Modal- Case study 3



Modal Participating Mass Ratios

File View Format-Filter-Sort Select Options

Units: As Noted

Modal Participating Mass Ratios

Filter:

OutputCase	StepType Text	StepNum Unitless	Period Sec	UX Unitless	UY Unitless	UZ Unitless	SumUX Unitless	SumUY Unitless	SumUZ Unitless	RX Unitless
MODAL	Mode	1	0.790432	0.8275	4.49E-07	2.988E-07	0.8275	4.49E-07	2.988E-07	6.117E-08
MODAL	Mode	2	0.575858	2.445E-06	0.26843	8.232E-08	0.8275	0.26843	3.811E-07	0.03837
MODAL	Mode	3	0.373533	6.877E-08	0.50577	1.987E-06	0.8275	0.7742	2.368E-06	0.07949
MODAL	Mode	4	0.312452	7.359E-10	0.05408	1.785E-06	0.8275	0.82828	4.153E-06	0.00607
MODAL	Mode	5	0.221023	3.424E-05	4.884E-06	2.747E-06	0.82753	0.82829	6.899E-06	9.915E-07
MODAL	Mode	6	0.215349	4.085E-08	2.913E-05	7.891E-08	0.82753	0.82831	6.978E-06	5.069E-08
MODAL	Mode	7	0.205808	2.747E-05	2.281E-06	0.02443	0.82756	0.82832	0.02444	1.173E-05
MODAL	Mode	8	0.200771	1.297E-08	1.744E-05	2.664E-05	0.82756	0.82833	0.02446	0.01381
MODAL	Mode	9	0.178753	3.166E-05	3.287E-05	2.555E-05	0.82759	0.82837	0.02449	8.459E-06
MODAL	Mode	10	0.175889	1.23E-06	9.519E-05	1.387E-06	0.82759	0.82846	0.02449	6.282E-06
MODAL	Mode	11	0.162067	4.304E-07	0.00078	3.981E-06	0.82759	0.82924	0.02449	0.00058
MODAL	Mode	12	0.152089	7.189E-07	3.861E-07	0.05378	0.82759	0.82924	0.07827	1.038E-05
MODAL	Mode	13	0.149253	6.177E-10	4.846E-05	1.689E-05	0.82759	0.82929	0.07829	0.02934
MODAL	Mode	14	0.132193	3.58E-06	1.016E-06	0.00101	0.8276	0.82929	0.0793	2.596E-06
MODAL	Mode	15	0.129747	3.843E-09	1.999E-06	6.531E-07	0.8276	0.82929	0.0793	0.00062
MODAL	Mode	16	0.120222	3.991E-07	1.028E-05	0.46432	0.8276	0.8293	0.54361	4.114E-05

