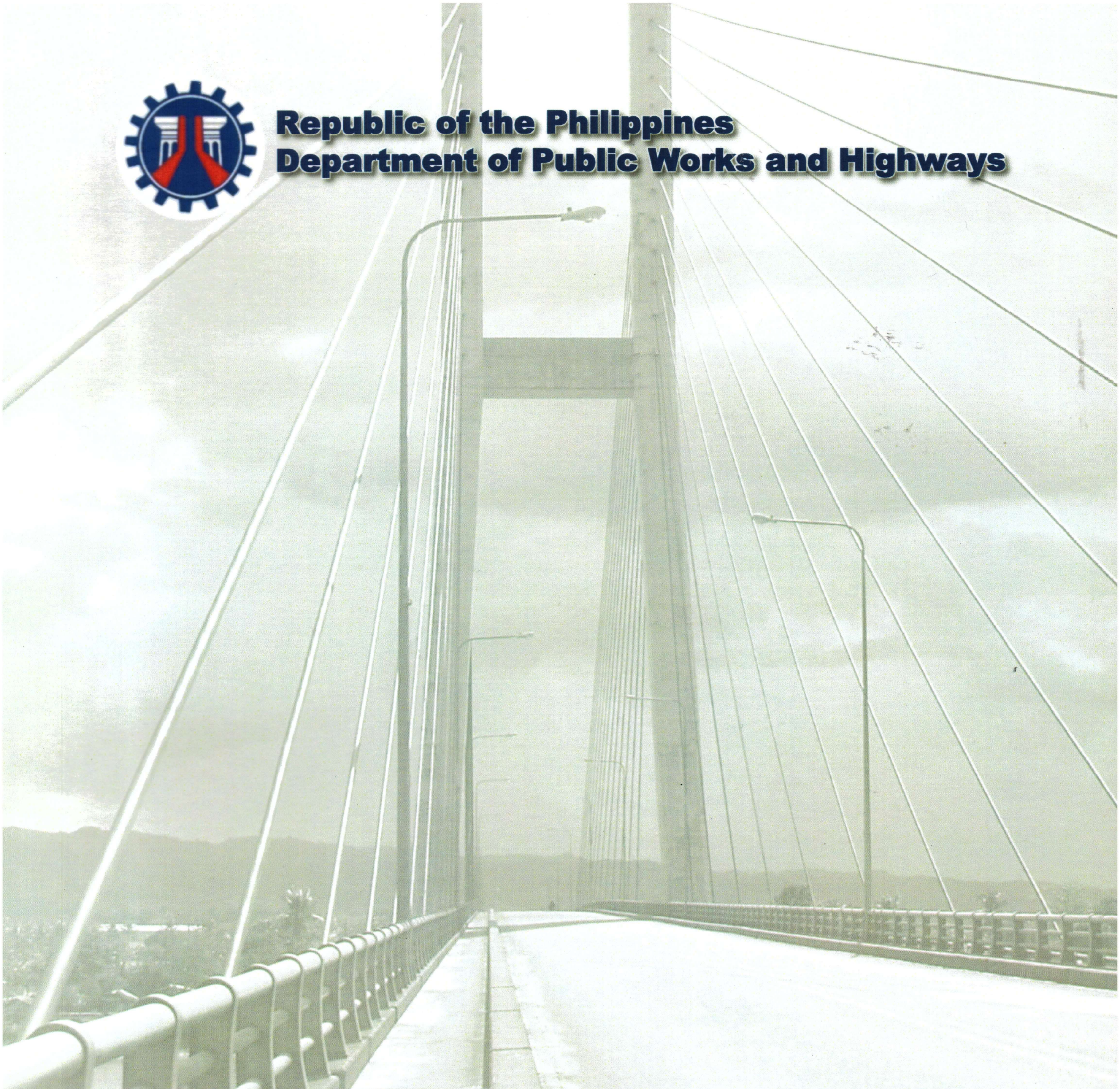
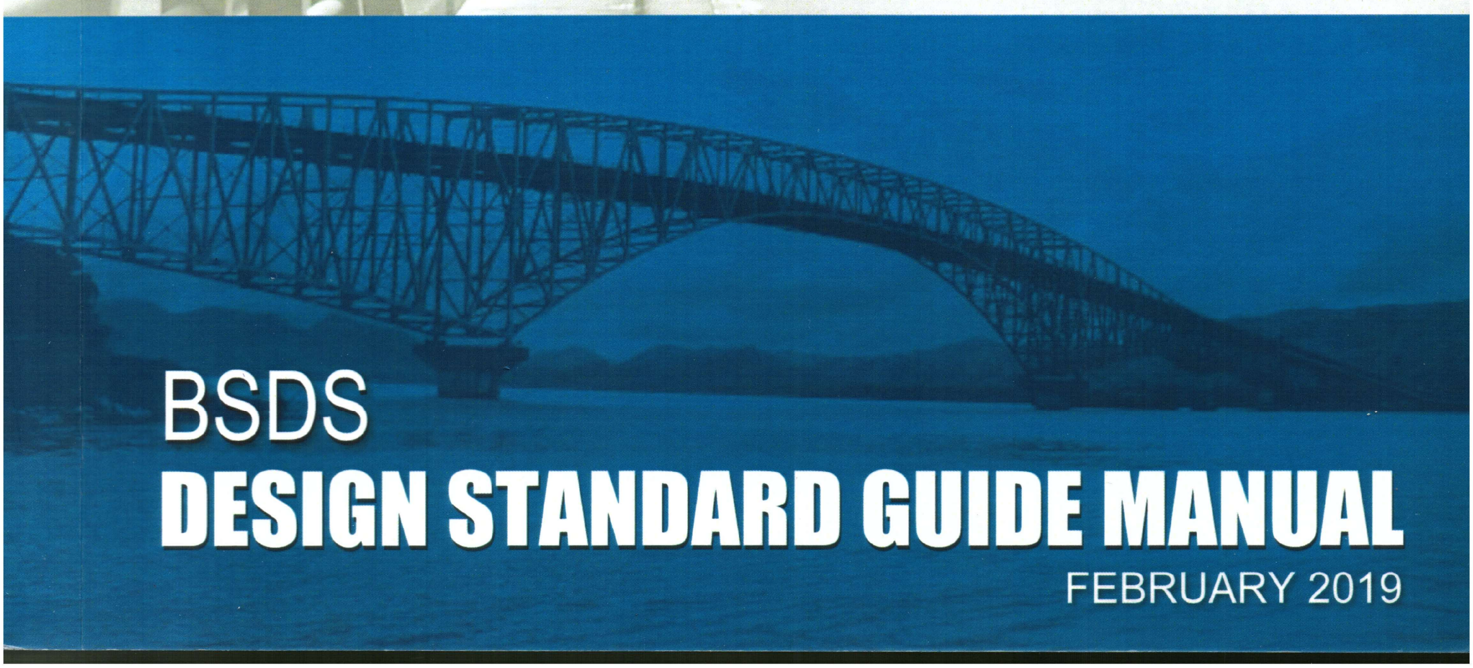
Republic of the Philippines
Department of Public Works and Highways





BSDS

DESIGN STANDARD GUIDE MANUAL

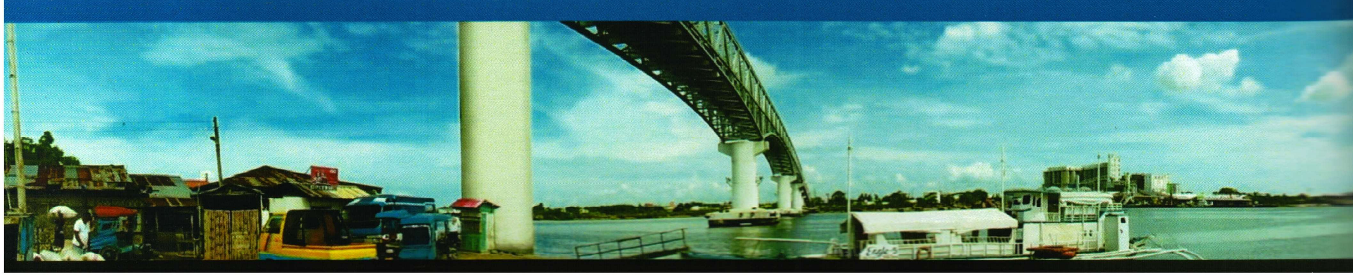
FEBRUARY 2019

Republic of the Philippines
Department of Public Works and Highways

The DPWH Guide Specifications for BSDS Design Standard Guide Manual is prepared under the
Consulting Services for the Detailed Design and Tender Assistance of the Metro Manila Priority Bridges Seismic Improvement Project (MMPBSIP) JICA Loan No. PH-P260 (Rebidding)

as implemented by:

in association with:



PREFACE

The development of the DPWH BSDS Design Standard Guide Manual is part of Technology Transfer under the Detailed Design and Tender Assistance of the Metro Manila Priority Bridges Seismic Improvement Projects (MMPBSIP) with financial assistance from Japan International Cooperation Agency (JICA). The object of this manual is to supplement the application and understanding of the seismic design of bridges in accordance with the DPWH LRFD Bridge Seismic Design Specifications (BSDS 2013). The manual also includes a design example of bridge isolation design of a conventional bridge in accordance with the procedures of the Highway Bridge Seismic Isolation Design Specifications (1st Edition, 2019) manual which is issued separately.

The compilation of the contents of the manual is a collective effort of the members of the Technology Transfer Team of the Consultant and the Core Engineer Group (CEG) of the DPWH. The members of the teams are listed overleaf, whose contributions are highly appreciated.

It is recommended that the manual be used as reference and guide for the DPWH engineers in the seismic design of bridges under large-scale earthquake.

While it is believed the data in the examples are correct for the specific project example, the information and data presented herein does not indicate full applicability to other similar projects. The designer is held liable to the verification of the data of his/her own design works.

The computer software program used in the dynamic analysis of the examples does not constitute an endorsement of software product for the future design works.

Acknowledgment is given to the Japan International Cooperation Agency (JICA) for the support in the preparation of this manual.

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ABBREVIATION

AASHTO	American Association of State Highway and Transportation Officials
ASD	Allowable Stress Design
BSDS	Bridge Seismic Design Specification
CQC	Complete Quadratic Combination
DGCS	Design Guidelines, Criteria and Standards
DPWH	Department of Public Works and Highways
EGM	Earthquake Ground Motion
FLS	Fatigue Limit Scale
ITHA	Inelastic Time History Analysis
LFD	Load Factor Design
LRFD	Load and Resistance Factor Design
LSD	Limit State Design
MDOF	Multi Degree of Freedom
MM	Multimode Method
OC	Operational Class
PEIS	PHIVOLCS Earthquake Intensity Scale
PGA	Peak Ground Acceleration
PHIVOLCS	Philippine Institute of Volcanology and Seismology
SDOF	Single Degree of Freedom
SLS	Serviceability Limit State
SM	Single-mode Method
SPL	Seismic Performance Level
SPZ	Seismic Performance Zones
SRSS	Square Root of the Sum of the Square
ULM	Uniform Load Method
ULS	Ultimate Limit State
WSD	Working Stress Design

CHAPTER 1: GENERAL

Chapter 1 General

1.1 Applicability

(1) This manual (BSDS *STANDARD DESIGN GUIDE MANUAL*) was prepared to provide guidelines to DPWH engineers in the seismic design and constructions of conventional new bridges under design large earthquake as an extreme event.

(2) The manual will serve as reference for the proper use and application of the principles of BSDS 2013 and its Interim Revision February 2019.

(3) The manual provides examples of seismic analysis of a particular bridge to assist in the understanding of site specific and other application of BSDS.

(4) The manual also provides comprehensive example of structural design of pier and abutment in compliance to the provisions of DGCS, 2015 or AASHTO LRFD.

(5) While it is believed the sample calculations given in the manual are well thought it is the responsibility of the designer to perform specific engineering study for the specific project.

1.2 Main Scope of Manual

(1) Analysis method (Simplified, Linear and Non-Linear analysis)

(2) Analysis Example

(3) Seismic Design of Pier and Abutment

(4) Calculation of Unseating Prevention Device

(5) Gap Bearings Adjacent Girders and Substructure

(6) Calculation of Liquefaction

1.3 Definitions and Notations/Symbols

Refer to the BSDS 2013, for definitions, notations and symbols.

CHAPTER 2: GENERAL CONSIDERATIONS FOR SEISMIC DESIGN

Chapter 2 General Considerations for Seismic Design

2.1 Design Philosophy

- Bridges play a major role as evacuation and emergency routes during a major disaster such as an earthquake. Therefore, it is necessary the bridges shall be designed to ensure the seismic performance by the Operational Class (OC) and the required Level of the design Earthquake Ground Motion (EGM) corresponding to an earthquake with return period event of 1000 years (7% probability of exceedance in 75 years) for life safety performance objective under the large earthquake.

- The design of bridges shall comply with minimum concepts specified in the DPWH D.O. No. 75 “DPWH Advisory for Seismic Design of Bridges”, 1992 as follows:

  1. Continuous bridges with monolithic multi-column bents have high degree of redundancy are recommended. Expansion joints and hinges should be kept to minimum.

  2. Decks should be made continuous if bridge is multi-span simple span.

  3. Restrainers or unseating device are required to all joints. Generous seat length should be provided to piers and abutments to prevent from loss of span.

  4. Transverse reinforcements in the zones of yielding are essential to confine the longitudinal bars and the concrete within the core of column.

  5. Plastic hinging should be forced to occur in the ductile column regions of the pier rather than the foundation.

  6. The stiffness of the bridge as a whole should be considered in the analysis including the soil-structure interaction.

- The following shall be taken into account in the design of bridges:

  1. Topographical, geological, geotechnical soil and other site conditions that may affect the seismic performance of the bridge.

  2. Selection of appropriate structural system with high seismic performance that is capable of fully resisting the earthquake forces utilizing the strength and ductility of the structural members.

- The following two levels of EGM shall be considered in the BSDS:

  1. Level 1 EGM, considering seismic hazard from small to moderate earthquake with high probability of occurrence during the bridge service life (100-year return period), for seismic serviceability design objective to ensure normal bridge functions.

2. Level 2 EGM, considering a seismic hazard corresponding to an earthquake with return period event of 1,000 years (7% probability of exceedance in 75 years) for life safety performance objective under the large earthquake.

2.2 Flowcharts

Note: The Articles shall be referred to BSDS 2013 (1st Edition)

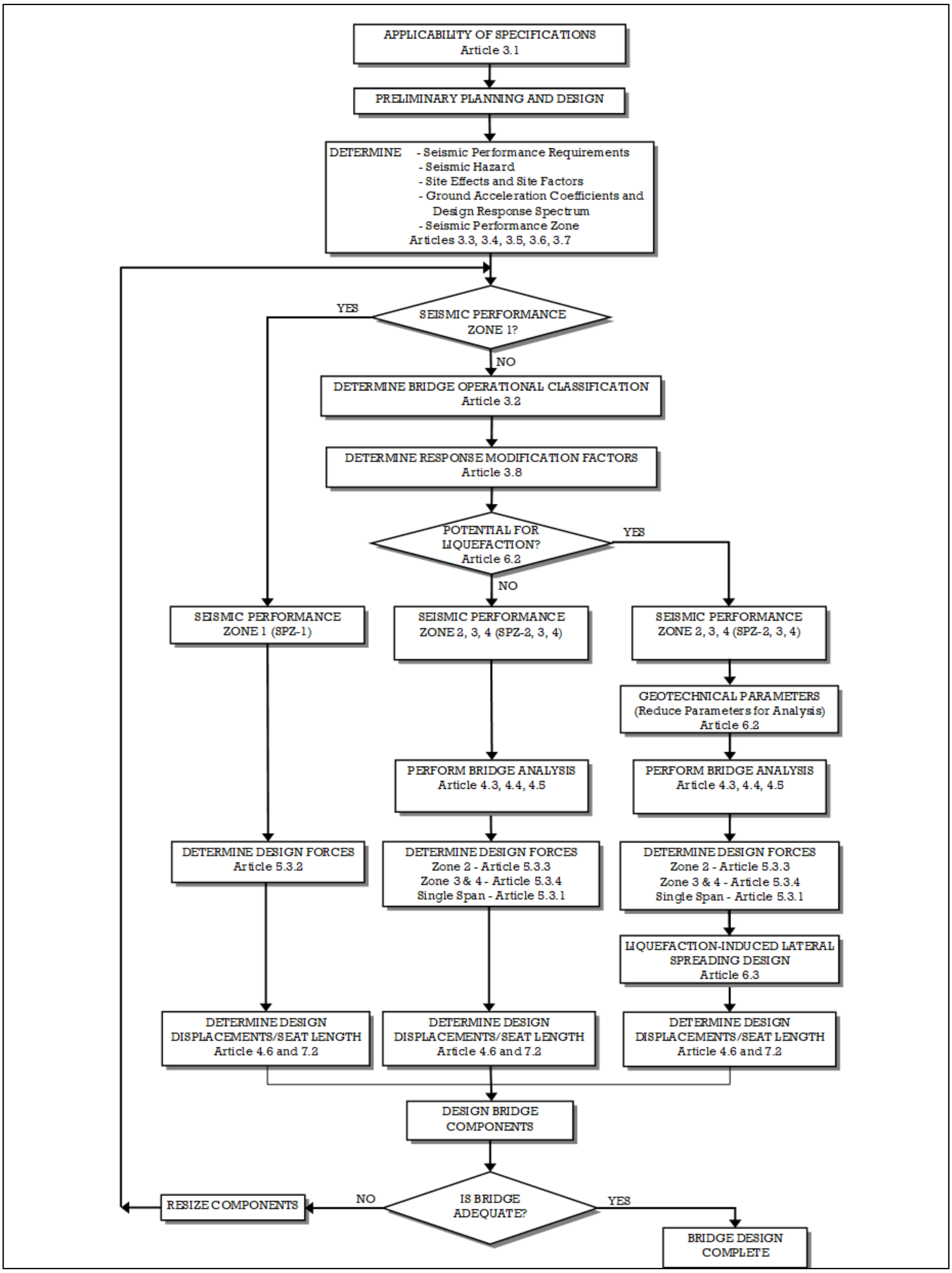


Figure 2.2-1 Seismic Design Procedure Flow Chart

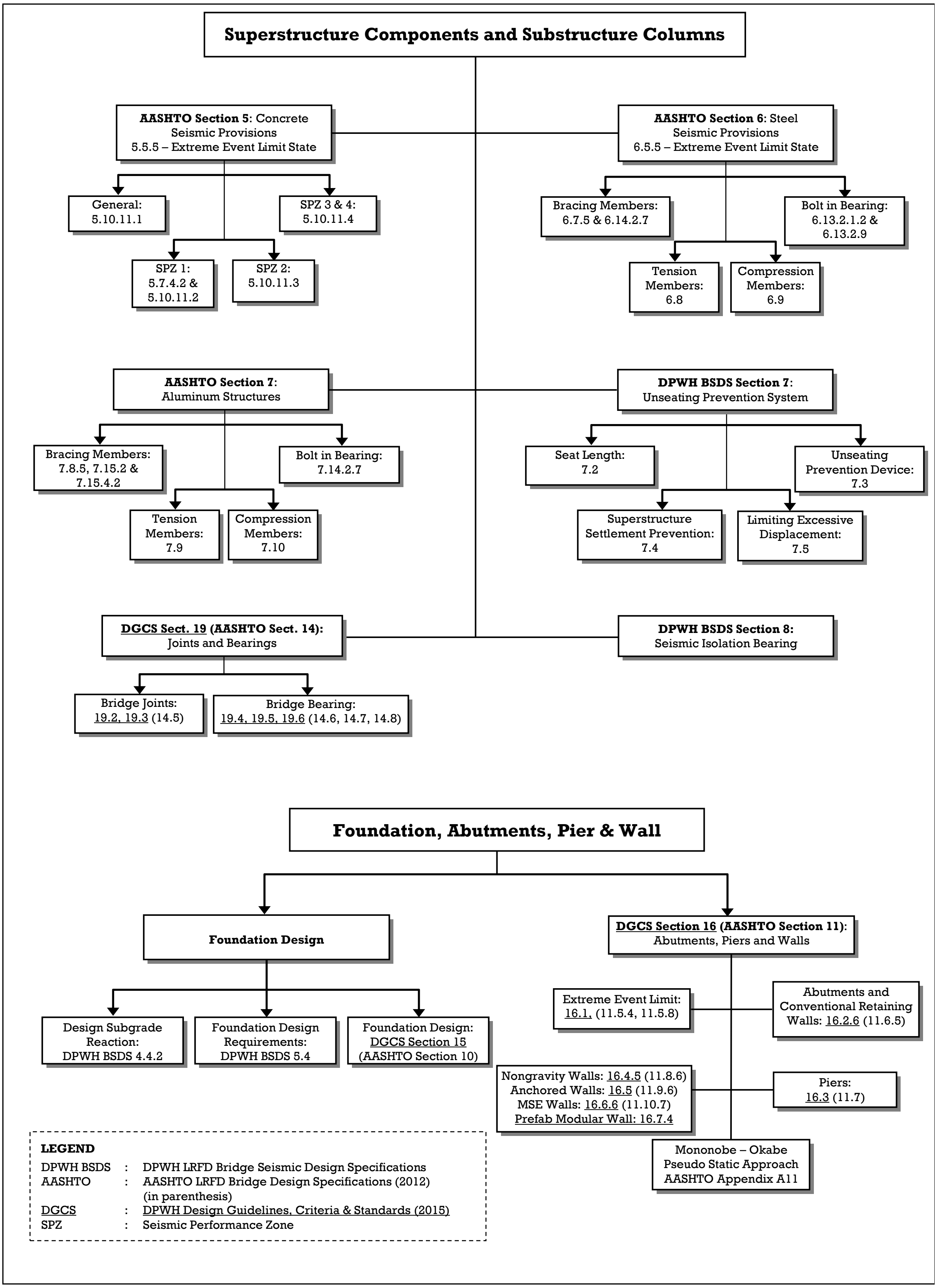


Figure 2.2-2 Seismic Detailing and Foundation Design Flow Chart

2.3 General Requirements

2.3.1 Bridge Operational Classification

(1) For the purpose of seismic design, bridges shall be classified into one of the following three operational categories:

Table 2.3-1 Operational Classification of Bridges

Operational Classification (OC)	Serviceability Performance	Description
OC-I (Critical Bridges)	• Bridges that must remain open to all traffic after the Level 2 design earthquake, i.e. 1,000-year return period event. • Other bridges required by DPWH to be open to emergency vehicles and vehicles for security/defense purposes immediately after an earthquake larger than the Level 2 design earthquake (AASHTO recommends a 2,500-year return for larger earthquakes).	Important bridges that meet any of the following criteria: • Bridges that do not have detours or alternative bridge route (e.g. bridges that connect islands where no other alternative bridge exist), • Bridges on roads and highways considered to be part of the regional disaster prevention route, • Bridges with span ≥ 100m, • Non-conventional bridges or special bridge types such as suspension, cable stayed, arch, etc. • Other bridge forms such as double-deck bridges, overcrossings or overbridges that could cause secondary disaster on important bridges/structures when collapsed, • As specified by the DPWH or those having jurisdiction on the bridge.
OC-II (Essential Bridges)	• Bridges that should, as a minimum, be open to emergency vehicles and for security/defense purposes within a short period after the Level 2 design earthquake, i.e. 1,000-year return period event.	Bridges located along the following roads/highways: • Pan-Philippine Highway, • Expressways (Urban and Inter-urban expressways), • Major/Primary national arterial highways (North-South Backbone, East-West Lateral, Other Roads of Strategic Importance), • Provincial, City and Municipal roads in view of disaster prevention and traffic strategy. Additionally, bridges that meet any of the following criteria:

		• Bridges with detours greater than 25 kilometers • As specified by the DPWH or those having jurisdiction on the bridge
OC-III (Other Bridges)	• All other bridges not required to satisfy OC-I or OC-II performance.	• All other bridges not classified as OC-I or OC-II

The DPWH or those having jurisdiction shall classify the bridge into one of the above three operational categories.

(2) The basis of classification shall include social/survival and security/defense requirements. In classifying a bridge, considerations should be given to possible future changes in conditions and requirements.

2.3.2 Earthquake Ground Motion and Seismic Performance of Bridges

Table 2.3-2 Earthquake Ground Motion and Seismic Performance of Bridges

Earthquake Ground Motion (EGM)	**Bridge Operational Classification**		
	OC-I (Critical Bridges)	**OC-II (Essential Bridges)**	**OC-III (Other Bridges)**
Level 1 (Small to moderate earthquakes which are highly probable during the bridge service life, 100-year return)	SPL-1 *(Keep the bridge sound function; resist seismic forces within elastic limit)*	SPL-1 *(Keep the bridge sound function; resist seismic forces within elastic limit)*	SPL-1 *(Keep the bridge sound function; resist seismic forces within elastic limit)*
Level 2 (Large earthquakes with a 1,000-year return period)	SPL-2 *(Limited seismic damage and capable of immediately recovering bridge functions without structural repair)*	SPL-2 *(Limited seismic damage and capable of recovering bridge function with structural repair within short period)*	SPL-3 *(May suffer damage but should not cause collapse of bridge or any of its structural elements)*

2.3.3 Seismic Performance of Bridges

Table 2.3-3 Seismic Performance of Bridges

Seismic Performance	Seismic Safety Design	Seismic Serviceability Design	Seismic Repairability Design	
			Emergency Repairability	Permanent Repairability
Seismic Performance Level 1 (SPL-1): *Keeping the sound function of bridges*	Ensure safety against girder unseating; resist earthquake within elastic range	Ensure normal bridge functions	No repair work is needed to recover bridge functions	Only easy and minor repair works are needed
Seismic Performance Level 2 (SPL-2): *Limited damages and recovery*	Ensure safety against collapse and girder unseating	Capable of recovering functions within a short period after the earthquake event	Capable of recovering functions by emergency repair works	Capable of easily undertaking permanent repair work
Seismic Performance Level 3 (SPL-3): *No critical damages*	Ensure safety against collapse and girder unseating	-	-	-

2.3.4 Ground types (Site Class) for Seismic Design

Table 2.3-4 Ground Types (Site Class) for Seismic Design

Ground Type*	Soil Profile Description	Characteristic Value of Ground, T_G (s)
Type I	Hard (Good diluvial ground and rock)	$T_G < 0.2$
Type II	Medium (Diluvial and alluvial ground not belonging to Types I and III)	$0.2 \leq T_G < 0.6$
Type III	Soft (Soft ground and alluvial ground)	$0.6 \leq T_G$

** The Ground Type shall be determined quantitatively based on the Characteristic Value of Ground (T_G).*

where:

T_G : Characteristic value of ground (s)

H_i : Thickness of the *i-th* soil layer (m)

V_{si} : Average shear elastic wave velocity of the *i-th* soil layer (m/s) *i* : Numbers of the *i-th* soli layer from the ground surface when the ground is classified into *n* layers from the ground.

2.3.5 Values of Site Factor, F_{pga}, F_a, and F_v on Acceleration Spectrum

Table 2.3-5 Values of Site Factor, F_{pga}, at Zero-Period on Acceleration Spectrum

Ground Type (Site Class)	Spectral Acceleration Coefficient at Period 0.2 sec (S_S)[1]					
	$PGA \leq 0.10$	$PGA = 0.20$	$PGA_S = 0.30$	$PGA = 0.40$	$PGA = 0.50$	$PGA \geq 0.80$
I	1.2	1.2	1.1	1.0	1.0	1.0
II	1.6	1.4	1.2	1.0	0.9	0.85
III	2.5	1.7	1.2	0.9	0.8	0.75

Note:
[1] Use straight-line interpolation for intermediate values of ***PGA***.

Table 2.3-6 Values of Site Factor, F_a, for Short-Period Range on Acceleration Spectrum

Ground Type (Site Class)	Spectral Acceleration Coefficient at Period 0.2 sec (S_S)[1]					
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S = 1.25$	$S_S \geq 2.0$
I	1.2	1.2	1.1	1.0	1.0	1.0
II	1.6	1.4	1.2	1.0	0.9	0.85
III	2.5	1.7	1.2	0.9	0.8	0.75

Note:
[1] Use straight-line interpolation for intermediate values of S_S.

Table 2.3-7 Values of Site Factor, F_v, for Long-Period Range on Acceleration Spectrum

Ground Type (Site Class)	Spectral Acceleration Coefficient at Period 1.0 sec (S_1)[1]					
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 = 0.50$	$S_1 \geq 0.80$
I	1.7	1.6	1.5	1.4	1.4	1.4
II	2.4	2.0	1.8	1.6	1.5	1.5
III	3.5	3.2	2.8	2.4	2.4	2.0

Note:
[1] Use straight-line interpolation for intermediate values of S_1.

2.3.6 Design Response Spectrum

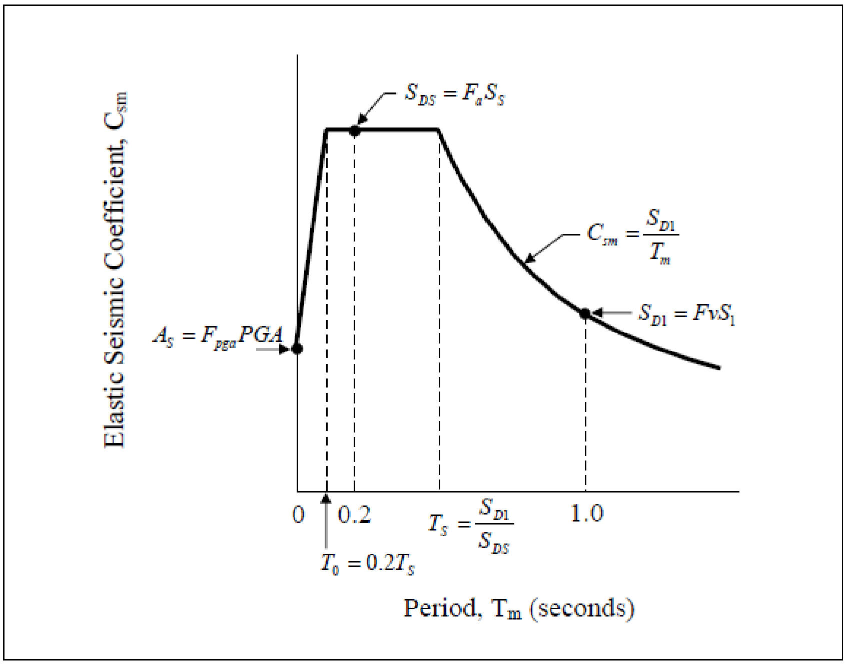


Figure 2.2-3 Design Response Spectrum

2.3.7 Seismic Performance Zones (SPZ)

Table 2.3-8 Seismic Performance Zones (SPZ)

Acceleration Coefficient, S_{D1}	Seismic Performance Zone
$S_{D1} \leq 0.15$	1
$0.15 < S_{D1} \leq 0.30$	2
$0.30 < S_{D1} \leq 0.50$	3
$0.50 < S_{D1}$	4

2.3.8 Response Modification Factors – Substructures

Table 2.3-9 Response Modification Factors – Substructures

Substructure	Operational Category		
	OC-I (Critical)	OC-II (Essential)	OC-III (Others)
Wall-type piers – larger dimension	1.5	1.5	2.0
Reinforced concrete pile bents • Vertical piles only • With batter piles	 1.5 1.5	 2.0 1.5	 3.0 2.0
Single columns	1.5	2.0	3.0
Steel or composite steel and concrete pile bents • Vertical piles only • With batter piles	 1.5 1.5	 3.5 2.0	 5.0 3.0
Multiple column bents	1.5	3.5	5.0

Table 2.3-10 Response Modification Factors – Connections

Connection	All Operational Categories
Superstructure to abutment	0.8
Expansion joints within a span of the superstructure	0.8
Columns, piers, or pile bents to cap beams or superstructure	1.0
Columns or piers to foundations	1.0

CHAPTER 3: BASIC KNOWLEDGE OF EARTHQUAKE ENGINEERING AND STRUCTURAL DYNAMICS

Chapter 3 Basic Knowledge of Earthquake Engineering and Structural Dynamics.

3.1 Basic Knowledge of Earthquake Engineering

3.1.1 General

The basic knowledge of earthquake engineering required for bridge design is introduced in this Chapter. The introduced knowledge may be minimum and limited. Therefore, it is recommended that further reference is made to earthquake engineering related books for more detailed information or inquiry.

3.1.2 Causes of Earthquake

According to the definition in seismology, an earthquake is a phenomenon of ground shaking caused by movement at the boundary of tectonic plates of the Earth's crust by the sudden release of stress. The edges of tectonic plates are made by trench (or fractures or fault). Most earthquakes occur along the trench lines when the plates slide past each other or collide against each other.

There are mainly two types or causes of earthquake. One is caused by the movement of tectonic plate of the Earth's crust and the other is caused by the movement of active faults in the continental plate. There is another type called volcanic earthquake, in which the magma stored in reservoirs moves upwards, fractures the rock, and squeezes through, causing earthquakes usually with magnitudes not much significant.

The major characteristics of the two main types of earthquake and the location of plate boundaries and active faults in the Philippines defined by PHIVOLCS (Philippine Institute of Volcanology and Seismology) are shown in **Table 3.1-1.**

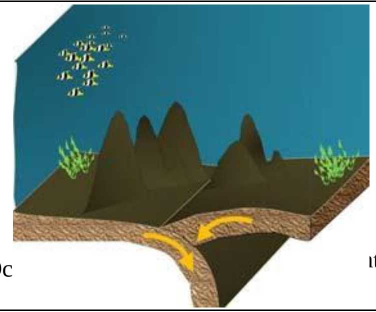
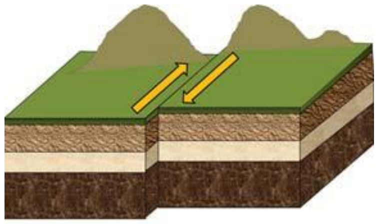
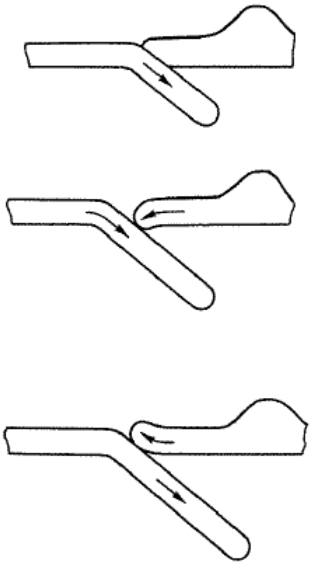
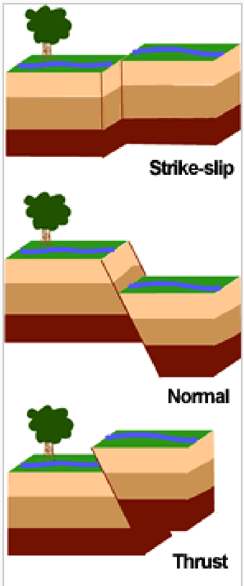
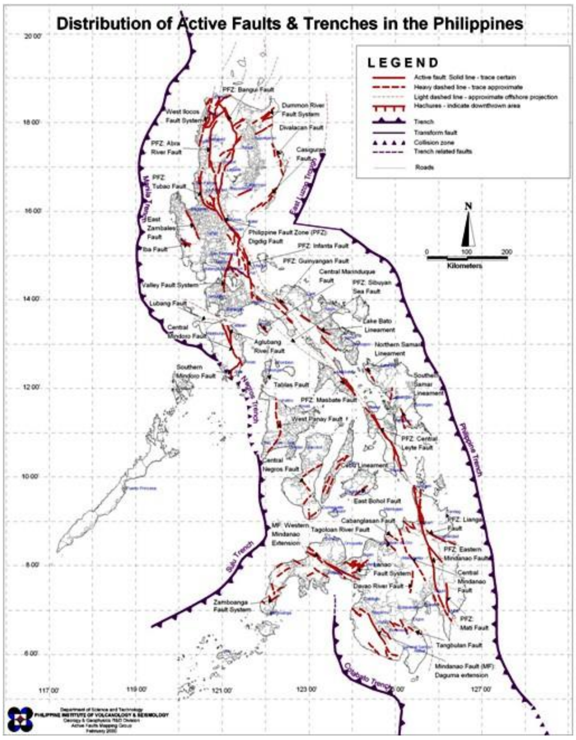
Plate Boundary Type of Earthquake

Most earthquakes occur along the edge of the oceanic and continental plates. The Earth's crust is made up of several plates. The plates under the oceans are called oceanic plates and the rest are continental plates. The plates are moved around by the motion of a deeper part of the mantle that lies underneath the crust. These plates are always bumping into each other, pulling away from each other, or past each other. Earthquakes usually occur where two plates run into each other or slide past each other.

Active Fault Type of Earthquake

Earthquakes can also occur far from the edges of plates, along active faults. Active faults are cracks in the earth where sections of a plate move in different directions. Active faults are caused by all that bumping and sliding the plates do. There are three main types of active fault movement which may cause an earthquake, namely; normal fault, reverse (thrust) fault and strike-slip fault.

Table 3.1-1 Types of Earthquake

Type	Plate Boundary Earthquake	Active Fault Earthquake
Image	Oc  ıtal	
Mechanism of Occurrence of Earthquake	(ate  (a) O‹ (b) C an (c) The stress of continental plate is released	
Locations of Plate Boundary and Active Faults in the Philippines	 (Source: PHIVOLCS)	

3.1.3 Velocity and Transmission of Seismic Wave

There are several kinds of seismic wave, and they all move in different directions as shown in **Figure 3.1-1**. When the seismic wave is transmitted in bedrock or the ground, the amplitude becomes small. The phenomenon of decrement of the seismic wave is called dumping.

The two main types of waves are “body waves” and “surface waves”. Body waves can travel through the Earth’s crust, but surface waves can only move along the surface of the ground. Traveling through the Earth’s crust, body waves arrive before the surface waves emitted by an earthquake. The body waves are of a higher frequency than surface waves. The transmission of each kind of seismic wave is explained in **Table 3.1-1.**

Body Wave (P Wave and S Wave)

The first kind of body wave is the primary wave (P wave). This is the fastest seismic wave, and consequently the first to arrive at a seismic station. P waves are also known as compressional waves. Subjected to a P wave, particles move in the same direction that the wave is moving in, which is the direction that the energy is traveling.

The other type of body wave is the secondary wave (S wave). An S wave is slower than a P wave and can only move through solid rock. S waves move rock particles up and down, or from side-to-side perpendicular to the direction that the wave is traveling. Travelling only through the crust, surface waves are of a lower frequency than body waves. Though they arrive after body waves, it is surface waves that are almost entirely responsible for the damage and destruction associated with earthquakes. This damage and the strength of the surface waves are reduced in deeper earthquakes.

Surface Wave (Love Wave and Rayleigh Wave)

The two main types of surface waves are “Love wave” and “Rayleigh wave”. Love wave is the fastest surface wave and moves the ground from side-to-side. Confined to the surface of the Earth’s crust, Love waves produce entirely horizontal motion.

Rayleigh wave rolls along the ground just like a wave rolls across a lake or an ocean. Since this wave rolls, it moves the ground up and down and from side-to-side in the same direction that the wave is moving. Most of the shaking felt from an earthquake is due to Rayleigh wave, which can be much larger than the other waves.

The velocity of a seismic wave depends on the density or hardness (modulus of elasticity) of the ground material. The velocity of P wave at ground surface is approximately 5 to 6 km/sec and the velocity of S wave is approximately 3 to 4 km/sec, that is, 60-70% of P wave. Surface wave is slightly slower than S wave. All waves are transmitted from the epicenter at the same time as an earthquake occurs.

However, the time lag of arrival of P waves and S waves become big depending on the distance from the epicenter as shown in **Figure 3.1-2.** This time lag is called as S-P time or duration of preliminary tremors. When the duration of preliminary tremors (sec) is multiplied by 8, it becomes the distance (km) to the epicenter. For example, if the duration of a preliminary tremor is 10 seconds, the distance to the epicenter could be evaluated at approximately 80 km.

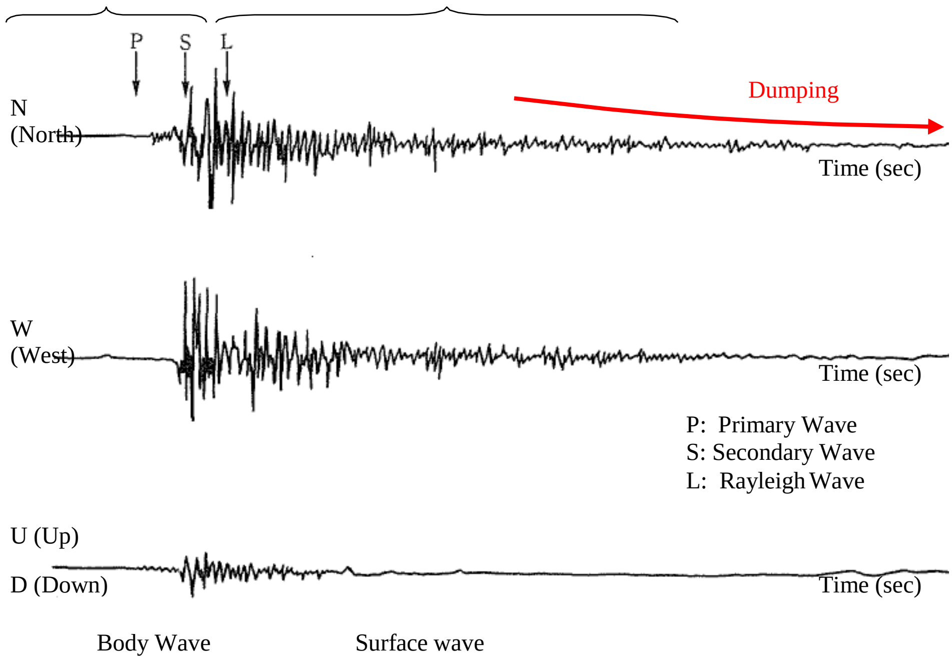


Figure 3.1-1 Example of Seismic Wave in Different Directions

Table 3.1-2 Kinds of Seismic Wave Transmission

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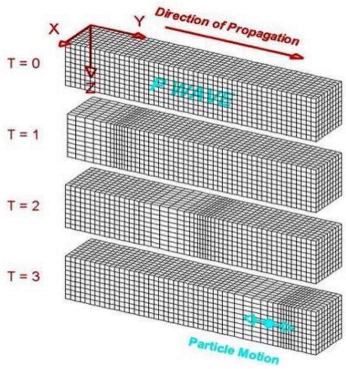
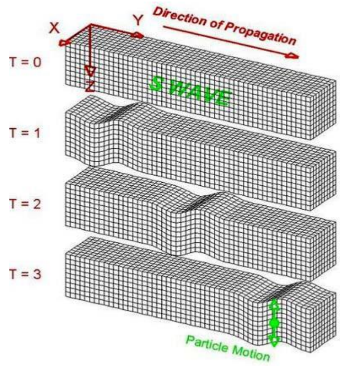
<tr><td>Primarv Wave (P</td><td>Secondarv Wave (S</td></tr>
<tr><td>
</td><td>
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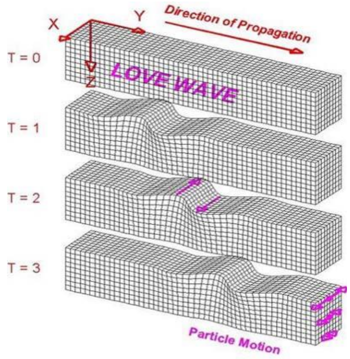
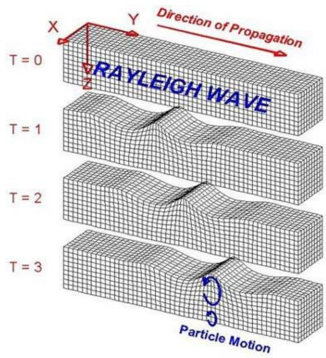
<tr><td>Love</td><td>Ravleigh</td></tr>
<tr><td>
</td><td>
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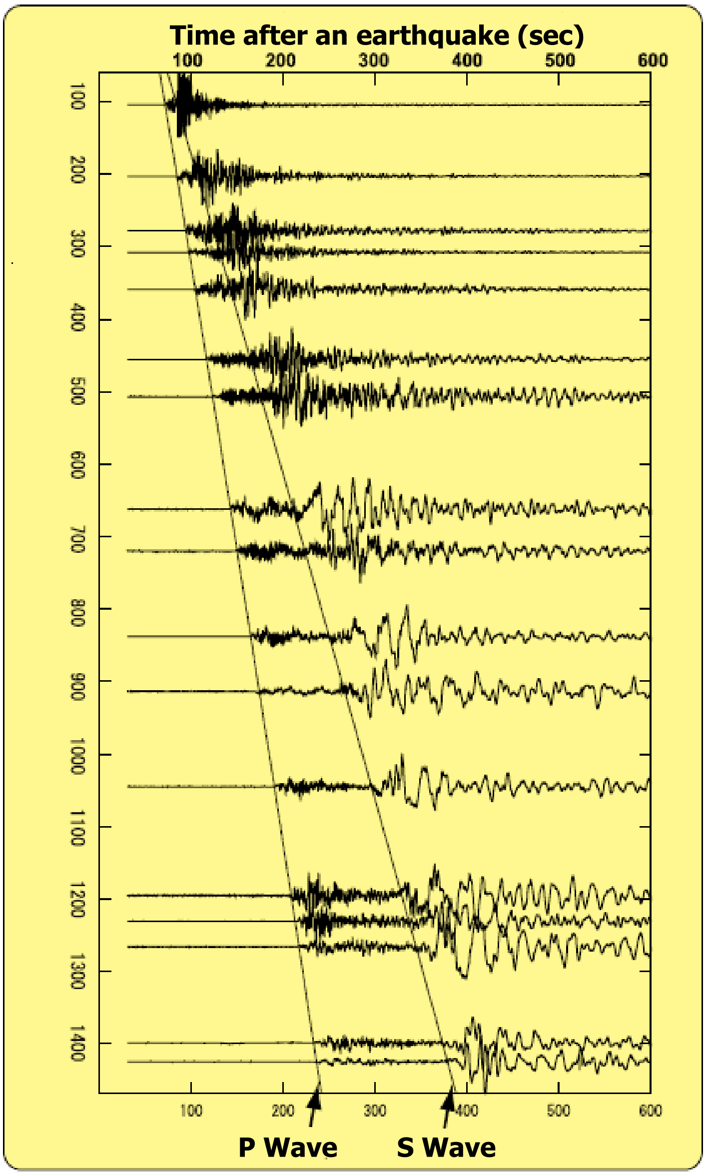
(Source: Michigan Tech, Geological and Mining Engineering and Science, http://www.geo.mtu.edu/UPSeis/index.html23)