Record: << < 1 > >> of 40 Add Tables... Done

Figure B.10. Modal participating mass ratio-Case study 3

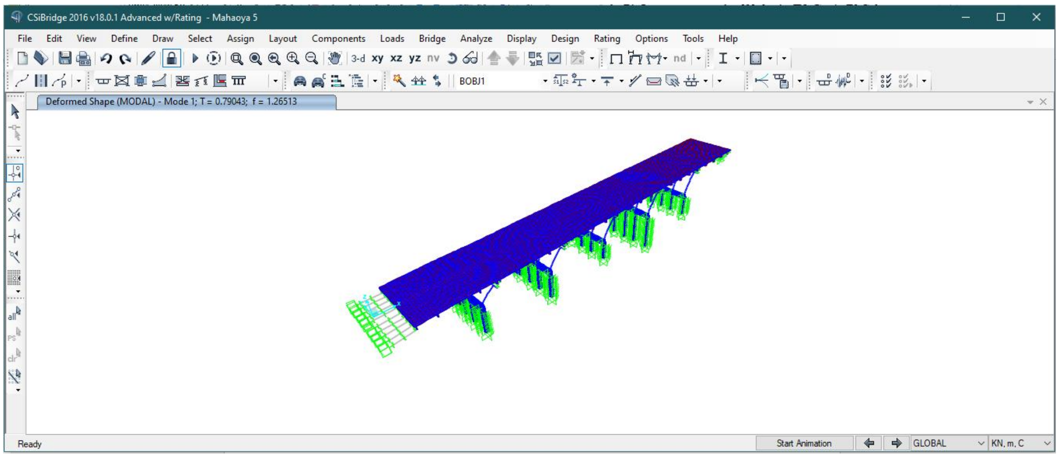


Figure B.11. Deformed shape mode 1-Case study 3

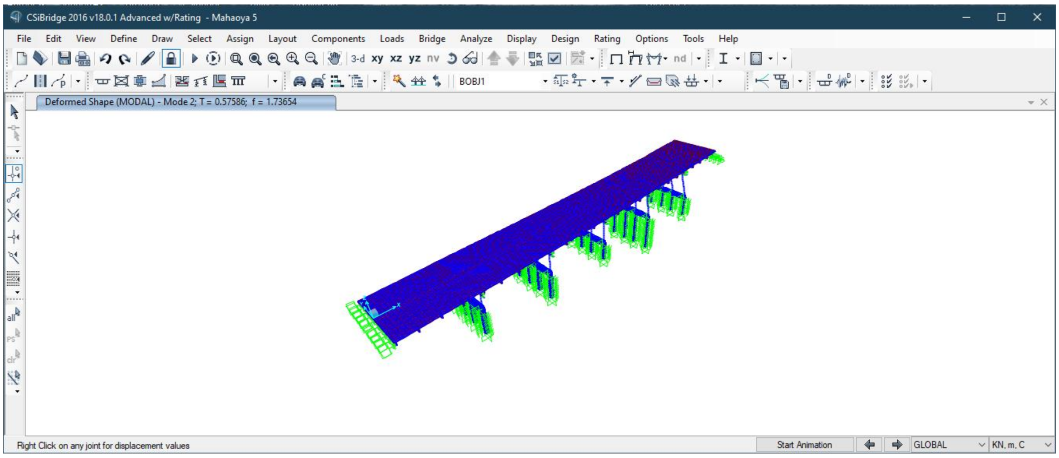


Figure B.12. Deformed shape mode 2- Case study 3

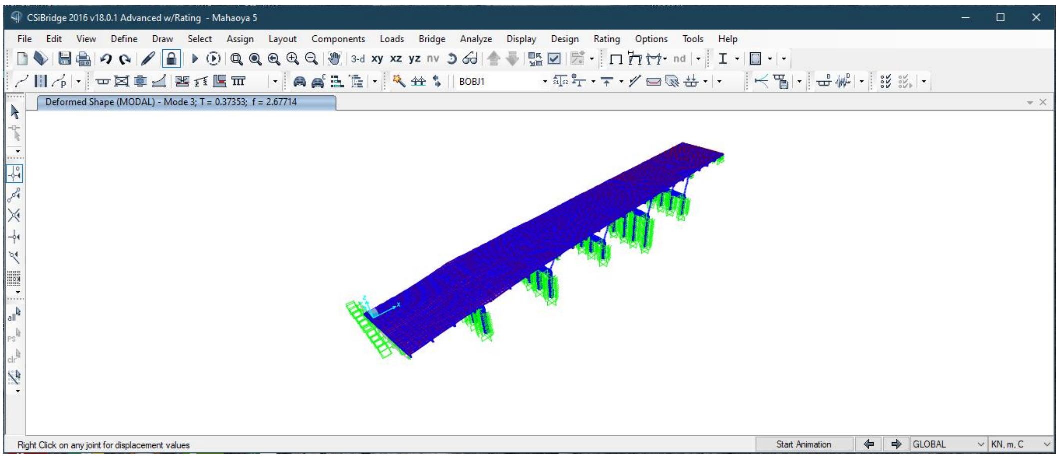


Figure B.13. Deformed shape mode 3- Case study 3

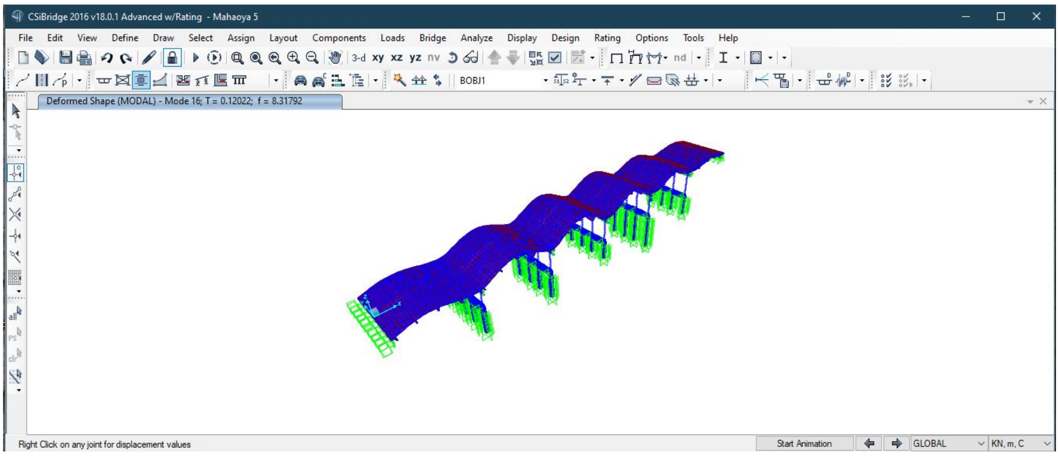
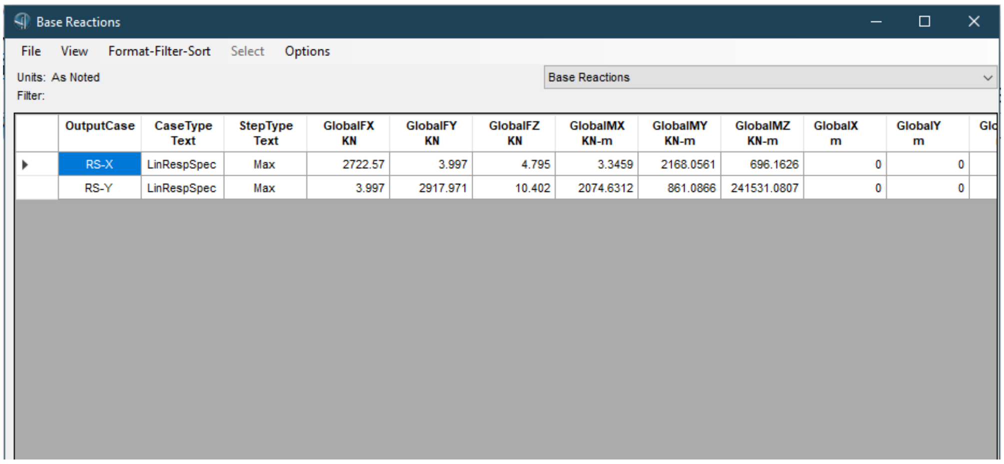


Figure B.14. Deformed shape mode 16- Case study 3

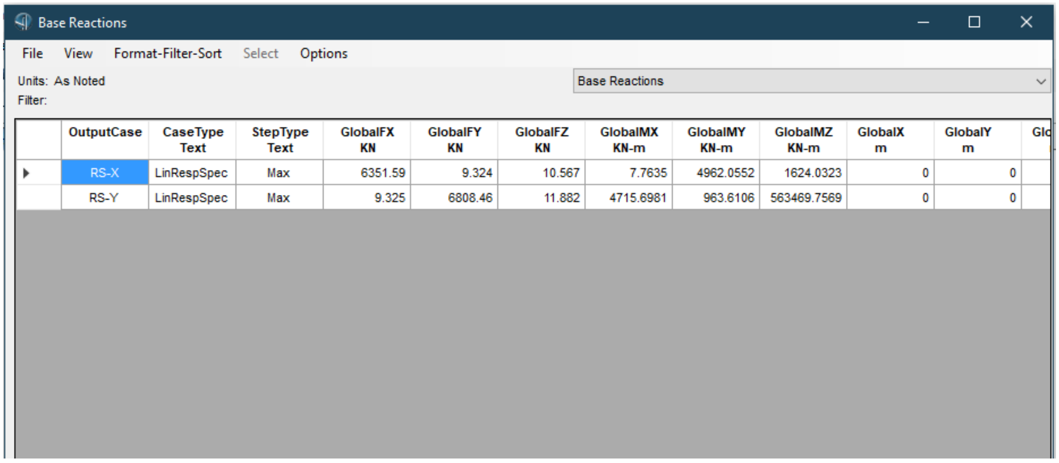


Base Reactions

Units: As Noted

OutputCase	CaseType Text	StepType Text	GlobalFX KN	GlobalFY KN	GlobalFZ KN	GlobalMX KN-m	GlobalMY KN-m	GlobalMZ KN-m	GlobalX m	GlobalY m
RS-X	LinRespSpec	Max	2722.57	3.997	4.795	3.3459	2168.0561	696.1626	0	0
RS-Y	LinRespSpec	Max	3.997	2917.971	10.402	2074.6312	861.0866	241531.0807	0	0

Figure B.15. Base reactions for ductile behavior- Case study 3



Base Reactions

Units: As Noted

OutputCase	CaseType Text	StepType Text	GlobalFX KN	GlobalFY KN	GlobalFZ KN	GlobalMX KN-m	GlobalMY KN-m	GlobalMZ KN-m	GlobalX m	GlobalY m
RS-X	LinRespSpec	Max	6351.59	9.324	10.567	7.7635	4962.0552	1624.0323	0	0
RS-Y	LinRespSpec	Max	9.325	6808.46	11.882	4715.6981	963.6106	563469.7569	0	0

Figure B.16. Base Reactions for limited ductile behavior-Case study 3

APPENDIX C

Drawings of Original Design – Bridge at Mudduwa Batugedara (Non RDA) Road in Rathnapura

APPENDIX D

Drawings of Original Design- Mahaoya Bridge in Central Expressway 42+240 km