(Source: Sapporo District Meteorological Observatory,
http://www.jma-net.go.jp/sapporo/knowledge/jikazanknowledge/jikazanknowledge2_2.html)

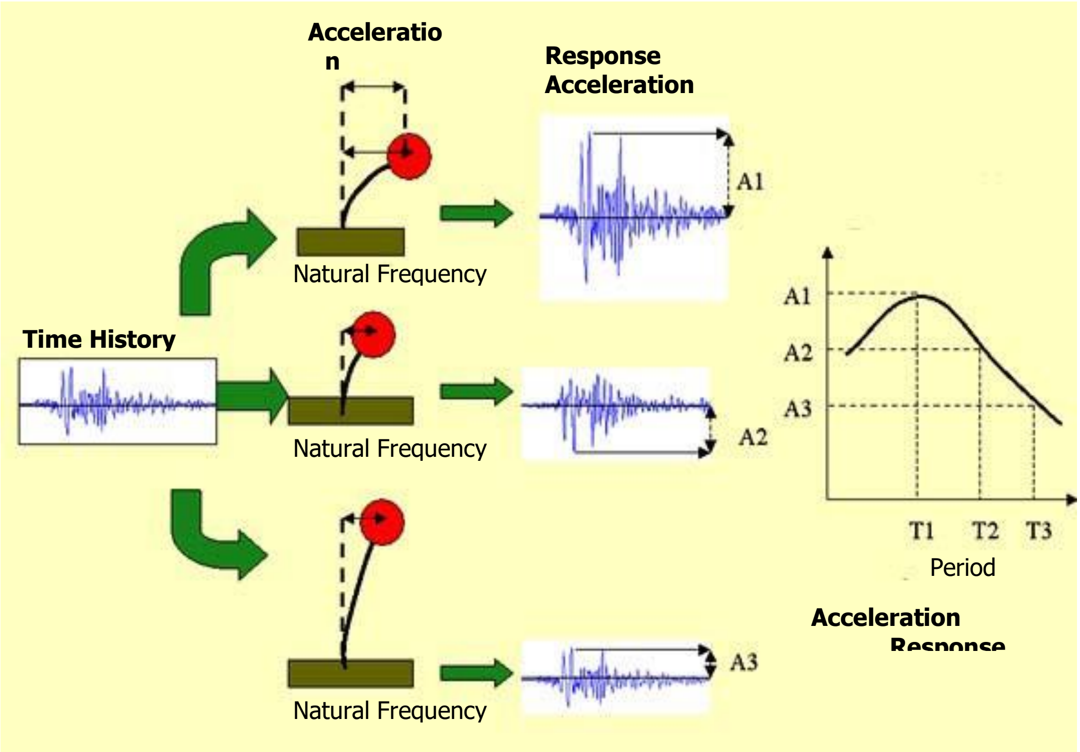
Figure 3.1-2 Example of Seismic Wave Transmission to Different Locations

3.1.4 Time History Wave and Spectrum of Earthquake

Several period waves are contained in a time history earthquake wave. A time history earthquake wave could be recomposed into each period by its intensity, and its transform is called Fourier spectrum. However, it is difficult to find the influence to the structure during earthquake by the observation of Fourier spectrum. A better method to understand its behavior is to use response spectrum.

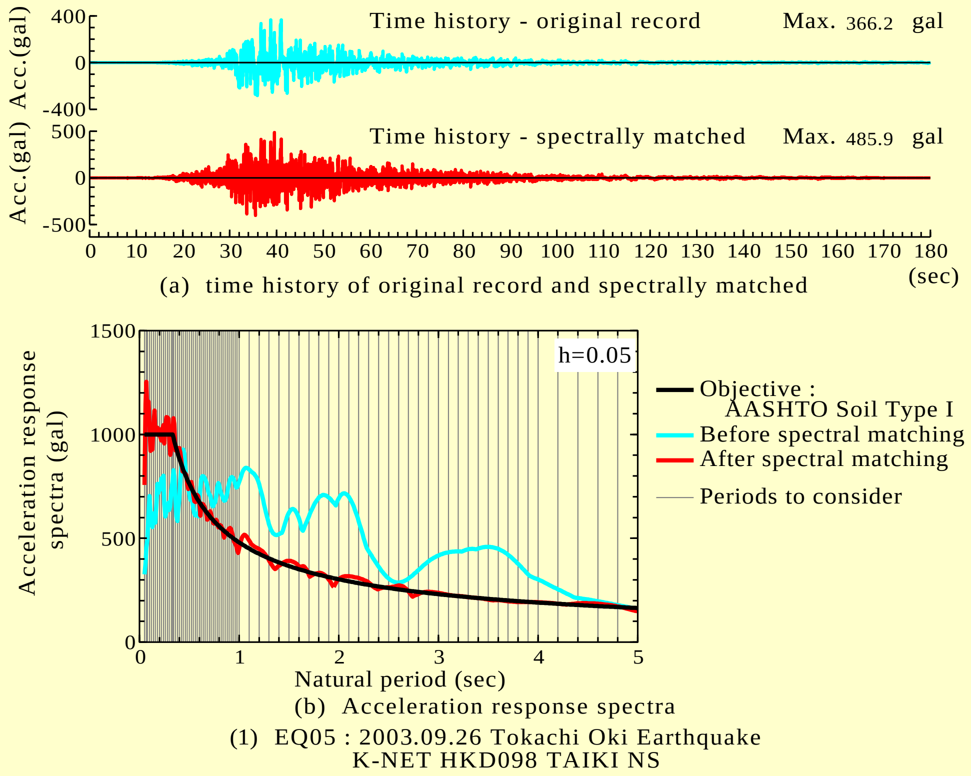
As shown in **Figure 3.1-3**, a response spectrum is simply a plot of the peak of a series of steady-state response with single-degree-of-freedom system varying natural frequency that are forced into motion by the same base vibration. The resulting plot can then be used to pick off the response of any linear system, given its natural frequency of vibration. In the case of acceleration, the response spectrum is called an acceleration response spectrum.

Figure 3.1-4 shows an example of transformation of the acceleration response spectrum from the observed time history wave of a previous earthquake in Japan. The figures include the matching of the target response spectrum by modification of time history wave.

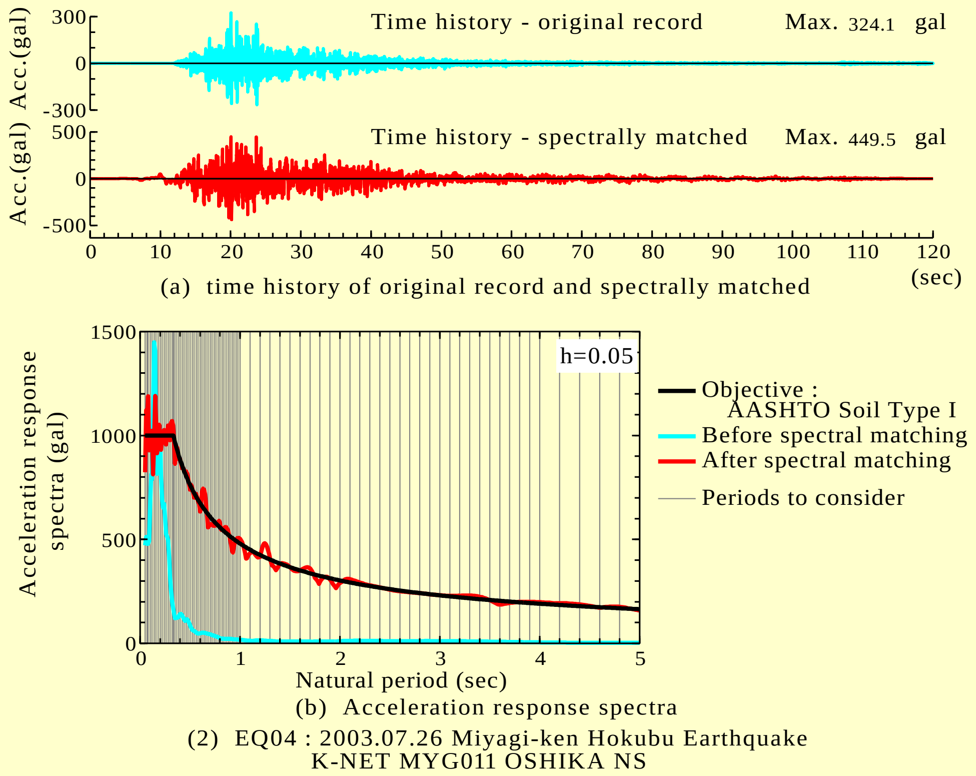


(Source: Japan Meteorological Agency)

Figure 3.1-3 Procedure of Transformation of Response Spectrum from Time History Wave



(1) Tokachi Oki Earthquake, 2003



(2) Miyagi-ken Hokubu Earthquake, 2003

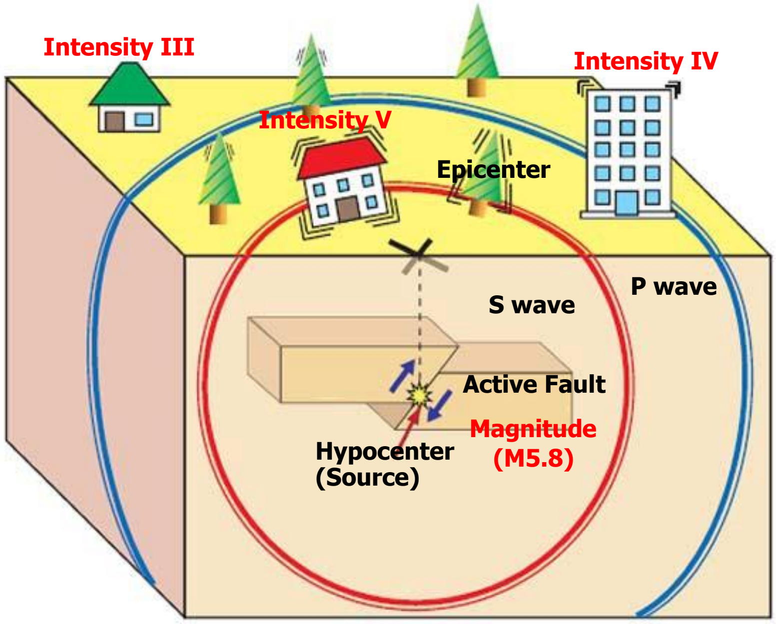
(Source: JICA Study Team)

Figure 3.1-4 Example of Time History Earthquake Wave and Acceleration Response Spectrum

3.1.5 Intensity of Earthquakes (Magnitude, Seismic Intensity Scale and Engineering Seismic Coefficient)

(1) General

Basically, an earthquake is measured by its Magnitude and Intensity. The Magnitude indicates the amount of energy released at the source of one earthquake and is measured by the Magnitude Scale. The intensity of an earthquake at a particular locality indicates the violence of earth motion produced there by the earthquake. It is determined from reported effects of the tremor on human beings, furniture, buildings, geological structure, etc. In the Philippines, the PHIVOLCS Earthquake Intensity Scale (PEIS) is adopted, which classifies earthquake effects into ten scales. When an earthquake occurs, its magnitude can be given a single numerical value by the Magnitude Scale. However, the intensity is variable over the area affected by the earthquake, with high intensities near the epicenter and lower values further away. These are allocated a value depending on the effects of the shaking according to the Intensity Scale.



(Source: Sapporo District Meteorological Observatory, http://www.jma-net.go.jp/sapporo)

Figure 3.1-5 Deference between Magnitude and Intensity

(2) Magnitude Scale

Richter Magnitude Scale

In 1935, Charles Richter and Beno Gutenberg developed the local magnitude scale (Ml), which is popularly known as the Richter magnitude scale, to quantify medium-sized earthquakes between magnitude 3.0 and 7.0. This scale was based on the ground motion measured by a particular type of seismometer at a distance of 100 km from the earthquake's epicenter. For this reason, there is an upper limit on the highest measurable magnitude, and all large earthquakes will tend to have a local magnitude of around 7. Since this Ml scale was simple to use and corresponded well with the damage which was observed, it was extremely useful for engineering earthquake-resistant structures and gained common acceptance.

Moment Magnitude Scale (Mw)

The moment magnitude scale (Mw) is used by seismologists to measure the size of earthquake in terms of the energy released. The magnitude is based on the seismic moment of the earthquake, which is equal to the rigidity of the Earth multiplied by the average amount of slip on the fault and the size of the area that slipped. The scale was developed in the 1970's to succeed the 1930's Richter magnitude scale (Ml). Even though the formulae are different, the new scale retains the familiar continuum of magnitude values defined by the older one. The Mw is now the scale used to estimate magnitude for all modern large earthquakes by the United States Geological Survey (USGS).

(3) Seismic Intensity Scale

The Philippine Institute of Volcanology and Seismology (PHIVOLCS) is the government agency that is monitoring earthquakes that affect the Philippines. PHIVOLCS provided the earthquake intensity scale to determine the destructiveness of earthquake, as shown in **Table 3.1-3**

Table 3.1-3 PHIVOLCS Earthquake Intensity Scale (PEIS)

Scale		PGA (g values)	Description
I	Scarcely Perceptible	0.0005	Perceptible to people under favorable circumstances. Delicately balanced objects are disturbed slightly. Still water in containers oscillates slowly.
II	Slightly Felt	0.0009	Felt by few individuals at rest indoors. Hanging objects swing slightly. Still water in containers oscillates noticeably.
III	Weak	0.0011	Felt by many people indoors especially in upper floors of buildings. Vibration is felt like one passing of a light truck. Dizziness and nausea are experienced by some people. Hanging objects swing moderately. Still water in containers oscillates moderately.
IV	Moderately Strong	0.0050	Felt generally by people indoors and by some people outdoors. Light sleepers are awakened. Vibration is felt like a passing of heavy truck. Hanging objects swing considerably. Dining plates, glasses, windows and doors rattle. Floors and walls of wood framed buildings creak. Standing motor cars may rock slightly. Liquids in containers are slightly disturbed. Water in containers oscillates strongly. Rumbling sound may sometimes be heard.
V	Strong	0.0100	Generally felt by most people indoors and outdoors. Many sleeping people are awakened. Some are frightened, some run outdoors. Strong shaking and rocking felt throughout building. Hanging objects swing violently. Dining utensils clatter and clink; some are broken. Small, light and unstable objects may fall or overturn. Liquids spill from filled open containers. Standing vehicles rock noticeably. Shaking of leaves and twigs of trees are noticeable.
VI	Very Strong	0.1200	Many people are frightened; many runs outdoors. Some people lose their balance. Motorists feel like driving in flat tires. Heavy objects or furniture move or may be shifted. Small church bells may ring. Wall plaster may crack. Very old or poorly built houses and man-made structures are slightly damaged though well-built structures are not affected. Limited rock-falls and rolling boulders occur in hilly to mountainous areas and escarpments. Trees are noticeably shaken.

VII	Destructive	0.2100	Most people are frightened and run outdoors. People find it difficult to stand in upper floors. Heavy objects and furniture overturn or topple. Big church bells may ring. Old or poorly-built structures suffer considerable damage. Some well-built structures are slightly damaged. Some cracks may appear on dikes, fish ponds, road surface, or concrete hollow block walls. Limited liquefaction, lateral spreading and landslides are observed. Trees are shaken strongly. (Liquefaction is a process by which loose saturated sand lose strength during an earthquake and behave like liquid).
VIII	Very Destructive	0.3600-0.5300	People panicky. People find it difficult to stand even outdoors. Many well-built buildings are considerably damaged. Concrete dikes and foundation of bridges are destroyed by ground settling or toppling. Railway tracks are bent or broken. Tombstones may be displaced, twisted or overturned. Utility posts, towers and monuments mat tilt or topple. Water and sewer pipes may be bent, twisted or broken. Liquefaction and lateral spreading cause man-made structures to sink, tilt or topple. Numerous landslides and rock-falls occur in mountainous and hilly areas. Boulders are thrown out from their positions particularly near the epicenter. Fissures and fault-rapture may be observed. Trees are violently shaken. Water splash or top over dikes or banks of rivers.
IX	Devastating	0.7110-0.8600	People are forcibly thrown to ground. Many cry and shake with fear. Most buildings are totally damaged. Bridges and elevated concrete structures are toppled or destroyed. Numerous utility posts, towers and monument are tilted, toppled or broken. Water sewer pipes are bent, twisted or broken. Landslides and liquefaction with lateral spreading and sand-boils are widespread. The ground is distorted into undulations. Trees shake very violently with some toppled or broken. Boulders are commonly thrown out. River water splashes violently on slops over dikes and banks.
X	Completely Devastating	1.1500<	Practically all man-made structures are destroyed. Massive landslides and liquefaction, large scale subsidence and uplifting of land forms and many ground fissures are observed. Changes in river courses and destructive seethes in large lakes occur. Many trees are toppled, broken and uprooted.

(Source: PHIVOLCS)

(4) Engineering Seismic Coefficients

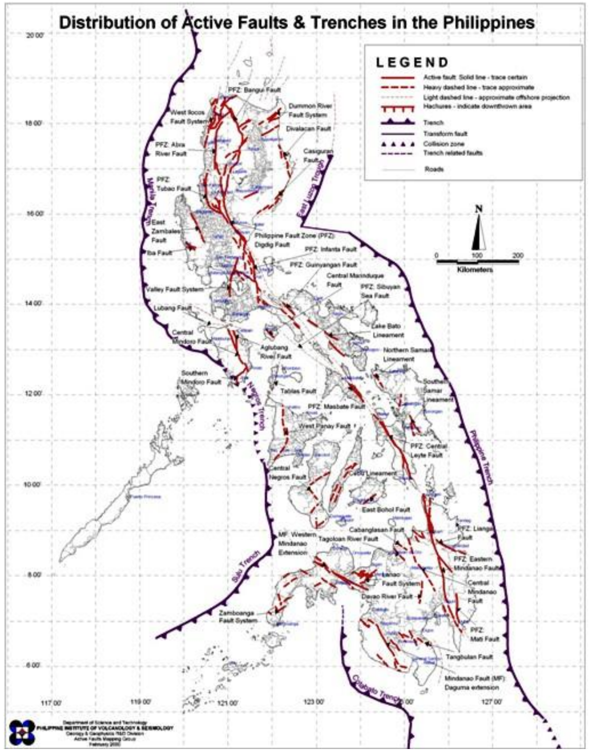
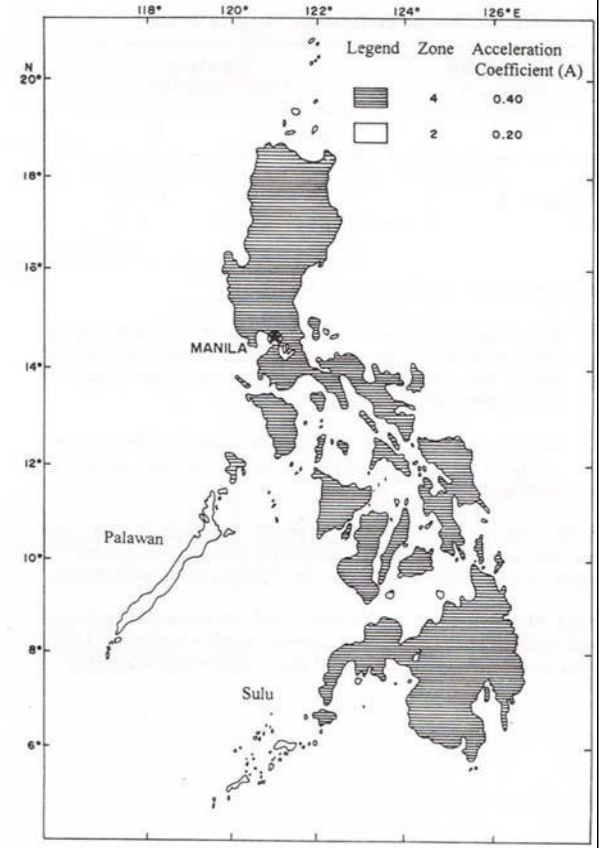
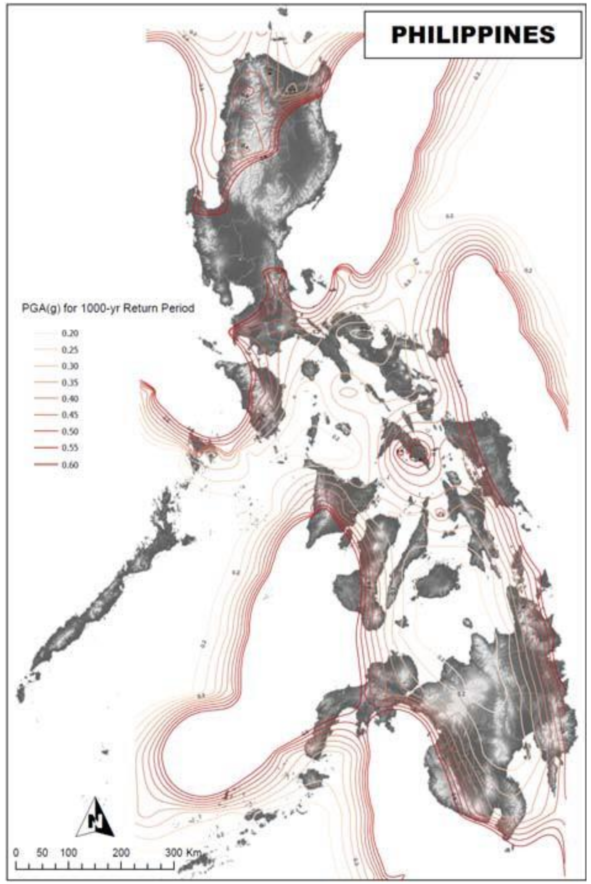
The PHIVOLCS Earthquake Intensity Scale (PEIS) described above show the destructivity impact of earthquakes qualitatively. However, PEIS is not used for bridge seismic design. The bridge seismic design expresses the strength of earthquake by the seismic coefficient or Peak Ground Acceleration (PGA) of the ground surface.

The seismic coefficient of bridge seismic design (k) is formulated as follows, expressing the ratio between the maximum acceleration of the ground surface (α) and the acceleration of gravity (g).

$$k = \frac{\alpha}{g} = \frac{\alpha(\text{gal})}{980(\text{gal})} \approx \frac{\alpha}{1000}$$

Table 3.1-4, shows a comparison including (1) the location of existing trench and fault, which was shown in Error! Reference source not found.; (2) the seismic zone map, which is currently used by DPWH for bridge seismic design with acceleration coefficient (A) of 0.40, except for Palawan with A = 0.20; and (3) the proposed Peak Ground Acceleration (PGA) map provided by the Project.

Table 3.1-4 Comparison of Seismic Intensity for Bridge Design

		
(Source: PHIVOLCS)	(Source: 1997, 2nd Edition of NSCP Vol. 2 – Bridges, 1997, ASEP)	(Source: JICA Study Team)
(1) Location of Plate Boundary and Active Fault in the Philippines	(2) Philippine Seismic Zone Map (Currently applied for Bridge seismic design)	(3) Proposed Peak Ground Acceleration (g) for 1,000-year Return Period

3.2 Basic Knowledge of Structural Dynamics

3.2.1 General

The basic knowledge of structural dynamics required for bridge design is introduced in this Chapter. The introduced knowledge may be minimum and limited. Therefore, it is recommended that further reference is made to structural dynamics related books for more detailed information or inquiry.

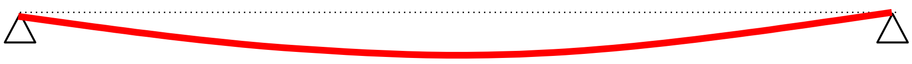
3.2.2 Characteristic Vibration of Structure and Seismic Load

Normal Mode

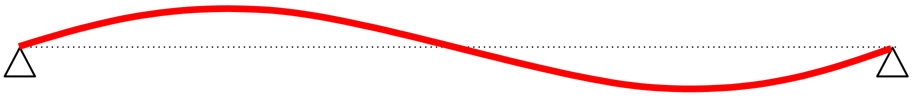
A Normal Mode is a pattern of motion in which all parts of the system move at the same frequency and with a fixed phase relation. The motion described by the normal mode is called resonance. The frequencies of the normal modes of a system are known as its natural frequencies or resonant frequencies. A physical object, such as a building, bridge, etc., has a set of normal modes that depend on its structure, materials and boundary conditions.

A mode of vibration is characterized by a modal frequency and a mode shape. It is numbered according to the number of half waves in the vibration. As shown in

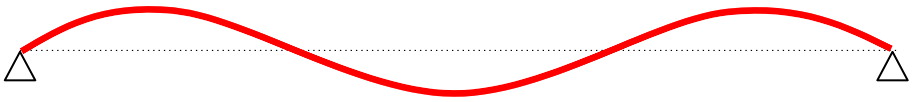
Figure *3.2-1*, if a vibrating beam with both ends pinned displayed a mode shape of half of a sine wave (one peak on the vibrating beam) it would be vibrating in Mode 1. If it had a full sine wave (one peak and one valley) it would be vibrating in Mode 2. **Figure 3.2-2** shows the case of cantilever such as bridge pier.



Mode 1 (Frequency: F1, Period: T1)



Mode 2 (Frequency: F2, Period: T2)



Mode 3 (Frequency: F3, Period: T3)

Figure 3.2-1 Example of Normal Modes of Beam

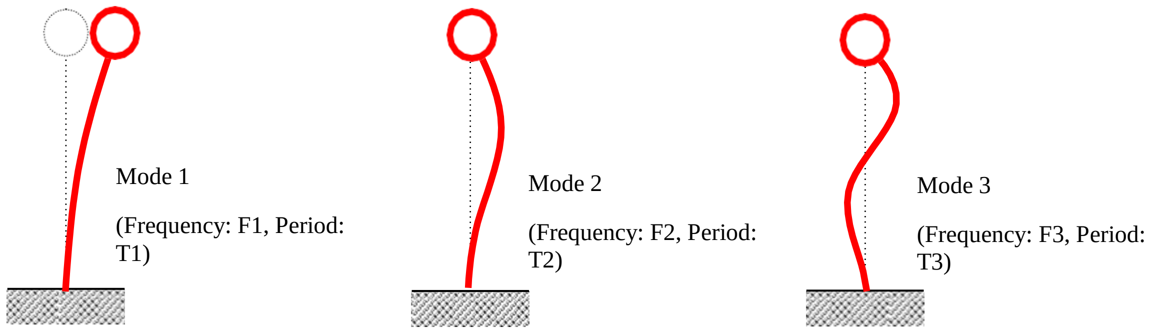


Figure 3.2-2 Example of Normal Modes of Cantilever

Resonance and Forced Vibration

In physics, resonance is the tendency of a bridge to vibrate with greater amplitude at some frequencies than at others. Frequencies at which the response amplitude is a relative maximum are known as the resonance frequencies. At these frequencies, even small periodic driving forces can produce large amplitude vibration, because the bridge stores vibration energy.

Forced vibration is a vibration caused forcibly by receiving the external force to fluctuate such as earthquakes. When the periods of forced vibration is the same or close to the natural frequency of the bridge, the vibration occurs remarkably. It is also called as resonance.

Acceleration Response Spectrum and Vibration Mode

The expected acceleration response of a bridge during earthquake is called as Acceleration Response Spectrum, which was explained in Section 1.4. The Design Response Spectrum for acceleration is developed, as shown in **Table 3.1-3** with site coefficient for Peak Ground Acceleration (PGA), 0.2-sec period spectral acceleration, and 1.0-sec period spectral acceleration in the Bridge Seismic Design Specification.

An example of calculation of Design Acceleration Spectrum is shown in **Figure 3.2-2.** The first natural period of ordinary bridge is basically short such as T1=0.5 (sec), which is defined with the strength of substructure and supported mass of superstructure. However, the natural period of high elevated bridge or bridges which adopt rubber bearings are longer than the ordinary bridge. In that case, the acceleration response can be estimated as smaller than that of ordinary bridge.

3.2.3 Material Non-linearity

The “linear” behavior could be defined as a property which could be “superposition relation” between the causes and effects. As an example, displacement of the vertical direction of bridge girder becomes large in proportion to the vertical load. In addition, as shown in **Figure 3.1-2**, the total displacement can be calculated by summing up the vertical displacement due to dead load, live load, etc. Its behavior could be called linear.

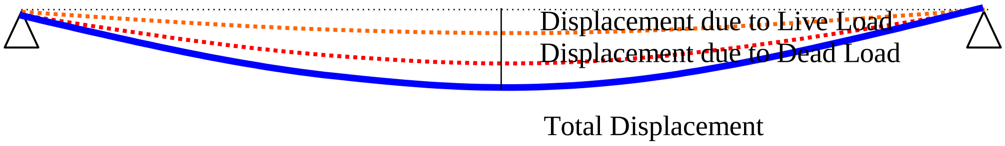


Figure 3.2-3 Example of Superposition Relation in Linear Property

On the other hand, non-linear means not linear in mathematical terms. In other words, it is the phenomenon that superposition relation is not formed. As an example, the reinforced concrete used in bridge construction (the stress-strain relation of reinforcing bar is as shown in **Figure 3.2-4**) does not appear to be on a straight line because the plastic deformation happens when the strain reaches the yield stress, and the strain grows after the yielding. Material non-linearity means that the straight line does not have stress and strain relationship in this way. However, it may be said that the above-mentioned superposition relationship is up to the yielding point of materials, because the stress-strain relation of reinforced bar is a straight line. The stress-strain relation of concrete is also non-linear when the strain of concrete is large as shown in Error! Reference source not found..

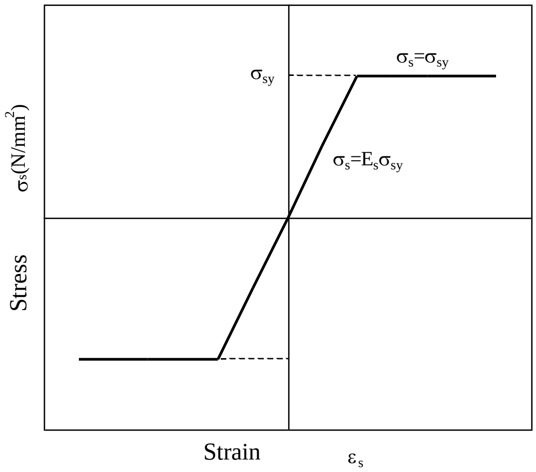


Figure 3.2-4 Ideal Stress-Strain Relation of Reinforcing Bar

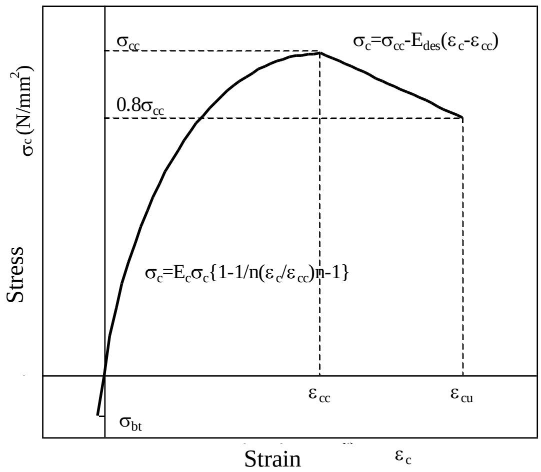


Figure 3.2-5 Ideal Stress-Strain Relation of Concrete

The material non-linearity is one of the important considerations for the seismic design, especially when large-scale earthquakes are considered because the material may behave in non-linear level.

The non-linearity horizontal force-displacement relation of reinforced concrete pier is shown in

Figure 3.2-6. The restitution force of reinforced concrete shall be considered when the bridge pier had suffered from a repetitive force such as a large-scale earthquake. Generally, the skeleton of repetitive force contains cracking of concrete, yielding of reinforced bar and, ultimately, compression of concrete in the tri-linear type of skeleton model such as Takeda Model. The stiffness of reinforced concrete is changed by major events such as cracking or yielding. When the bridge pier had behaved as non-linear, the residual displacement will remain after the earthquake.

Figure 3.2-7 shows an example of historical curve of the bending moment-curvature relation at pier bottom obtained by Non-linear time history response analysis.

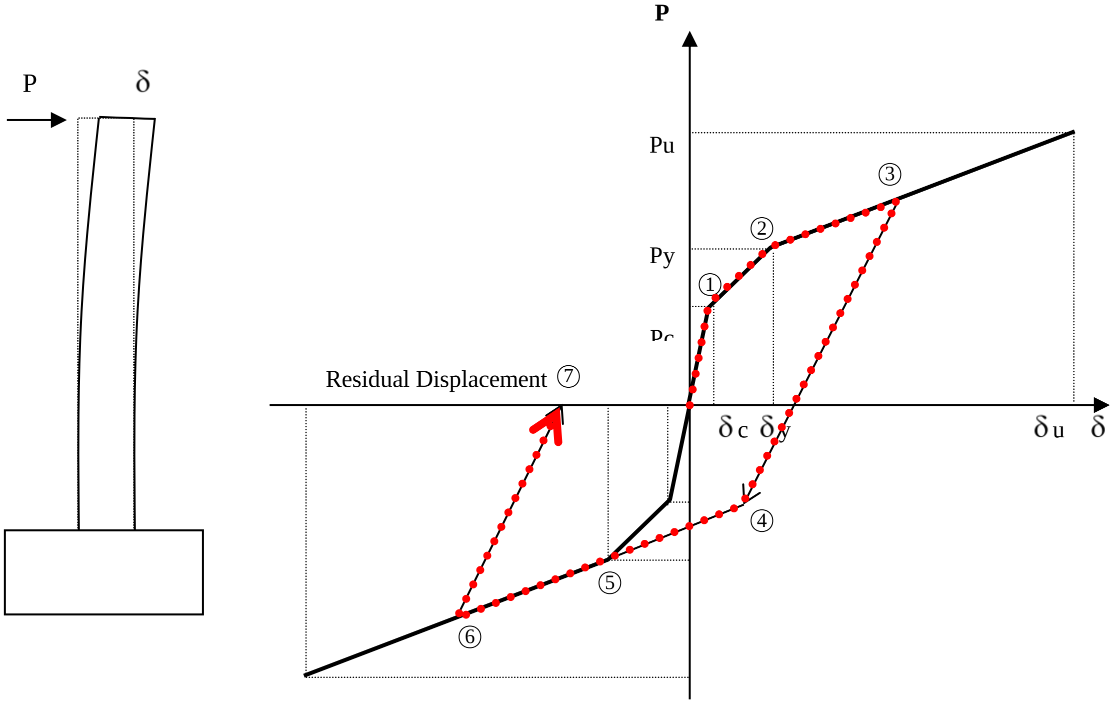


Figure 3.2-6 Non-linear Behavior of Reinforced Concrete Pier

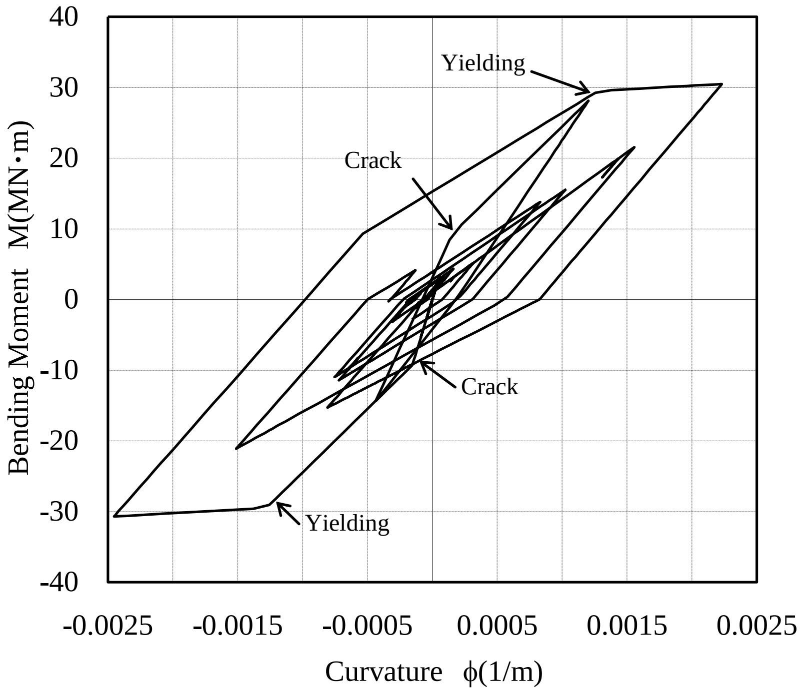


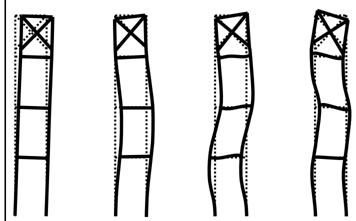
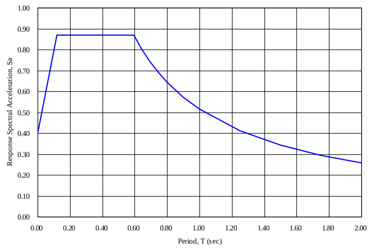
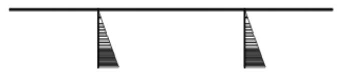
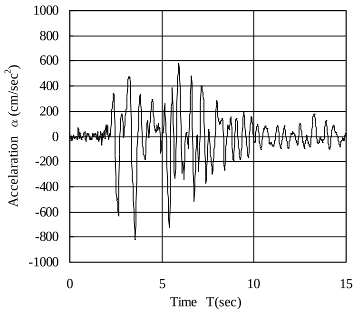
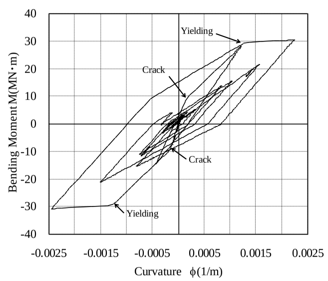
Figure 3.2-7 Example of Historical Curve of the Bending Moment-Curvature at Pier Bottom

3.2.4 Static Design and Dynamic Design Methods

The analysis method of seismic design is classified into static analysis and dynamic analysis. Since earthquake is a dynamic phenomenon and the response of a structure usually changes from time to time, dynamic analysis is desirable to use in the seismic design of bridges. However, if the behavior of the structure is not complicated, the static analysis has to be carried out, because the dynamic analysis is complicated.

The major dynamic analysis methods for bridge seismic design are shown in **Table 3.2-1.** These analysis methods have their own characteristics and the method shall be selected according to the type of bridge.

Table 3.2-1 Major Dynamic Analysis Methods for Bridge Seismic

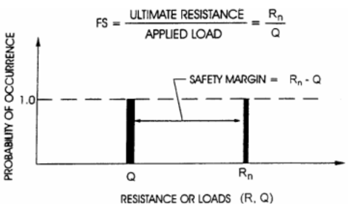
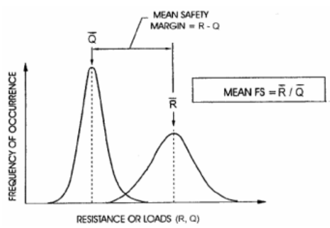
Analysis Method	Description	Analytic Model	Input Force for Seismic Design	Major Output	Remarks
Eigenvalue Analysis	Eigenvalue analysis is an analysis to obtain the vibration characteristics of the structure itself (natural period and mode shape). When the natural period of the structure is close to the distinction of seismic force, sympathetic vibration will occur. This may be caused by fatigue or the destruction of the bridge. It is important to identify the Eigenvalue to avoid sympathetic vibration of bridges during earthquake.	Linear	- N/A	-Natural Period -Mode Shape 	
Response Spectrum Analysis	The response spectrum shows a maximum response level (Acceleration, Velocity, etc.) of the single degree of freedom (SDOF) model (having one natural period and one dumping coefficient) during earthquake. The maximum response of the multi-degree of freedom (MDOF) model could be estimated to sum up multi-modal response. This is Response Spectrum Analysis.	Linear	- Acceleration Response Spectrum 	- Maximum Response (Acceleration, Velocity, Displacement and Section Forces, etc.) 	- Eigenvalue Analysis is required -Non-linear behavior is not considered
Time History Response Analysis	Time history response analysis has two kinds of analytical method, namely; “time history modal analysis method” and “direct numerical integration method”. The direct numerical integration method is usually used in bridge seismic design exercises on the response of structure by direct numerical integration. The stiffness matrix could be changed at every step of calculation on the direct numerical integration. Therefore, this method is popularly used for the non-linear analysis, such as a complicated structure including high frequency vibration modes.	Linear/ Non-linear	- Time History Seismic Wave 	- Time History Response (Acceleration, Velocity, Displacement and Section Forces, etc.) 	- Following integral calculus method are usually employed, Newmark Wilson θ β Runge-Kutta

3.2.5 Load Factor Design (LFD) and Load and Resistance Factor Design (LRFD)

In 1994, the first edition of the "AASHTO LRFD Bridge Design Specifications" was published, placing earthquake loading under Extreme Event I limit state. Similar to the 1992 edition, the LRFD edition accounts for column ductility using the response modification R factors. In 2008, the "AASHTO LRFD Interim Bridge Specifications" was published to incorporate more realistic site effects based on the 1989 Loma Prieta earthquake in California. Moreover, the elastic force demand is calculated using the 1,000-year maps as opposed to the earlier 500-year return earthquake.

The comparison of ASD (WSD), LFD and LRFD is shown in **Table 3.2-2**

Table 3.2-2 Comparison of ASD, LFD and LRFD

Design Method	**ASD: Allowable Stress Design** (WSD: Working Stress Design)	**LFD: Load Factor Design** (Strength Design)	**LRFD: Load and Resistance Factor Design** (Reliability Based Design/ LSD: Limit State Design)
Description	A method where the nominal strength is divided by a safety factor to determine the allowable strength. This allowable strength is required to equal or exceed the required strength for a set of ASD load combinations. 	LFD is a kind of the so-called Limit State Design (LSD) method. The limit state is a condition of a structure beyond which it no longer fulfills the relevant design criteria. The condition may refer to a degree of loading or other actions on the structure, while the criteria refer to structural integrity, fitness for use, durability or other design requirements. LSD requires the structure to satisfy three principal criteria: the Ultimate Limit State (ULS), the Serviceability Limit State (SLS) and the Fatigue Limit State (FLS).	The LRFD method subdivides the limit state of the structure compared to the LFD method. In addition, load factor and resistance factor are modified based on probability statistics data from a combination of limit state of various loads. The LRFD method modifies three equivalents to LFD method, such as Service Limit State, Fatigue & Fractural Limit State, Strength Limit State and Extreme Event Limit State, and the coefficient is changed. 
Basic Equation	$\Sigma DL + \Sigma LL \le R_u / FS$ where, FS: Factor of Safety	$\gamma(\Sigma\beta_{DL} DL + \Sigma\beta_{LL} LL) \le \phi R_u$ where, γ : Load Factor β : Load Combination Coefficient ϕ : Resistance Factor	$\eta(\Sigma\gamma_{DL} DL + \Sigma\gamma_{LL} LL) \le \phi R_u$ where, η : Load modifier γ : Load Factor ϕ : Resistance Factor
Advantage	- Simplistic	- Load factor applied to each load combination - Types of loads have different levels of uncertainty	- Accounts for variability - Uniform levels of safety - Risk assessment based on reliability theory
Limitation	- Inadequate account of variability - Stress not a good measure of resistance - Factor of Safety is subjective - No risk assessment based on reliability theory	- More complex than ASD - No risk assessment based on reliability theory	- Requires availability of statistical data - Resistance factors vary - Old habits

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CHAPTER 4: ANALYSIS METHOD

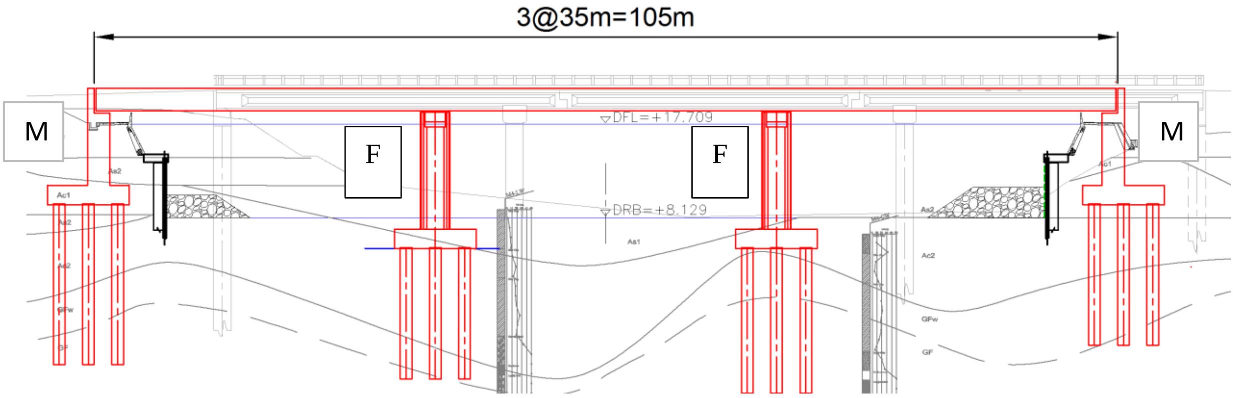
Chapter 4 Analysis Method

The primary purpose of this chapter is to present dynamic method for analyzing bridge structures when subjected to earthquake load. Basic concepts and assumptions were used in the following sample applications.

4.1 Simplified Method

4.1.1 Uniform Load Method

The uniform load method is essentially an equivalent static method that uses the uniform lateral load to compute the effect of seismic loads. For simple bridge structures with relatively straight alignment, small skew, balanced stiffness, relative light substructure, and with no hinges, uniform load method may be applied to analyze the structure for seismic loads. This method is not suitable for bridges with stiff substructures such as pier walls. This method assumes continuity of the structure and distributes earthquake force to all elements of the bridge and is based on the fundamental mode of vibration in either longitudinal or transverse direction (AASHTO,2012). The period of vibration is taken as that of an equivalent single mass-spring oscillator. The maximum displacement that occurs under the arbitrary uniform load is used to calculate the stiffness of the equivalent spring. The seismic elastic response coefficient C_{sm} or the Acceleration Response Spectrum ARS curve is then used to calculate the equivalent uniform seismic load using, which the displacements and forces are calculated. The following steps outline the uniform load method:



AASHTO Method

The following steps outline the uniform load method:

1. Idealize the structure into a simplified model and apply a uniform horizontal load P_o over the length of the bridge as shown in Figure above. It has units of force/unit length and may be arbitrarily set equal to 1 kN/m.

   p_0=1.0KN/m
   L=105/2=52.5m

2. Calculate the static displacements v_{smax} under the uniform load po using static analysis.

$$v_{s\,Max} = \frac{Wuh^3}{3EI} = P_0 L \times h^3/(3EI) = 1\times 52.5\times 14.8\text{^}3/(3\times 81{,}400{,}000) = 0.0007\text{m}$$

3. Calculate bridge lateral stiffness K.

$$K = \frac{p_0 L}{v_{s\,Max}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \text{Eq 4.1.1}$$

K=1×52.5/0.0007=75000KN/m

4. Calculate the total weight *W* of the structure including structural elements and other relevant Loads.

$$W=\int w(x)dx \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \text{Eq 4.1.2}$$

5. Calculate the period of the structure T_n using the following equation:

$$T_m = 2\pi\sqrt{\frac{W}{gK}} = 2\times\pi\sqrt{}(8767.5/9.8/75000) = 0.685 \text{ sec}$$

6. Calculate the equivalent static earthquake force pe using the ARS curve.

$P_e = C_{sm}W/L$ Equivalent static seismic loading per unit length

From following Spectrum,

$C_{sm}=S_{D1}/T_m=0.64/0.685=0.93$

P_e = 0.93*8767.5/52.5=155.3 KN/m

JRA Method

$$v_{s\,Max} = \frac{Wuh^3}{3EI} + \frac{0.8W_p h_p{}^3}{8EI} \quad \ldots\ldots\ldots\ldots\ldots \text{JRA}$$

Wu: Dead load of superstructure = 167KN / m × (35 + 35 / 2) = 8767.5KN

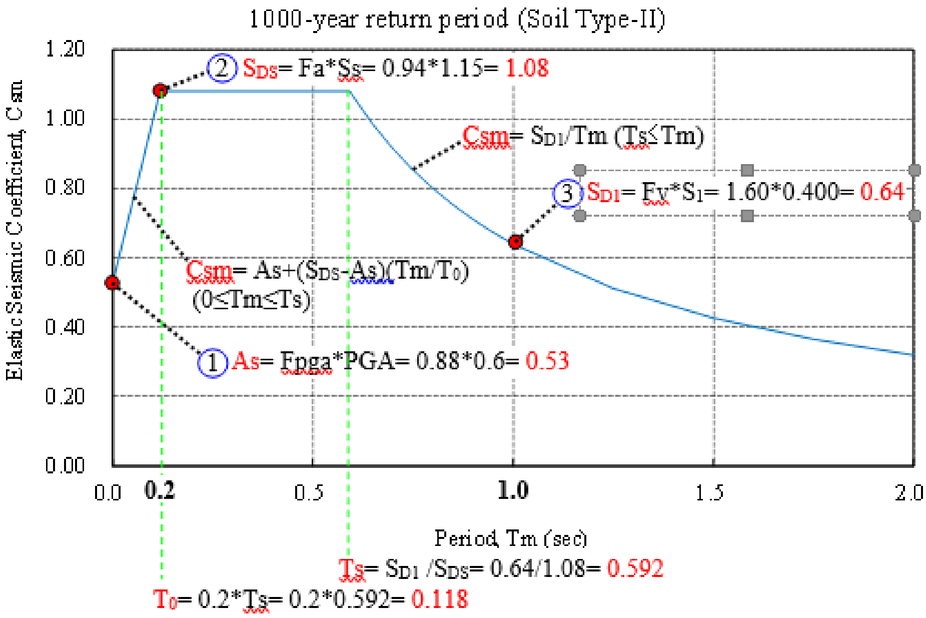
Wp: Dead load of pier = (11×3.14/4×2.8^2+2.9× ((2.8+9.1) / 2×0.75+1.25×9.1)) ×24
= 2829KN

EI = 27,000,000KN/m2 × (3.14×2.8^4)/64=81,400,000KN・m2

$V_{S\,Max}$ = 8767.5 × 14.8^3 / (3 × 81,400,000) + 0.8×2829×13.0^3 / (8 × 81,400,000)
= 0.116 + 0.008 = 0.123m

K = (8767.5 + 2829) / 0.123 = 94280KN/m

$$T_m = 2\pi\sqrt{\frac{W}{gK}} = 2\times\pi\times\sqrt{}\ (8767.5 + 2829) / 9.8 / 94280 = 0.70\text{sec}$$



4.1.2 Single Mode Spectral Method

The single-mode spectral analysis is based on the assumption that earthquake design forces for structures respond predominantly in the first mode of vibration. This method is most suitable to regular linear elastic bridges to compute the forces and deformations, but not applicable for irregular bridges (unbalanced spans, unequal in the columns, etc.) because higher modes of vibration affect the distribution of the forces and resulting displacements significantly (Chen 2014). This method can be applied to both continuous and noncontinuous bridge superstructures.

The inertial forces $p_e(x)$ are calculated using the natural period and the design forces and displacement are then computed using static analysis as shown in the example below.

(1) Longitudinal Direction

Displacement due to $P_0 = 1$ KN/m.

Superstructure uniformly 0.00157m

$\alpha = \int v(x)dx = 0.00157*105 = 0.165$

$\beta = \int w(x)v(x)dx = \int 167 * 0.00157dx = 27.5$

$\gamma = \int w(x)v(x)^2dx = \int 167 * 0.00157^2 dx = 0.043$

$T_m = 2\pi\sqrt{\ }(\gamma / (p_0*g*\alpha) = 2*3.14\sqrt{\ }(0.043 / 1.0*9.8*0.165) = 1.02$sec

$p_e(x) = \beta C_m / \gamma * w(x)v(x) = 27.5*1.0/0.043*167*0.00157 = 167$KN

(2) Transverse Direction

Refer to Excel sheet

$\alpha = 0.016$

$\beta = 7.59$

$\gamma = 2.33\times10^{-6}$

$T_m = 2\pi\sqrt{\ }(.00405084 / (1.0*9.8*0.016) = 1.0$ sec

$p_e(x) = \beta C_m / \gamma * w(x)v(x) = 7.59*0.53/2.33\times10^{-6} * v(x)$

$v(x) = Dy$ (on next page)

Table 4.1-1 Division Number 105/36=2.9m

Displacement -Y (Transverse direction)							
NODE	LOAD	DY (m)	wy (kN)	wy*DY	wy*DY^2	Rx	Rz
1	UNIT LOAD Y	0.000018	240	0.00432	7.776E-08	-0.000007	0.00002
2	UNIT LOAD Y	0.000199	484.3	0.096376	1.918E-05	-0.000018	0.000019
3	UNIT LOAD Y	0.000362	484.3	0.175317	6.346E-05	-0.000030	0.000016
4	UNIT LOAD Y	0.000495	484.3	0.239729	0.0001187	-0.000042	0.000013
5	UNIT LOAD Y	0.000589	484.3	0.285253	0.000168	-0.000053	0.000008
6	UNIT LOAD Y	0.000653	484.3	0.316248	0.0002065	-0.000053	0.000006
7	UNIT LOAD Y	0.00068	484.3	0.329324	0.0002239	-0.000053	0.000001
8	UNIT LOAD Y	0.00066	484.3	0.319638	0.000211	-0.000053	-5E-06
9	UNIT LOAD Y	0.000594	484.3	0.287674	0.0001709	-0.000053	-9E-06
10	UNIT LOAD Y	0.000495	484.3	0.239729	0.0001187	-0.000042	-1.3E-05
11	UNIT LOAD Y	0.00036	484.3	0.174348	6.277E-05	-0.000030	-1.6E-05
12	UNIT LOAD Y	0.000197	484.3	0.095407	1.88E-05	-0.000018	-1.9E-05
13	UNIT LOAD Y	0.000017	484.3	0.008233	1.4E-07	-0.000006	-0.00002
1287	UNIT LOAD Y	0.00008	484.3	0.038744	3.1E-06	-0.000011	0.000019
1288	UNIT LOAD Y	0.00014	484.3	0.067802	9.492E-06	-0.000015	0.000019
1289	UNIT LOAD Y	0.000256	484.3	0.123981	3.174E-05	-0.000022	0.000018
1290	UNIT LOAD Y	0.00031	484.3	0.150133	4.654E-05	-0.000026	0.000017
1291	UNIT LOAD Y	0.00041	484.3	0.198563	8.141E-05	-0.000034	0.000015
1292	UNIT LOAD Y	0.000454	484.3	0.219872	9.982E-05	-0.000038	0.000014
1293	UNIT LOAD Y	0.000531	484.3	0.257163	0.0001366	-0.000045	0.000011
1294	UNIT LOAD Y	0.000562	484.3	0.272177	0.000153	-0.000049	0.00001
1295	UNIT LOAD Y	0.000667	484.3	0.323028	0.0002155	-0.000053	0.000004
1296	UNIT LOAD Y	0.000677	484.3	0.327871	0.000222	-0.000053	0.000002
1297	UNIT LOAD Y	0.000679	484.3	0.32884	0.0002233	-0.000053	-1E-06
1298	UNIT LOAD Y	0.000672	484.3	0.32545	0.0002187	-0.000053	-3E-06
1299	UNIT LOAD Y	0.000643	484.3	0.311405	0.0002002	-0.000053	-6E-06
1300	UNIT LOAD Y	0.000621	484.3	0.30075	0.0001868	-0.000053	-8E-06
1301	UNIT LOAD Y	0.000565	484.3	0.27363	0.0001546	-0.000050	-0.00001
1302	UNIT LOAD Y	0.000532	484.3	0.257648	0.0001371	-0.000046	-1.2E-05
1303	UNIT LOAD Y	0.000454	484.3	0.219872	9.982E-05	-0.000038	-1.4E-05
1304	UNIT LOAD Y	0.000409	484.3	0.198079	8.101E-05	-0.000034	-1.5E-05
1305	UNIT LOAD Y	0.000309	484.3	0.149649	4.624E-05	-0.000026	-1.7E-05
1306	UNIT LOAD Y	0.000254	484.3	0.123012	3.125E-05	-0.000022	-1.8E-05
1307	UNIT LOAD Y	0.000139	484.3	0.067318	9.357E-06	-0.000014	-1.9E-05
1308	UNIT LOAD Y	0.000078	484.3	0.037775	2.946E-06	-0.000010	-1.9E-05
1309	UNIT LOAD Y	0.000613	484.3	0.296876	0.000182	-0.000053	0.000007
1310	UNIT LOAD Y	0.000634	240	0.15216	9.647E-05	-0.000053	0.000007
		0.016008	17430.5	7.593391	0.00405084		
		α		**β**	**Ƴ**		

4.2 Linear Analysis

4.2.1 Model Analysis

Equation of motion of multi-freedom system is expressed as follows

$\mathbf{M\ddot{D}+C\dot{D}+KD=MZ}$ …………………………………………………… (4.2.1)

$\mathbf{Z=\ddot{Z}L}$ ………………………………………………………………….. (4.2.2)

M: Mass Matrix, **C**: Damping Matrix, **K**: Stiffness Matrix, **D:** Displacement vector

$\mathbf{\ddot{Z}}$: Acceleration vector of the ground, **L**: Acceleration distribution vector.

D can be dissolved by mode vector **φ** and generalized coordinate **q**.

$\mathbf{D}=\varphi_1 q_1+\varphi_2 q_2+\cdots+\varphi_n q_n=\Sigma\varphi_j q_j$ ………………………… (4.2.3)

$\mathbf{M\varphi \ddot{q} +C\varphi\dot{q}+K\varphi q = -M\ddot{Z}}$ …………………………………………… (4.2.4)

Multiply transposed Matrix φ^T

$\mathbf{\varphi^T M\varphi \ddot{q} +\varphi^T C\varphi \dot{q} +\varphi^T K\varphi q = -\varphi^T M\ddot{Z}}$ ……………………………….. (4.2.5)

$$\phi^T M\phi = \overline{M} = \begin{bmatrix} \overline{M}_1 & 0 & \dots & 0 \\ 0 & \overline{M}_2 & \dots & 0 \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & \dots & \overline{M}_n \end{bmatrix}$$

$$\phi^T C\phi = \overline{C} = \begin{bmatrix} \overline{C}_1 & 0 & \dots & 0 \\ 0 & \overline{C}_2 & \dots & 0 \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & \dots & \overline{C}_n \end{bmatrix}$$

$$\phi^T K\phi = \overline{K} = \begin{bmatrix} \overline{K}_1 & 0 & \dots & 0 \\ 0 & \overline{K}_2 & \dots & 0 \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & \dots & \overline{K}_n \end{bmatrix}$$

Eq 4.2.5 can be dissolved into n number independent 1 degree freedom equations.
In case of 2 dimension, L_x^T=(1010・・・・・10), L_y^T= (0101・・・・・・０１)

$\omega_j^2=\overline{K_j}/\overline{M_j}$, $h_j=\overline{C_j}/(2\omega_j\overline{M_j})$, $f_j=\overline{F_j}/\overline{M_j}$ ……………………………………… (4.2.6)

$\ddot{q}_j+2h_j\omega_j\dot{q}_j+\omega_j^2 q_j = f_j$ ………………………………………………………. (4.2.7)

$f_j= -\beta_j\ddot{z}$ ………………………………………………………………… (4.2.8)

ParticipationFactor $\beta_j=\varphi_j^T ML_j/\overline{M_j}$ ………………………………………….. (4.2.9)

$$q_j = \exp\{-h_j\omega_j t\}\{\overline{A}_j \cos\omega_j' t + \overline{B}_j \sin\omega_j' t\} - \frac{\beta_j}{\omega_j'}\int_0^t \ddot{z}(\tau)\exp\{-h_j\omega_j(t-\tau)\}\sin\omega_j'(t-\tau)d\tau \quad \text{------------------} \quad (4.2.10)$$

Where $\omega_j' = \omega_j\sqrt{(1-h_j^2}$

Effective mass $m_j = (\varphi_j^T ML_j)^2/\overline{M}_j$ ………………………………… (4.2.11)

4.2.2 Response Spectrum method

We can obtain the right answer of the time history multi-freedom structural system by above mentioned modal analysis. However, time history response is not necessarily required and only the maximum response is necessary for seismic design of the structures.

Maximum response in j-th can be obtained from Eq. 4.2.10.

$$\left.\begin{array}{l} S_{Dj} = q_{jmax} \\ S_{Vj} = \dot{q}_{jmax} \approx \omega S_{Dj} \end{array}\right\} \quad \text{--} \quad (4.2.12)$$

$$S_{Aj} = (\ddot{q}_j + \ddot{z})_{max} \approx \omega^2 S_{Dj}$$

Maximum response of j order mode is $Rj = \phi_j \beta_j S_{Dj}$

Design value shall be obtained as the CQC （Complete Quadratic Combination) value.

$$R_{max} = \left[\sum_{j=1}^{N}\sum_{j=1}^{N} R_i \rho_{ij} R_j\right]^{1/2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.. \quad (4.2.13)$$

Where,

$$\rho_{ij} = \frac{8\xi^2(1+r)r^{3/2}}{(1-r^2)^2+4\xi^2 r(1+r)^2}, \qquad r = \omega_j/\omega_i$$

R_{max} : Peak Response
Rj: Peak response of i-th mode
R: Natural frequency ratio of i-th mode to j-th mode
ξ :Damping ratio

4.2.3 Time History Direct Integration Analysis

(Newmark's β Method)

$$\mathbf{m}\ddot{\boldsymbol{u}} + \mathbf{c}\dot{\boldsymbol{u}} + \mathbf{ku} = \mathbf{p(t)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.2.14)$$

Dividing p(t) into small time increment, **p**i and **P**i+1

$$\mathbf{m}\ddot{\mathbf{u}}_{i+1} + \mathbf{c}\dot{\mathbf{u}}_{i+1} + \mathbf{k}\mathbf{u}_{i+1} = \mathbf{p}_{i+1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.. \quad (4.2.15)$$

$$\dot{\mathbf{u}}_{i+1} = \dot{\mathbf{u}}_i + \frac{1}{2}\Delta t(\ddot{\mathbf{u}}_t + \ddot{\mathbf{u}}_{t+1}) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.2.16)$$

$$\mathbf{u}_{i+1} = \mathbf{u}_i + \Delta t\dot{\mathbf{u}}_i + \left(\frac{1}{2} - \beta\right)\Delta t^2 \ddot{\mathbf{u}}_i + \beta \Delta t^2 \ddot{\mathbf{u}}_{i+1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.2.17)$$

Obtaining unknown vector $\ddot{u}_{i+1}$, *by* putting (4.2.17) into (4.2.14),

$$\ddot{\mathbf{u}}_{i+1} = \left(\mathbf{m} + \frac{\Delta t}{2}\mathbf{c} + \beta\Delta t^2 \mathbf{k}\right)^{-1}\left[\boldsymbol{p}_{i+1} - \boldsymbol{c}\left(\dot{\boldsymbol{u}}_i + \frac{\Delta t}{2}\ddot{\mathbf{u}}_i\right) - \boldsymbol{k}\left\{\boldsymbol{u}_i + \Delta t\dot{\boldsymbol{u}}_i + \left(\frac{1}{2} - \boldsymbol{\beta}\right)\Delta t^2 \ddot{\mathbf{u}}_i\right\}\right] \cdot (4.2.18)$$

Therefore, response value $\ddot{\mathbf{u}}_{i+1}, \dot{\mathbf{u}}_{i+1}, \mathbf{u}_{i+1}$ can be obtained from known value $\ddot{\boldsymbol{u}}_i, \dot{\boldsymbol{u}}_i, \mathbf{u}_i$**.**
1/4, 1/6 are usually adopted as β.
Time increment for direct integration is as follows.

$$\Delta \text{ t} = \frac{T_p}{10} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.2.19)$$

Where, Tp = the highest modal period being considered.

4.3 Inelastic Time History Analysis

In seismic engineering, it is nowadays common to distinguish between so-called *force based* and *displacement-based* analysis techniques. Although it is not always strictly defined what these two expressions comprise in detail, it appears justified to make a difference between these two conceptual approaches. Many existing seismic codes including the current BSDS can be considered as **force-based**. **This Chapter does not have the status of a code but may rather be considered as a guide.**

While force-based analysis represents the more traditional approach, modern displacement-based analysis methods bear some conceptual advantages. They may be considered as more accurate, but they are also somewhat more demanding with respect to the knowledge of the analyzing engineer. One possible alternative to traditional force-based analysis is represented by an *inelastic time history analysis* (ITHA) which may be considered as the most complete analysis technique. In this method the inelastic behavior of the system is explicitly modeled – including the hysteretic response of the members under cyclic loading. Using this model, a real dynamic analysis is conducted in which the differential equation of motion is solved (numerically) for a given ground motion exciting the base of the structure.

As ITHA is conceptually able to capture the important phenomena related to the seismic response of the system, the quality of the analysis results only depends on the accuracy of the inelastic structural model and the adequacy of the used input ground motions. The remaining uncertainties resulting from these two issues should not be underestimated so that even this advanced analysis technique does not necessarily guarantee a fully realistic assessment result.

As inelastic time history analysis gives a complete picture of the entire response, including the inelastic force and displacement time history of the individual members, it might be considered as superordinate to *force based* or *displacement-based* analyses techniques. Therefore, ITHA is not only the conceptually most realistic analysis approach, but it also gives the most complete set of structural response data. These also allow the computation of the energy dissipated by individual members. Such data can theoretically be used for damage estimations taking the cyclic response into account (provided an appropriate damage model is available).

Despite these considerable advantages of ITHA, it may not be the best choice for the analysis of structures in ordinary cases, e.g. in an engineering company. Aside from the fact that ITHA can become computationally rather demanding, its application also requires advanced knowledge of the method. The sophisticated numerical solution strategies of the inelastic dynamic problem are sensitive to several aspects and convergence is not always guaranteed. Furthermore, the hysteretic modeling of the structure requires a considerable amount of additional data to fully characterize the inelastic system behavior, whose realistic determination might not always be straightforward. Other aspects, as e.g. the choice of adequate ground motions or appropriate viscous damping models, need to be considered in addition. The large amount of required data and the sophistication of the problem can make ITHA somewhat prone to errors, especially if the analyzing engineer is not sufficiently familiar with the potential sources of errors.

ITHA is dynamic analysis, which considers material nonlinearity of a structure. Considering the efficiency of the analysis, nonlinear elements are used to represent important parts of the structure, and the remainder is assumed to behave elastically. Explanation of material non-linearity were defined in Chapter 3.

4.3.1 Analysis Method

When the structure enters the nonlinear range, or has nonclassical damping properties, modal analysis cannot be used. A numerical integration method sometimes referred to as time history analysis, is required to get more accurate responses of the structure. In a time history analysis, the time scale is divided into a series of smaller steps, $d\tau$. Let us say the response at *i*th time interval has already determined and is denoted by *ui* , *ùi* ,*üi*. Then, the response of the system at *i*th time interval will satisfy the equation of motion (Equation 4.3.2).

$$[M]\{\ddot{u}_i\}+[C]\{\dot{u}_i\}+[K]\{u_i\}=-[M]\{\ddot{u}_{gi}\} \quad \text{……………………………….. (4.3.2)}$$

The time stepping method enables us to step ahead and determine the responses u_i+1 , $\dot{u}_{i+1}$, $\ddot{u}_{i+1}$ at *i* + 1_{th} time interval by satisfying the Equation 4.3.2. Thus, the equation of motion at *i* + 1th time interval will be

$$[M]\{\ddot{u}_{i+1}\}+[C]\{\dot{u}_{i+1}\}+[K]\{u_{i+1}\}=-[M]\{\ddot{u}_{gi+1}\} \quad \text{……………………………. (4.3.3)}$$

Equation 4.3.3 needs to be solved before proceeding to the next time step. By stepping through all the time steps, the actual response of the structure can be determined at all time instants. Direct integration must be used for inelastic time history analysis of a structure, which contains nonlinear elements of the Element Type. If a structure contains nonlinear elements of the Force Type only, much faster analysis can be performed through modal superposition. From this point on, inelastic time history analysis by direct integration is explained.

4.3.1.1 Direct Integration Method

These procedures will allow the nodal displacements to be determined at different time increments for a given dynamic system. By applying a direct integration scheme, equation is integrated using a **numerical step-by-step procedure**. The methods do not require any transformations of the equations into different forms and are therefore considered as direct. Direct numerical integration is based on fulfilling two fundamental conditions, (1) instead of satisfying previous equation at any time *t* the aim is to satisfy it only at discrete time intervals separated by an increment D*t*. The result of this is that static equilibrium, which includes the effect of inertia and damping forces, is sought at discrete time instances within the studied time interval. The second condition (2) is that the variation of displacements, velocities and accelerations within each time interval D*t* is assumed. These assumptions will determine the accuracy and stability of the solution procedure. This method must be applied to solve **non-linear** problems.

There are two classifications of direct integration: ***explicit* and *implicit.*** When a direct computation of the dependent variables can be made in terms of **known quantities**, the computation is said to be **explicit.** When the dependent variables are defined by **coupled sets of equations**, and either a matrix or iterative technique is needed to obtain the solution, the numerical method is said to be **implicit**.

There are several numerical methods as explained in some other books (e.g Chopra 2012) both explicit and implicit, however, in this chapter only Newmark integration method was explained.

4.3.1.2 Newmark Method

In 1959, N. M. Newmark developed a family of time-stepping methods to solve for second order differential equation in dynamic analysis based on the following equations:

$$\dot{u}_{i+1} = \dot{u}_i + \left[(1-\gamma)\Delta t\right]\ddot{u}_i + \left(\gamma\Delta t\right)\ddot{u}_{i+1} \qquad (4.3.4a)$$

$$u_{i+1} = u_i + \left(\Delta t\right)\dot{u}_i + \left[\left(0.5-\beta\right)\left(\Delta t\right)^2\right]\ddot{u}_i + \left[\beta\left(\Delta t\right)^2\right]\ddot{u}_{i+1} \qquad \text{..............................} \qquad (4.3.4b)$$

The parameters β and γ define the variation of acceleration over a time step and determine the stability and accuracy characteristics of the method. Typical selection for γ is 1/2, and $1/6 \leq \beta \leq 1/4$ is satisfactory from all points of view, including that of accuracy. These two equations, combined with the equilibrium equation at the end of the time step, provide the basis for computing *ui*+1, $\dot{u}_{i+1}$, and $\ddot{u}_{i+1}$ at time *i* + 1 from the known *ui* , $\dot{u}_i$, and $\ddot{u}_i$ at time *i* . Iteration is required to implement these computations because the unknown $\ddot{u}_{i+1}$ appears in the right side of Eq. (4.3.4).

For linear systems it is possible to modify Newmark's original formulation, however, to permit solution of Eqs. (4.3.4a) and (4.3.4b) without iteration. Before describing this modification, we demonstrate that two special cases of Newmark's method are the well-known constant average acceleration and linear acceleration methods.

4.3.1.3 Stability

Numerical procedures that lead to bounded solutions if the time step is shorter than some stability limit is called *conditionally stable procedures*. Procedures that lead to bounded solutions regardless of the time-step length are called *unconditionally stable procedures*. The average acceleration method is unconditionally stable. The linear acceleration method is stable if $\Delta t/Tn < 0.551$, and the central difference method is stable if $\Delta t/Tn < 1/\pi$.Obviously, the latter two methods are conditionally stable.

The stability criteria are not restrictive (i.e., they do not dictate the choice of time step) in the analysis of SDF systems because $\Delta t/Tn$ must be considerably smaller than the stability limit (say, 0.1 or less) to ensure adequate accuracy in the numerical results. Stability of the numerical method is important, however, in the analysis of MDF systems, where it is often necessary to use unconditionally stable methods.

4.3.1.4 Nonlinear Systems: Newmark's Method

In this section, Newmark's method described earlier for linear systems is extended to nonlinear systems. Recall that this method determines the solution at time *i* + 1 from the equilibrium condition at time *i* + 1, i.e., Eq. (4.3.1) for nonlinear systems. Because the resisting force $(f_s)_{i+1}$ is an implicit nonlinear function of the unknown *ui*+1, iteration is required in this method. This requirement is typical of implicit methods. It is instructive first to develop the Newton–Raphson method of iteration for static analysis of a nonlinear SDF system (refer to: Chopra 2012). The Newton-Raphson Algorithm for systems with several or many DoF's follows exactly the same procedure as the algorithm for SDoF systems. Only difference: Scalar values are replaced by the corresponding vectorial quantities. In most FE-analysis programs both Newton-Raphson Algorithms as well as other algorithms are typically combined in a general solver in order to obtain a successful convergence of the iteration process for many structural analysis problems. Steps of numerical calculation as shown in **Table 4.3-1**.

Table 4.3-1 Newmark's method: Non-Linear system (Chopra 2012)

Special cases

(1) Average acceleration method ($\gamma = \frac{1}{2}, \beta = \frac{1}{4}$)

(2) Linear acceleration method ($\gamma = \frac{1}{2}, \beta = \frac{1}{6}$)

1.0 *Initial calculations*

1.1 State determination: $(f_S)_0$ and $(k_T)_0$.

1.2 $\ddot{u}_0 = \dfrac{p_0 - c\dot{u}_0 - (f_S)_0}{m}$.

1.3 Select Δt.

1.4 $a_1 = \dfrac{1}{\beta(\Delta t)^2}m + \dfrac{\gamma}{\beta \Delta t}c; \quad a_2 = \dfrac{1}{\beta \Delta t}m + \left(\dfrac{\gamma}{\beta} - 1\right)c;$ and

$a_3 = \left(\dfrac{1}{2\beta} - 1\right)m + \Delta t\left(\dfrac{\gamma}{2\beta} - 1\right)c.$

2.0 *Calculations for each time instant,* $i = 0, 1, 2, \ldots$

2.1 Initialize $j = 1$, $u_{i+1}^{(j)} = u_i$, $(f_S)_{i+1}^{(j)} = (f_S)_i$, and $(k_T)_{i+1}^{(j)} = (k_T)_i$.

2.2 $\hat{p}_{i+1} = p_{i+1} + a_1 u_i + a_2 \dot{u}_i + a_3 \ddot{u}_i$.

3.0 *For each iteration,* $j = 1, 2, 3 \ldots$

3.1 $\hat{R}_{i+1}^{(j)} = \hat{p}_{i+1} - (f_S)_{i+1}^{(j)} - a_1 u_{i+1}^{(j)}$.

3.2 Check convergence; If the acceptance criteria are not met, implement steps 3.3 to 3.7; otherwise, skip these steps and go to step 4.0.

3.3 $(\hat{k}_T)_{i+1}^{(j)} = (k_T)_{i+1}^{(j)} + a_1$.

3.4 $\Delta u^{(j)} = \hat{R}_{i+1}^{(j)} \div (\hat{k}_T)_{i+1}^{(j)}$.

3.5 $u_{i+1}^{(j+1)} = u_{i+1}^{(j)} + \Delta u^{(j)}$.

3.6 State determination: $(f_S)_{i+1}^{(j+1)}$ and $(k_T)_{i+1}^{(j+1)}$.

Replace j by $j + 1$ and repeat steps 3.1 to 3.6; denote final value as u_{i+1}.

4.0 *Calculations for velocity and acceleration*

4.1 $\dot{u}_{i+1} = \dfrac{\gamma}{\beta \Delta t}(u_{i+1} - u_i) + \left(1 - \dfrac{\gamma}{\beta}\right)\dot{u}_i + \Delta t\left(1 - \dfrac{\gamma}{2\beta}\right)\ddot{u}_i$.

4.2 $\ddot{u}_{i+1} = \dfrac{1}{\beta(\Delta t)^2}(u_{i+1} - u_i) - \dfrac{1}{\beta \Delta t}\dot{u}_i - \left(\dfrac{1}{2\beta} - 1\right)\ddot{u}_i$.

5.0 *Repetition for next time step.* Replace i by $i + 1$ and implement steps 2.0 to 4.0 for the next time step.

4.3.2 Hysteresis Model

In structural analysis, Hysteretic can be define as:

- The dependence of the state of a system on its history.
- Plots of a single component of the moment often form a loop or hysteresis curve, where there are different values of one variable depending on the direction of change of another variable.
- The lag in response exhibited by a body in reacting to changes in the forces affecting it.

Many different hysteretic models have been proposed in the past trying to simulate the inelastic behavior of RC Structures, and in nowadays they are used to obtain Inelastic Earthquake Responses. There are several hysteretic modeled based on experimental observations that was introduced such as Takeda Model. The Response is mainly related with the Energy dissipation capacity of each hysteretic model and parameters are those which can influence on the shape (fatness and longness) of a hysteresis loop.

The Takeda hysteresis model was developed by Takeda, Sozen and Nielsen [1970], Otani [1981] and Kabeyasawa, Shiohara, Otani, Aoyama [1983] to represent the force-displacement hysteretic properties of RC structures. The Takeda model according to Otani (1981) includes (a) stiffness changes at flexural cracking and yielding, (b) rules for inner hysteresis loops inside the outer loop, and (c) unloading stiffness degradation with deformation. The hysteresis rules are extensive and comprehensive (**Figure 4.3-1**). In this chapter the modified Takeda Model [Ref: Kabeyasawa, Shiohara, Otani, Aoyama; May 1983.

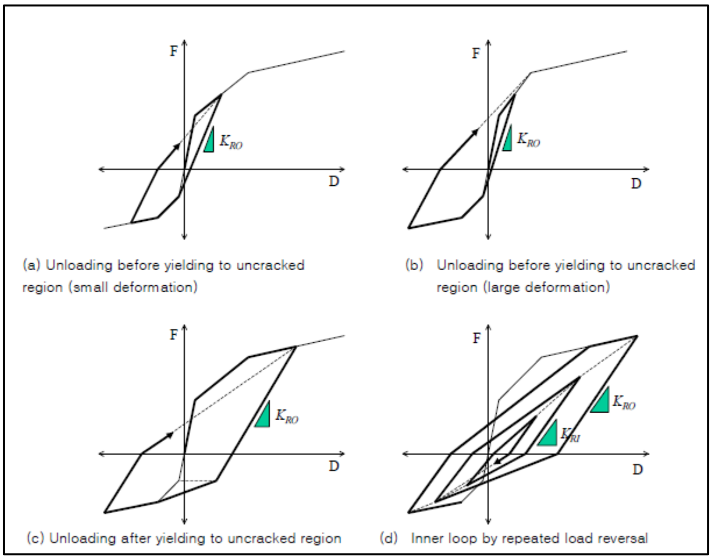


Figure 4.3-1 Takeda hysteresis model – Ref: Hysteresis Models of Reinforced Concrete for Earthquake Response Analysis by Otani [May 1981]

The main difference with the other models is that it has Hysteresis rules for inner Hysteresis loops inside the outer loop and also it has unloading stiffness degradation as follows:

$$K_{RO} = \left(\frac{F_y + F_c}{D_y + D_c}\right) \cdot \left(\frac{D_y}{D_m}\right)^{\beta} \quad \text{(4.3.5)}$$

where,

KRO	:	Unloading stiffness of the outer loop
FC	:	First yield force in the region opposite to unloading point
FY	:	Second yield force in the region to which unloading point belongs
DC	:	First yield displacement in the region opposite to unloading point
DY	:	Second yield displacement in the region to which unloading point belongs
DM	:	Maximum deformation in the region to which unloading point belongs
B	:	Constant for determining the unloading stiffness of the outer loop

If the sign of load changes in the process, the coordinates progress towards the maximum deformation point on the skeleton curve in the region of the proceeding direction. If yielding has not occurred in the region, the coordinates continue to progress without changing the unloading stiffness until the load reaches the first yield force. Upon reaching the first yield force, it progresses towards the second yield point.

Inner loop is formed when unloading takes place before the load reaches the target point on the skeleton curve while reloading is in progress, which takes place after the sign of load changes in the process of unloading. Unloading stiffness for inner loop is determined by the following equation.

$$K_{RI} = \gamma K_{RO} \qquad (4.3.6)$$

where:

KRI : Unloading stiffness of inner loop

KRO : Unloading stiffness of the outer loop in the region to which the start point of unloading belongs.

γ : Unloading stiffness reduction factor for inner loop

In the above equation, β=0.0 for calculating K_{RO} and γ=1.0 for calculating K_{RI} are set if the second yielding has not occurred in the region of unloading. In the case where the sign of load changes in the process of unloading in an inner loop, the load progresses towards the maximum deformation point, if it exists on the inner loop in the region of the proceeding direction. If the maximum deformation point does not exist on the inner loop, the load directly progresses towards the maximum deformation point on the skeleton curve. If the maximum deformation point exists and there exists multiple inner loops, it progresses towards the maximum deformation point, which belongs to the outermost inner loop. Also, if loading continues through the point, it progresses towards the maximum deformation point on the skeleton curve.

4.3.2.1 Relationships between Force-Displacement (F-Δ) and Moment-Curvature (M-Ø)

In this section the relationship between Force-Displacement (F-Δ) and Moment-Curvature (M-Ø) is explained.

By specifying a plastic hinge length, Lp, increasing curvature demands on a SDOF cantilever system with height H can be translated to an equivalent displacement response in accordance with Equation (4.3.7).

$$\begin{aligned} D_t &= D_e + D_p \\ &= \frac{\phi_e \times H^2}{3} + (\phi_t - \phi_e) \times L_p H \end{aligned} \qquad (4.3.7)$$

where De is the elastic displacement component, Dp is the plastic deformation component associated with the inelastic rotation of a plastic hinge, *Øt* is the total curvature at the plastic hinge location and *Øe* is the elastic curvature. Note that the ratio of the total displacement to the yield displacement (i.e. the displacement ductility demand) can be expressed for a cantilever in terms of the curvature ductility demand by Equation (4.7).

$$\frac{D_t}{D_y} = \mu_\Delta = 1 + 3(\mu_\phi - 1) \times \frac{L_p}{H} \qquad (4.3.8)$$

After reaching a total displacement of Δt, the Takeda model instructs the structure to unload with a reduced stiffness given by Equation (4.4).

If we assume, for simplicity, that there is no strain hardening and note that the Takeda model is specified for NLTHAs in a Moment-Curvature environment, then the elastic curvature recovered in unloading the structure from a total displacement demand of Dt is given by Equation (4.3.9).

$$\phi_{un} = \phi_t - \frac{F}{K_{RO}} \quad \text{…………………………………………………………………….. (4.3.9)}$$

The ratio of the elastic displacement recovered in unloading to the yield displacement of a cantilever is therefore given by Equation (4.3.10).

$$\frac{D_{un}}{D_y} = \frac{\phi_{un} H^2}{3} \frac{3}{\phi_y H^2} = \mu_\phi^{\alpha} \quad \text{………………………………….. ……………………. (4.3.10)}$$

Dividing Equation (4.3.7) by Equation (4.3.10), we obtain Equation (4.10) which expresses the ratio of the total displacement demand to the unloading displacement as a function of the curvature ductility demand, the ratio Lp/H, and the alpha factor.

$$\frac{D_t}{D_{un}} = \frac{1}{\mu_\phi^{\alpha}} + 3\left(\mu_\phi - 1\right) \frac{L_p}{H \mu_\phi^{\alpha}} \quad \text{……………………………………………………….. (4.3.11)}$$

The inelastic demand estimations in the direct displacement-based design approach developed by *Priestley et al.* Based on regression analysis, *Priestley et al.* calibrated an individual set of parameters to be used with the above equations for each of the considered hysteretic models. These parameters are given in Error! Reference source not found. for selected hysteretic models.

Note that for the parameter λ two values are given for each hysteretic rule. The upper value is to be used if a constant elastic viscous damping coefficient is considered appropriate in the original inelastic model, whereas the lower value corresponds to tangent stiffness proportional damping in the inelastic model. The positive λ value for the constant damping model results in an increasing damping ratio $\xi el_{,eff(\mu\Delta)}$ with increasing ductility demand μ_Δ. It thus (partly) compensates for the decreasing critical damping coefficient $ccr_{,eff(\mu\Delta)}$. In contrast, the negative λ value for the tangent stiffness proportional damping model yields a decreasing damping ratio $\xi el_{,eff(\mu\Delta)}$ as the tangent stiffness proportional damping coefficient decreases stronger with ductility than the effective critical damping coefficient.

According to *Priestley et al. 2007* refer to his book *"Displacement-Based Seismic Design of Structures"* **For reinforced concrete piers, the "thin" Takeda model may be considered most representative**. The fat Takeda having higher energy dissipation is rather appropriate for RC beams and frame structures.

– "Thin" Takeda hysteresis with a post-yield stiffness ratio of γ = 0.05 unloading stiffness parameter α = 0.5 and reloading stiffness parameter β = 0.

– "Fat" Takeda hysteresis with a post-yield stiffness ratio of γ = 0.05 unloading stiffness parameter α = 0.3

$$\xi_{eq,tot}\left(\mu_\Delta, T_{eff}\right) = \xi_{el,eff}\left(\mu_\Delta\right) + \xi_{eq,hyst}\left(\mu_\Delta, T_{eff}\right)$$

$$\xi_{el,eff}\left(\mu_\Delta\right) = \xi_{el,0} \cdot \mu_\Delta^\lambda \qquad \text{...} \quad (4.3.12)$$

$$\xi_{eq,hyst}\left(\mu_\Delta, T_{eff}\right) = a \cdot \left(1 - \frac{1}{\mu_\Delta^b}\right) \cdot \left(1 + \frac{1}{\left(T_{eff} + c\right)^d}\right)$$

Table 4.3-2 Parameter λ and parameters a, b, c, and d for the use of equation (4.11) for various hysteretic models according to Priestley et. al, 2007

Hysteretic Model	Elastic Viscous Damping Model	Parameters				
		Eq. (4.32)	Equation (4.33)			
		λ [-]	a [-]	b [-]	c [-]	d [-]
Elasto-Plastic $r = 0$	constant	0.127	0.224	0.336	-0.002	0.250
	tang. stiff. prop.	-0.341				
Takeda "Fat" $r = 0.05,\ \alpha = 0.3,\ \beta = 0.6$	constant	0.312	0.305	0.492	0.790	4.463
	tang. stiff. prop.	-0.313				
Takeda "Thin" $r = 0.05,\ \alpha = 0.5,\ \beta = 0$	constant	0.340	0.215	0.642	0.824	6.444
	tang. stiff. prop.	-0.378				
Ramberg-Osgood $r_{RO} = 7$	constant	-0.060	0.289	0.622	0.856	6.460
	tang. stiff. prop.	-0.617				

4.3.2.2 Material Non-Linearity

Concrete material nonlinearity is incorporated into analysis using a nonlinear stress–strain relationship **Figure 4.3-2** shows idealized stress–strain curves for unconfined and confined concrete in uniaxial compression. Tests have shown that the confinement provided by closely spaced transverse reinforcement can substantially increase the ultimate concrete compressive stress and strain. The confining steel prevents premature buckling of the longitudinal compression reinforcement and increases the concrete ductility. Extensive research has been made to develop concrete stress–strain relationships (Hognestad, 1951; Popovics, 1970; Kent and Park, 1971; Park and Paulay, 1975; Wang and Duan, 1981; Mander et al., 1988a and 1988b; Hoshikuma et al., 1997). AASHTO Guide Specifications for LRFD Seismic Bridge Design 2011 recommended the use of Mander stress-strain model for confined concrete.

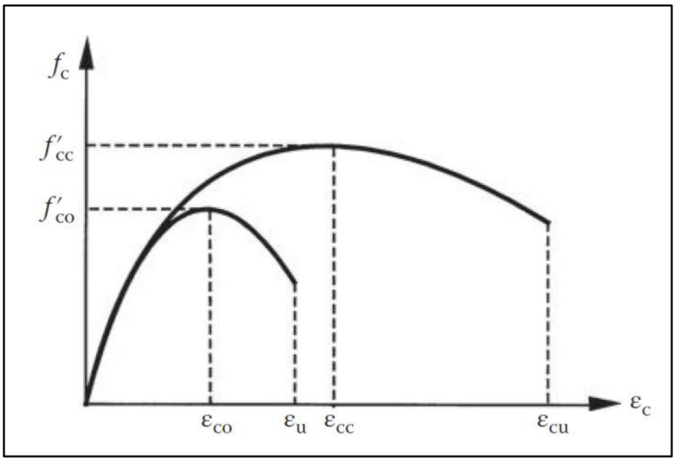


Figure 4.3-2 Idealized stress–strain curves of concrete in uniaxial compression

4.3.2.3 Confined Concrete – Mander's Model

Analytical models describing the stress–strain relationship for confined concrete depend on the confining transverse reinforcement type (such as hoops, spiral, or ties) and shape (such as circular, square, or rectangular). Some of those analytical models are more general than others in their applicability to various confinement types and shapes. A general stress–strain model (**Figure 4.3-3**) for confined concrete applicable (in theory) to a wide range of cross sections and confinements was proposed by Mander et al. (1988a and 1988b) and has the following form:

$$fc = \frac{f'_{cc}(\varepsilon_c / \varepsilon_{cc})r}{r - 1 + (\varepsilon_c / \varepsilon_{cc})^r} \quad \text{(4.3.13)}$$

$$\varepsilon_{cc} = \varepsilon_{co}\left(1 + 5\left(\frac{f'_{cc}}{f'_{co}} - 1\right)\right) \quad \text{(4.3.14)}$$

$$r = \frac{E_c}{E_c - E_{\sec}} \quad \text{(4.3.15)}$$

$$E_{\sec} = \frac{f'_{cc}}{E_c} \quad \text{(4.3.16)}$$

where f'_{cc} and ε_{cc} are peak compressive stress and corresponding strain for confined concrete. f'_{cc} and ε_{cu} , which depend on the confinement type and shape, are calculated as follows:

(1) Confined Peak Stress

1. For concrete circular section confined by circular hoops or spiral (**Figure 4.3-4**)

$$f'_{cc} = f'_{co}\left(2.254\sqrt{1 + \frac{7.94 f'_i}{f'_{co}}} - \frac{2 f'_i}{f'_{co}} - 1.254\right) \quad \text{(4.3. 15)}$$

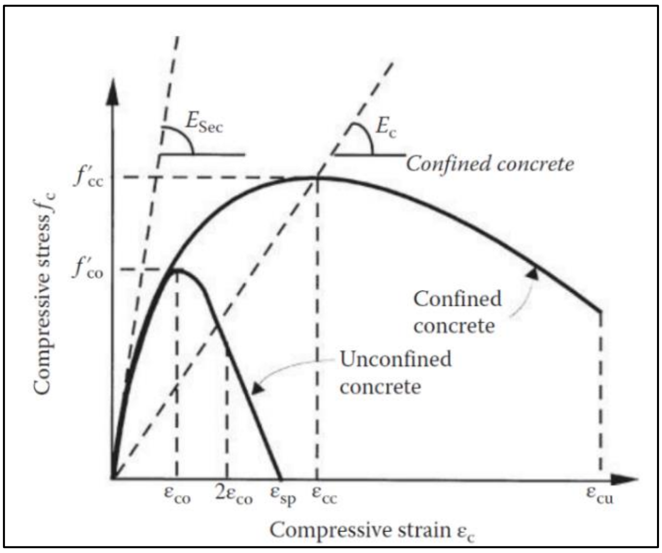


Figure 4.3-3 Stress-strain curves of concrete-Mander model

$$f_1' = \frac{1}{2} K_e \rho_s f_{yh} \quad \ldots\ldots \quad (4.3.17)$$

$$K_e = \begin{cases} (1-s'/2d_s)^2/(1-\rho_{cc}) & \text{For Circular hoops} \\ (1-s'/2d_s)/(1-\rho_{cc}) & \text{For Circular Spiral} \end{cases} \quad \ldots\ldots \quad (4.3.18)$$

$$\rho_s = \frac{4A_{sp}}{d_s s} \quad \ldots\ldots \quad (4.3.19)$$

where f_1 is the effective lateral confining pressure, K_e confinement effectiveness coefficient, f_{yh} the yield stress of the transverse reinforcement, s' the clear vertical spacing between hoops or spiral; s the center to center spacing of the spiral or circular hoops, ds centerline diameter of spiral or hoops circle, ρ_{cc} the ratio of the longitudinal reinforcement area to the cross-section core area, ρ_s is the ratio of the transverse confining steel volume to the confined concrete core volume, and A_{sp} the bar area of transverse reinforcement.

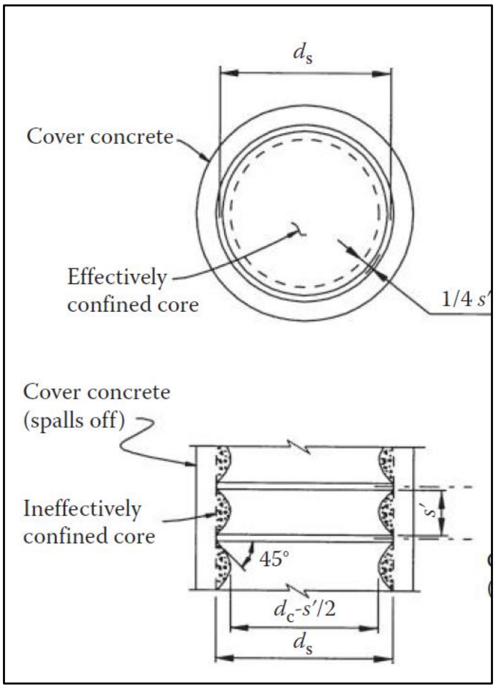


Figure 4.3-4 Confined core for hoop reinforcement

2. For rectangular concrete section confined by rectangular hoops (**Figure 4.3-6**)
   The rectangular hoops may produce two unequal effective confining pressures f'_{1x} and f'_{1y} in the principal x- and y-direction defined as follows:

$$f_{1x}' = K_e \rho_x f_{yh} \quad \ldots\ldots \quad (4.3.20)$$

$$f_{1y}' = K_e \rho_y f_{yh} \quad \ldots\ldots \quad (4.3.21)$$

$$K_e = \frac{\left[1 - \sum_{i=1}^{n} \frac{\left(w_i'\right)^2}{6 b_c d_c}\right]\left(1 - \frac{s'}{2b_c}\right)\left(1 - \frac{s'}{2d_c}\right)}{\left(1 - \rho_{cc}\right)} \quad \ldots\ldots \quad (4.3.22)$$

$$\rho_x = \frac{A_{sx}}{s d_c} \quad \ldots\ldots \quad (4.3.23)$$

$$\rho_y = \frac{A_{sy}}{s b_c} \quad \ldots\ldots \quad (4.3.24)$$

where f_{yh} is yield strength of transverse reinforcement; w_i_ the i[th] clear distance between adjacent longitudinal bars; b_c and dc core dimensions to centerlines of hoop in x and y direction (where $b \geq d$), respectively; A_{sx} and A_{sy} are the total area of transverse bars in x and y direction, respectively.

Once f'1x and f'1y are determined, the confined concrete strength f'_{cc} can be found using the chart shown in **Figure 4.3-5** with f'_{1x} being greater or equal to f'_{1y} . The chart depicts the general solution of the "five-parameter" multi-axial failure surface described by William and Warnke (1975).

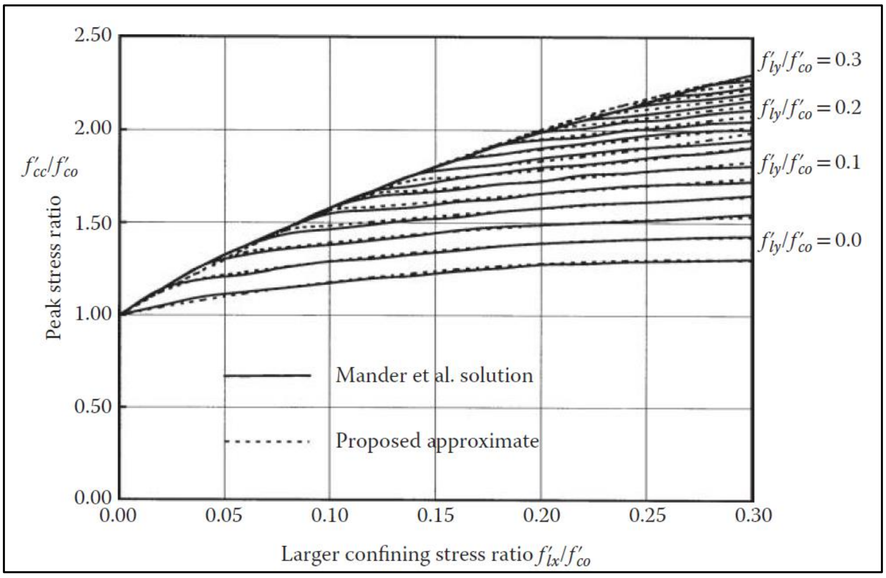


Figure 4.3-5 Peak stress of confined concrete. (Chen et. al 2014)

Note that setting $f'_1 = 0.0$ in Equations 4.3.17, 4.2.20, and 4.3.21 will produce Mander's expression for unconfined concrete. In this case and for concrete strain $\varepsilon_c > 2\varepsilon_{co}$, a straight line that reaches zero stress at the spalling strain ε_{sp} is assumed.

4.3.2.4 Confined Concrete Ultimate Compressive Strain

Defining the ultimate compressive strain as the longitudinal strain at which the first confining hoop fracture occurs, and using the energy balance approach, Mander et al. (1984) produced an expression for predicting the ultimate compressive strain that can be solved numerically. A conservative and simple equation for estimating the confined concrete ultimate strain is given by Priestley et al. (1996).

$$\varepsilon_{cu} = 0.004 + \frac{1.4\rho_s f_{yh}\varepsilon_{su}}{f'_{cc}} \quad \text{(4.3.25)}$$

where ε_{su} is the steel strain at maximum tensile stress for rectangular section $\rho_s = \rho_x + \rho_y$ as defined previously. Typical values for ε_{cu} range from 0.012 to 0.05. Equation 4.3.25 is formulated for confined sections subjected to axial compression. It is noted that according to (Chen and Duan 2014), when Equation 4.3.26 is used for section in bending or combined bending and axial then it tends to be conservative by at least 50%.

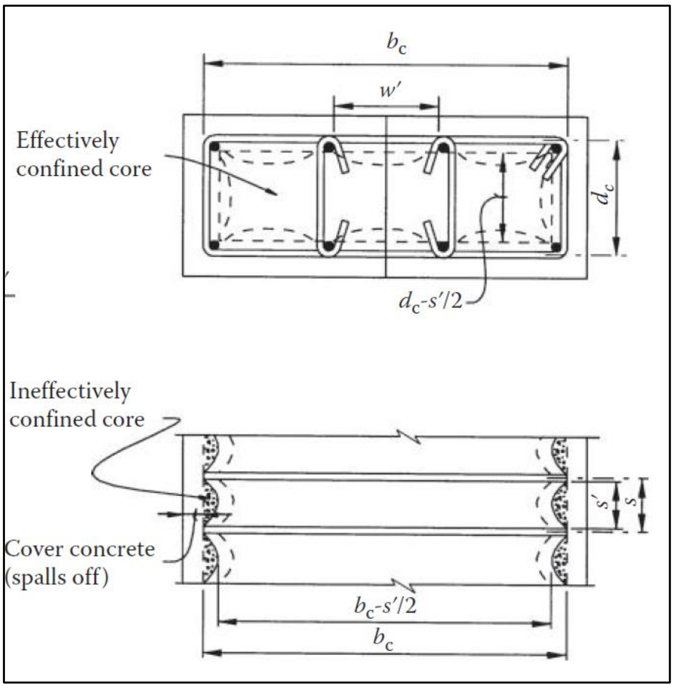


Figure 4.3-6 Confined core for rectangular hoop reinforcement (Chen et.al 2014)

4.3.2.5 Structural Steel and Reinforcement

For structural steel and non-prestressed steel reinforcement, its stress–strain relationship can be Idealized as four parts: elastic, plastic, strain hardening, and softening as shown in **Figure 4.3-7**. The simplest multilinear expression is

$$f_s = \begin{cases} E_s \varepsilon_s & 0 \le \varepsilon_s \le \varepsilon_y \\ f_y & \varepsilon_{sy} < \varepsilon_s \le \varepsilon_{sh} \\ f_y + \dfrac{\varepsilon_s - \varepsilon_{sh}}{\varepsilon_{su} - \varepsilon_{sh}}\left(f_u - f_y\right) & \varepsilon_{sh} < \varepsilon_s \le \varepsilon_{su} \\ f_u\left[1 - \dfrac{\varepsilon_s - \varepsilon_{su}}{\varepsilon_{sb} - \varepsilon_{su}}\left(f_{su} - f_{sb}\right)\right] & \varepsilon_{su} < \varepsilon_s \le \varepsilon_{sb} \end{cases} \quad \text{......} \quad (4.3.26)$$

where fs and ε_s is stress of strain in steel; Es the modulus of elasticity of steel = 29,000 ksi (200, 000 MPa); fy and ε_y yield stress and strain; ε_{sh} hardening strain; fsu and ε_{su} maximum stress and corresponding strain; and fsb and ε_{sb} rupture stress and corresponding strain.
For the reinforcing steel, the following nonlinear form can also be used for the strain-hardening portion (Chai et al., 1990):

$$f_s = f_y\left[\frac{m(\varepsilon_s - \varepsilon_{sh}) + 2}{60(\varepsilon_s - \varepsilon_{sh}) + 2} + \frac{(\varepsilon_s - \varepsilon_{sh})(60 - m)}{2(30r+1)^2}\right] \quad \text{for } \varepsilon_{sh} < \varepsilon_s \le \varepsilon_{su} \quad \text{......} \quad (4.3.27)$$

$$m = \frac{(f_{su}/f_y)(30r+1)^2 - 60r - 1}{15r^2} \quad \text{......} \quad (4.3.28)$$

$$r = \varepsilon_{su} - \varepsilon_{sh}$$

$$\text{......} \quad (4.3.29)$$

$$f_{su} = 1.5 f_y$$

$$\varepsilon_{sh} = \begin{cases} 14\varepsilon_y & \text{for Grade 40} \\ 5\,\varepsilon_y & \text{for Grade 60} \end{cases} \quad \text{......} \quad (4.3.30)$$

$$\varepsilon_{su} = \begin{cases} 0.14 + \varepsilon_{sh} & \text{for Grade 40} \\ 0.12 & \text{for Grade 60} \end{cases} \quad \text{......} \quad (4.3.31)$$

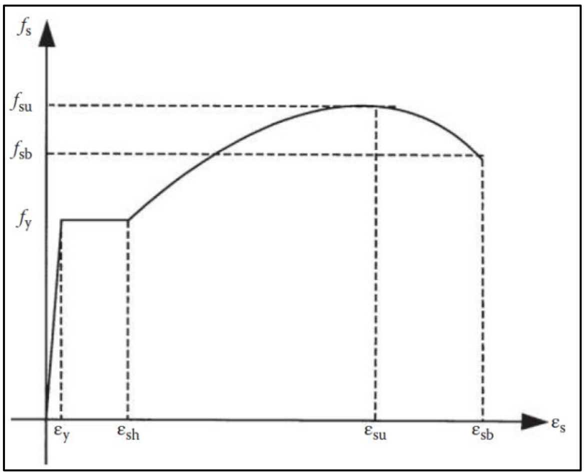


Figure 4.3-7 Idealized stress-strain curve of structural steel and reinforcement. (Chen el al. 2014)

For both strain-hardening and softening portions, Holzer et al. (1975) proposed the following expression:

$$f_s = f_y\left[1+\frac{\varepsilon_s-\varepsilon_{sh}}{\varepsilon_{su}-\varepsilon_{sh}}\left(\frac{f_u}{f_y}-1\right)\exp\left(1-\frac{\varepsilon_s-\varepsilon_{sh}}{\varepsilon_{su}-\varepsilon_{sh}}\right)\right] \quad \text{for } \varepsilon_{sh}<\varepsilon_s\le\varepsilon_{sb} \qquad \text{……………… (4.3.32)}$$

The nominal limiting values for stress and strain proposed by Holzer et al. (1975) as shown:

Table 4.3-3 Nominal Limiting Values for Structural Steel Stress–Strain Curves (Chen et. al 2014)

f_y ksi (MPa)	f_u ksi (MPa)	ε_y	ε_{sh}	ε_{su}	ε_{sb}
40 (280)	80 (550)	0.00138	0.0230	0.140	0.200
60 (420)	106 (730)	0.00207	0.0060	0.087	0.136
75 (520)	130 (900)	0.00259	0.0027	0.073	0.115

4.3.3 Plastic Hinges

The equivalent plastic hinge length, L_p as defined in FHWA Retrofitting Manual 2011 is given by semi-empirical equation below:

$$L_P = 0.08L + 4400\varepsilon_y d_b \quad \text{in mm} \qquad \text{………………………………………………. (4.3.33)}$$

where db and ε_y are the diameter and yield strain of the longitudinal tension reinforcement respectively, and L is the shear span or effective height. Another approached of estimating the length of plastic hinge (Priestley et al. 2007) is to use a simplified approach based on the concept of a "plastic hinge", of length $\mathbf{L_p}$, over which strain and curvature are considered to be equal to the maximum value at the column base. The plastic hinge length incorporates the strain penetration length $\mathbf{L_{sp}}$ as shown in **Figure 4.3-8**. Further, the curvature distribution higher up the column is assumed to be linear, in accordance with the bilinear approximation to the moment-curvature response. This tends to compensate for the increase in displacement resulting from tension shift, and, at least partially, for shear deformation. The strain penetration length, $\mathbf{L_{sp}}$ may be taken as:

$$L_{sp} = 0.022 f_{ye} d_{bl} \quad f_{ye} \quad \text{in Mpa} \qquad \text{…………………………………………………… (4.3.34)}$$

Where *fye* and *dbl* are the expected yield strength and diameter of the longitudinal reinforcement.

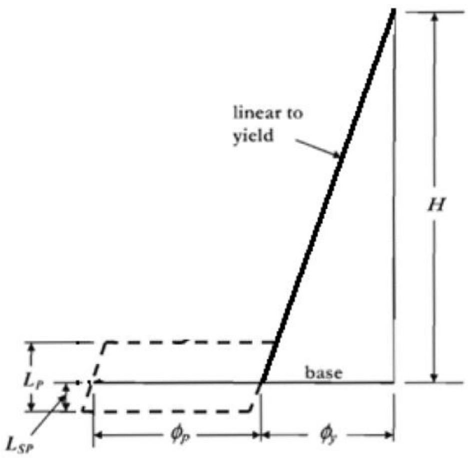


Figure 4.3-8 Idealization of curvature distribution –
[Ref: Priestly, M.J.N. Calvi G.M. Kowalsky M.J. (2007)]

and the plastic hinge length of column, Lp is given by:

$$L_P = kL_c + L_{sp} \geq 2L_{sp} \quad \text{……………………………………………………………. (4.3.35)}$$

where:

$$k = 0.2\left(\frac{f_u}{f_y} - 1\right) \leq 0.08 \quad \text{…………………………………………………………. (4.3.36)}$$

and where Lc is the length from the critical section to the point of contra-flexure in the member. Equation (4.3.36) emphasis the importance of the ratio of ultimate tensile strength to yield strength of the flexural reinforcement. If this value is high, plastic deformations spread away from the critical section as the reinforcement at the critical section strain-hardens, increasing the plastic hinge length. If the reinforcing steel has a low ratio of ultimate to yield strength, plasticity concentrates close to the critical section, resulting in a short plastic hinge length.

4.3.4 Multiple Support Excitation

In a structure with multiple supports, different time history forcing functions in terms of ground acceleration can be applied to different supports. In cases of long-span bridges (suspension bridge or cable stayed bridge), when the distance between the supports of a substructure is large, arrival time of seismic excitation varies. This effect can be considered using the "Multiple Support Excitation" function. The response of the bridge under multiple-support ground motions is generally different from those excited by identical support ground motion, because multiple-support ground motions may excite vibration modes not captured by using uniform support ground motions, and vice versa. The relative deviation is more severe for longer spans.

For the analysis of such systems the formulation of Section 4.3.2 is extended to include the degrees of freedom at the supports (**Figure 4.3-9**). The displacement vector now contains two parts: (1) **u***t* includes the *N* DOFs of the superstructure, where the superscript *t* denotes that these are total displacements; and (2) **u**g contains the Ng components of support displacements. The equation of dynamic equilibrium for all the DOFs is written in partitioned form:

$$\begin{bmatrix} m & m_g \\ m_g^T & m_{gg} \end{bmatrix}\begin{Bmatrix} \ddot{u}^t \\ \ddot{u}_g \end{Bmatrix}+\begin{bmatrix} c & c_g \\ c_g^T & c_{gg} \end{bmatrix}\begin{Bmatrix} \dot{u}^t \\ \dot{u}_g \end{Bmatrix}+\begin{bmatrix} k & k_g \\ k_g^T & k_{gg} \end{bmatrix}\begin{Bmatrix} u^t \\ u_g \end{Bmatrix}=\begin{Bmatrix} 0 \\ p_g(t) \end{Bmatrix} \quad \text{(4.3.37)}$$

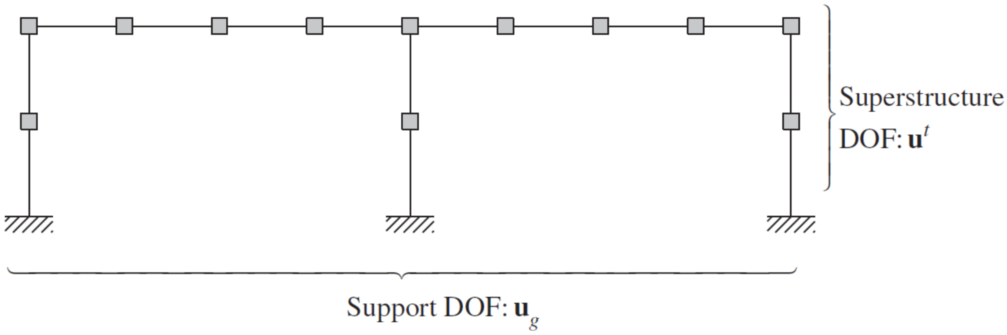


Figure 4.3-9 Definition of superstructure and support DOFs. (Chopra 2012)

In Eq. (4.3.37) the mass, damping, and stiffness matrices can be determined from the properties of the structure using the procedures presented earlier in this chapter, while the support motions $u_g(t)$, $\dot{u}_g(t), \ddot{u}_g(t)$ must be specified. It is desired to determine the displacements **u**t in the superstructure DOF and the support forces **p**g.

To write the governing equations in a form familiar from the earlier formulation for a single excitation, then:

$$\begin{Bmatrix} u^t \\ u_g \end{Bmatrix}=\begin{Bmatrix} u^s \\ u_g \end{Bmatrix}+\begin{Bmatrix} u \\ 0 \end{Bmatrix} \quad \text{(4.3.38)}$$

In this equation **u**s is the vector of structural displacements due to static application of the prescribed support displacements **u**g at each time instant. The two are related through

$$\begin{bmatrix} k & k_g \\ k_g^T & k_{gg} \end{bmatrix}\begin{Bmatrix} u^s \\ u_g \end{Bmatrix}=\begin{Bmatrix} 0 \\ p_g^s \end{Bmatrix} \quad \text{(4.3.39)}$$

where p_g^s are the support forces necessary to statically impose displacements ug that vary with time; obviously, u_s varies with time and is therefore known as the vector of quasi-static displacements.

With the total structural displacements split into quasi-static and dynamic displacements, Eq. (4.3.38), we return to the first of the two partitioned equations (4.3.38):

$$\mathbf{m}\ddot{\mathbf{u}}^t + \mathbf{m}_g\ddot{\mathbf{u}}_g + \mathbf{c}\dot{\mathbf{u}}^t + \mathbf{c}_g\dot{\mathbf{u}}_g + \mathbf{k}\mathbf{u}^t + \mathbf{k}_g\mathbf{u}_g = \mathbf{0} \quad \text{(4.3.40)}$$

Substituting Eq. (4.3.38) and transferring all terms involving **u**g and **u**s to the right side leads to

$$m\ddot{u} + c\dot{u} + ku = p_{eff}(t) \quad \text{……} \quad (4.3.41)$$

where the vector of effective earthquake forces is

$$\mathbf{p}_{\text{eff}}(t) = -(\mathbf{m}\ddot{\mathbf{u}}^s + \mathbf{m}_g\ddot{\mathbf{u}}_g) - (\mathbf{c}\dot{\mathbf{u}}^s + \mathbf{c}_g\dot{\mathbf{u}}_g) - (\mathbf{k}\mathbf{u}^s + \mathbf{k}_g\mathbf{u}_g) \quad \text{……} \quad (4.3.42)$$

This effective force vector can be rewritten in a more useful form. The last term drops out because Eq. (4.3.39) gives

$$\mathbf{k}\mathbf{u}^s + \mathbf{k}_g\mathbf{u}_g = \mathbf{0} \quad \text{……} \quad (4.3.43)$$

This relation also enables us to express the quasi-static displacements u_s in terms of the specified support displacements u_g:

$$u^s = \mathrm{I}u^g \qquad \mathrm{I} = -k^{-1}k_g \quad \text{……} \quad (4.3.44)$$

We call I the *influence matrix* because it describes the influence of support displacements on the structural displacements. Substituting Eqs. (4.3.43) and (4.3.44) in Eq. (4.3.42) gives

$$p_{eff}(t) = -(m\mathrm{I} + m_g)\ddot{u}_g(t) - (c\mathrm{I} + c_g)\dot{u}_g(t) \quad \text{……} \quad (4.3.45)$$

If the ground (or support) accelerations $\ddot{u}_g(t)$ and velocities $\dot{u}_g(t)$ are prescribed, $p_{eff}(t)$ is known from Eq. (4.3.45), and this completes the formulation of the governing equation [Eq. (4.3.41)].

Simplifying eqn. (4.3.45) since in practical application if the damping matrix are proportional to the stiffness matrix the damping term may approximately zero, hence,

$$p_{eff}(t) = -m\mathrm{I}\ddot{u}_g(t) \quad \text{……} \quad (4.3.47)$$

And for structures structure with multiple support motions

$$p_{eff}(t) = -\sum_{l=1}^{N_g} m\mathrm{I}_l\ddot{u}_{gl}(t) \quad \text{……} \quad (4.3.48)$$

The *l*th term in Eq. (4.3.47) that denotes the effective earthquake forces due to acceleration in the *l*th support DOF is of the same form for structures with single support (and for structures with identical motion at multiple supports). The two cases differ in an important sense, however: In the latter case, the influence vector can be determined by kinematics, but *N* algebraic equations [Eq. (4.3.47)] are solved to determine each influence vector I_l for multiple-support excitations.

CHAPTER 5: ANALYSIS EXAMPLE

Chapter 5 Analysis Example

This chapter consist of basic example of the seismic analysis of continuous bridge subject to earthquake loadings. Both response spectrum method and elastic time history analysis method has been employed in this exercise. The outline of the analysis will be as follows:

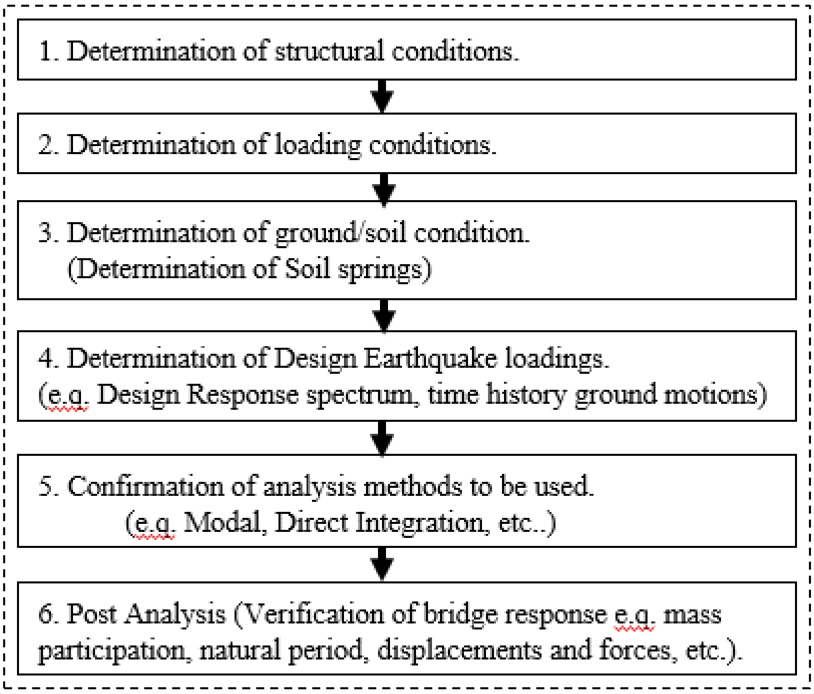


Figure 5-1 Outline of Analysis

5.1 Analysis Modelling

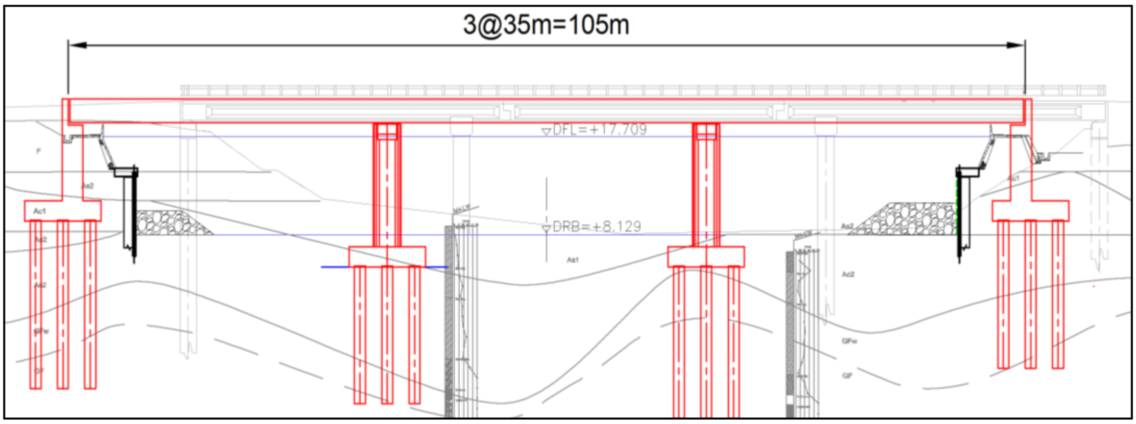
Quality of mathematical model as well as loading model according to actual condition and configuration based on particular guidelines is necessary.
The criteria for the seismic analysis and design of example bridge was principally conducted in accordance with provisions of Bridge Seismic Design Specifications (BSDS). In case of necessity of additional design criteria, basically, DGCS 2015 and AASHTO LRFD (6th edition) was referred to.

5.1.1 Structural Conditions

Structural conditions were set as follows. In regard with this, bridge profile and superstructure cross section are shown in **Figure 5.1-1**.

- Bridge type: 3 span continuous AASHTO girders – Type V
- Bridge length and span length: 35.0+35.0+35.0 = 105.0 (m)
- Total road width: 1.5 + 0.6 + 3.15 + 3.15 + 0.6 + 1.5 = 10.5 (m)
- Skew angle: 90 degrees (non-skewed straight bridge)
- Pier type: Single circular column
- Abutment type: Cantilever type
- Foundation type: Cast-in-place concrete pile (CCP) foundation (φ1200)
- Centroid of the superstructure: 1.8m from the column top (application point of superstructure mass)

– Bearing Restraint Condition: Longitudinal direction: M F F M (A1, P1, P2, and A2, respectively)
Transverse direction: F F F F (A1, P1, P2, and A2, respectively)
Note: M: movable, F: fixed



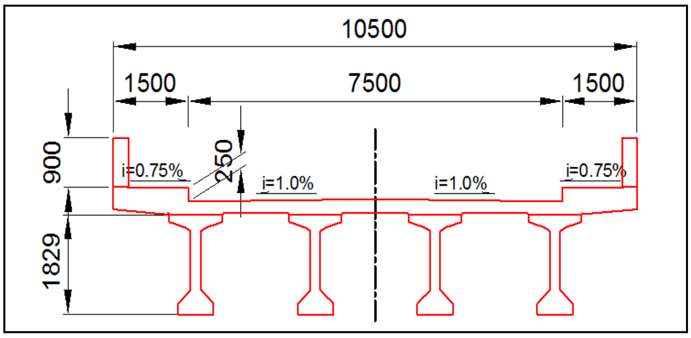


Figure 5.1-1 Bridge profile and Superstructure cross section

5.1.2 Bridge Importance (Bridge Operational Classification) (BSDS-Article 3.2)

Example Bridge is classified as "Other Bridge", as shown in **Table 5.1-1**. Considering the operational classification requirement, following two (2) design conditions were set. Design seismic force "An earthquake with 1,000-year return period" was applied to the bridge seismic design force in consideration of an active fault near the bridge and the location of the bridge (in Metro Manila) Response Modification Factors for Substructures (R-factor) A response modification factor (hereafter, called R-factor) for "Other Bridges" was applied to design of pier columns. As a relationship between "R-factor" and "Operational Category" is shown in **Table 5.1-2**, "**R=3.0**" was selected for design of single columns.

Table 5.1-1 Operational Classification of Bridges

Operational Classification (OC)	Performance
OC-I **(Critical Bridge)**	– Bridges that must remain open to all traffic after the design earthquake. – Other bridges required by DPWH to be open to emergency vehicles and vehicles for security/defense purposes immediately after an earthquake larger than the design earthquake.
OC-II **(Essential Bridge)**	– Bridges that should, as a minimum, be open to emergency vehicles and for security/defense purposes within a short period after the design earthquake. i.e. 1,000-year return period event.
OC-III **(Other Bridge) (Selected)**	– All other bridges not required to satisfy OC-I or OC-II performance

Table 5.1-2 Response Modification Factors, R

Substructure	Operational Category		
	Critical	**Essential**	**Others**
Wall-type piers – larger dimension	1.5	1.5	2.0
Reinforced concrete pile bents • Vertical piles only • With batter piles	 1.5 1.5	 2.0 1.5	 3.0 2.0
Single columns	1.5	2.0	3.0
Steel or composite steel and concrete pile bents • Vertical piles only • With batter piles	 1.5 1.5	 3.5 2.0	 5.0 3.0
Multiple column bents	1.5	3.5	5.0

Pertaining to seismic performance of the bridge, seismic design of sample Bridge was conducted, targeting seismic performance level 3 (SPL-3) against large earthquakes with a 1000-year return period. The definition of SPL-3 is shown in **Table 5.1-3**.

Table 5.1-3 Earthquake Ground Motion and Seismic Performance

Earthquake Ground Motion (EGM)	**Bridge Operational Classification**		
	OC-I (Critical Bridges)	**OC-II (Essential Bridges)**	**OC-III (Other Bridges)**
Level 1 (Small to moderate earthquakes which are highly probable during the bridge service life)	SPL-1 *(Keep the bridge sound function; resist seismic forces within elastic limit)*	SPL-1 *(Keep the bridge sound function; resist seismic forces within elastic limit)*	SPL-1 *(Keep the bridge sound function; resist seismic forces within elastic limit)*
Level 2 (Large earthquakes with a 1,000-year return period)	SPL-2 *(Limited seismic damage and capable of immediately recovering bridge functions without structural repair)*	SPL-2 *(Limited seismic damage and capable of recovering bridge function with structural repair within short period)*	SPL-3 *(May suffer damage but should not cause collapse of bridge or any of its structural elements)*

5.1.3 Material Properties

5.1.3.1 Material Properties shown in **Table 5.1-4** were applied in the design.

Table 5.1-4 Material Properties

Material	Strength	Remarks
Concrete	fc'= 41.0 (MPa); Compressive Strength at 28 days	- $E_c = 0.043 * \gamma_c^{1.5} * \sqrt{fc'} = 31{,}000$ (MPa) (rounded down) Note: γc= 2350 (kg/m3): unit weight of concrete - Applied to PC I-girders
	fc'= 28.0 (MPa); Compressive Strength at 28 days	- $E_c = 4800\sqrt{fc'} = 25{,}000$ (MPa) (rounded down) - Applied to all the substructure members and deck slab
Rebar	Fy= 414 (N/mm2); Grade60 steel	- Applied to all the substructure members - Applicable diameter: D16, D20, D22, D25, D28, D32, D36

5.1.3.2 Unit Weight

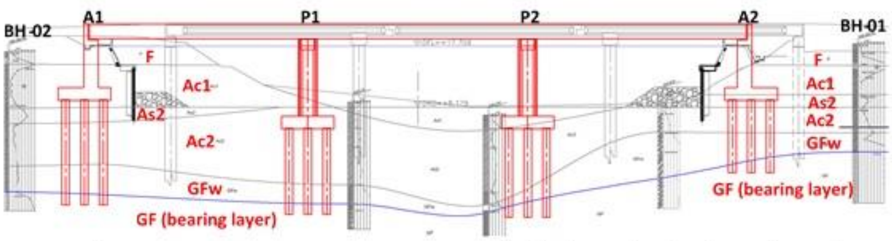
The following unit weights were applied in the design.

- Reinforced concrete: γc= 24.0 (kN/m3); rounded up for modification
- Water: γw= 10.0 (kN/m3)
- Soil (wet): γt= (result of soil tests) (kN/m3)
- Soil (saturated): γsat= γt+1.0 (kN/m3)
- Soil (backfill): γs= 19.0 (kN/m3)

5.1.4 Ground Conditions (BSDS-Article 3.5.1)

(1) Outline of Ground Conditions

The ground of the bridge site consists of seven (7) types of layers. Out of the seven layers, Guadalupe Formation (GF), which is classified as soft rock, has been selected as bearing layer of the site. Ground profile and soil parameters are shown in **Figure 5.1-2**



Ground Layer		N blows	γt kN/m3	C kN/m2	φ degrees	qu MPa
F	Embankment	24	17	-	42	-
As1	Gravely sand	-	-	-	-	-
Ac1	Silty clay	9	17	18	-	0.037
As2	Gravely sand	32	18	-	44	-
Ac2	Silty clay	21	18	18	-	-
GFw	Guadalupe formation (severely weathered)	36	19	-	46	-
GF	Guadalupe formation (soft rock)	50	20	-	46	4.83

Figure 5.1-2 Geological Profile and Soil Parameters

(2) Soil Type Classification

Ground types of the bridge site was classified as “Ground Type-II” in accordance with criteria defined in **Table 5.1-5**, in which ground characteristic value, T_G, defined by the following equation, was used as evaluation index **Figure 5.1-3** shows detail of the evaluation at two (2) boring locations (BH-01 and BH-02).

$$T_G = 4\sum_{i=1}^{n}\frac{H_i}{V_{si}}$$

where,

T_G : Characteristic value of ground (s)

Hi : Thickness of the i-th soil layer (m)

V_{si} : Average shear elastic wave velocity of the i-th soil layer (m/s)

i : Number of the i-th soil layer from the ground surface when the ground is classified into “n (No.) layers” from the ground surface to the surface of the base ground surface for seismic design.

Table 5.1-5 Ground Types (Site Class) for Seismic Design

Ground Type		Characteristic Value of Ground, T_G (s)
Type-I	Good diluvial ground and rock	$T_G < 0.2$
Type-II	Diluvial and alluvial ground not belonging to either Type-III or Type-I ground	$0.2 \leqq T_G < 0.6$
Type-III	Soft ground and alluvial ground	$0.6 \leqq T_G$

BH-01

Layer Name	Layer Type	Layer thickness Hi (m)	N-value	Vsi (m/s)	Hi/Vsi (s)
F	Clay	4.0	37.7	292.4	0.0137
Ac1	Clay	4.0	8.7	205.7	0.0194
As2	Sand	2.0	17.7	208.5	0.0096
Ac2	Clay	4.0	35.3	292.4	0.0137
GFw	Rock	8.0	44.5	283.5	0.0282
		$T_G = 4*\Sigma$(H/Vs)			0.338
		Soil Type			Type-II

Soil type	Definition
Type-I	$T_G<0.2$
Type-II	$0.2 \leq T_G<0.6$
Type-III	$0.6 \leq T_G$

BH-02

Layer Name	Layer Type	Layer thickness Hi (m)	N-value	Vsi (m/s)	Hi/Vsi (s)
F	Clay	3.0	22.7	283.1	0.0106
Ac1	Clay	5.0	7.0	191.3	0.0261
As2	Sand	4.0	45.7	286.0	0.0140
Ac2	Clay	7.0	26.3	292.4	0.0239
GFw	Rock	4.0	22.7	283.1	0.0141
		$T_G = 4*\Sigma$(H/Vs)			0.355
		Soil Type			Type-II

Soil type	Definition
Type-I	$T_G<0.2$
Type-II	$0.2 \leq T_G<0.6$
Type-III	$0.6 \leq T_G$

Figure 5.1-3 Soil Type Classification

5.2 Response Spectrum Analysis

Response spectra were used to represent the seismic demand on structures due to a ground motion record and **design spectra** were used for the seismic design of structures.

5.2.1 Design Acceleration Response Spectra

BSDS 3.4.1 General Procedure

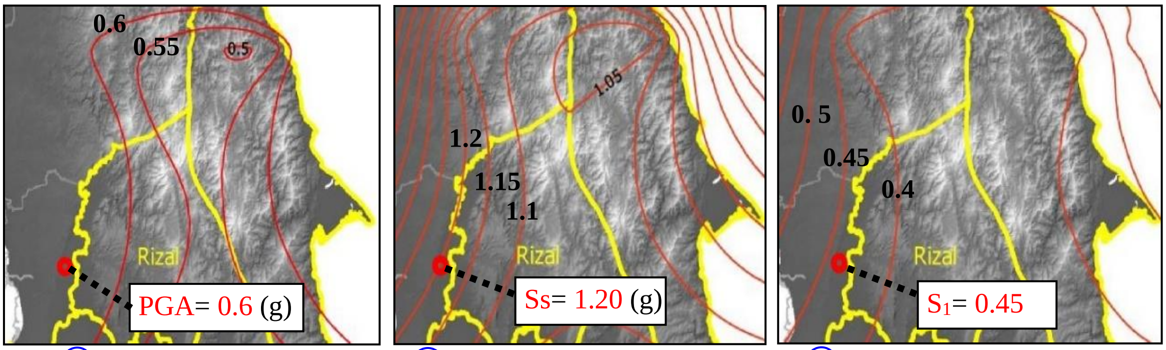
(1) The General Procedure shall use the peak ground acceleration coefficient (*PGA*) and the short and long-period spectral acceleration coefficients (*SS* and *S1* respectively) to calculate the design response spectrum as specified in Article 3.6.
The values of *PGA*, *SS* and *S1* shall be determined from the acceleration coefficient contour maps of Figures 3.4.1-1 to 3.4.1-3 for the Level 1 Earthquake Ground Motion and Figures 3.4.1-4 to 3.4.1-6 for Level 2 Earthquake Ground Motion of this Section for the entire Philippine archipelago and from Appendix 3A and 3B for the regional level acceleration coefficient contour maps as appropriate, or from site specific ground motion maps approved by the DPWH or the Owner.

(2) **For sites located between two contour lines, the higher value of the two-contour line shall be taken as the coefficient value.**

(3) The effect of ground type (site class) on the seismic hazard shall be as specified in Article 3.5.

(1) Identification of Acceleration Coefficients

By examination of acceleration contour maps for 1000-year return period earthquakes, specific values of three (3) acceleration coefficients were identified as PGA= 0.6, Ss= 1.20, and S_1= 0.45, respectively, as shown in **Figure 5.2-1.**



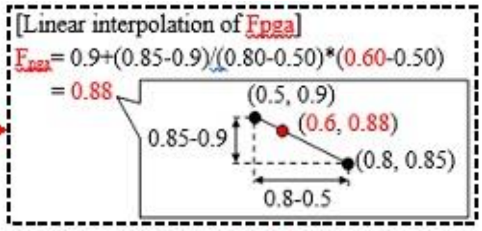
1 PGA Contour Map   2 Ss (0.2 sec.) Contour Map   3 S_1 (1.0 sec.) Contour Map

Figure 5.2-1 Acceleration Contour Maps

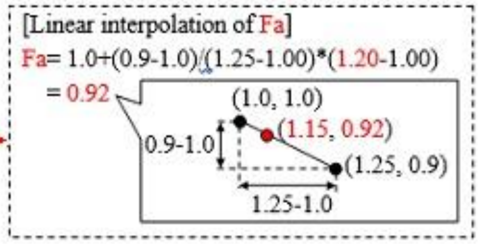
(2) Determination of Site Factors

Site factors of each acceleration coefficient were determined as Fpga= 0.88, Fa= 0.92, and Fv= 1.55, respectively, as shown in **Figure 5.2-2**.

	① Fpga (site factor for PGA)						
PGA (T=0)		PGA< 0.10	PGA= 0.20	PGA= 0.30	PGA= 0.40	PGA= 0.50	PGA≥ 0.80
Soil type	I	1.2	1.2	1.1	1.0	1.0	1.0
	II	1.6	1.4	1.2	1.0	0.9	0.85
	III	2.5	1.7	1.2	0.9	0.8	0.75



	② Fa (site factor for Ss)						
Ss (T=0.2)		Ss< 0.25	Ss= 0.50	Ss= 0.75	Ss= 1.00	Ss= 1.25	Ss≥ 2.0
Soil type	I	1.2	1.2	1.1	1.0	1.0	1.0
	II	1.6	1.4	1.2	1.0	0.9	0.85
	III	2.5	1.7	1.2	0.9	0.8	0.75



	③ Fv (site factor for S_1)						
S_1 (T=1.0)		S_1< 0.10	S_1= 0.20	S_1= 0.30	S_1= 0.40	S_1= 0.50	S_1≥ 0.80
Soil type	I	1.7	1.6	1.5	1.4	1.4	1.4
	II	2.4	2.0	1.8	1.6	1.5	1.5
	III	3.5	3.2	2.8	2.4	2.4	2.0

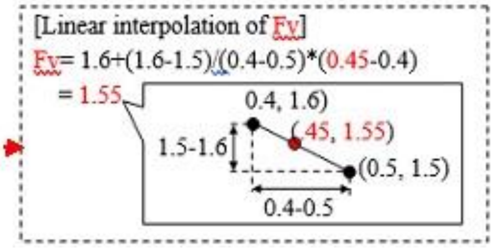


Figure 5.2-2 Site Factors

(3) Formulation of Design Acceleration Response Spectrum

The five-percent-damped-design response spectrum was formulated by the following two (2) steps, as shown in **Figure 5.2-3**

- Step.1: Calculate and plot the coordinates of the following points in the graph.
(0, Fpga*PGA), (0.2*Ts, Fa*Ss), (0.2, Fa*Ss), (S_{D1}/S_{DS}, Fa*Ss), (1.0, Fv*S_1)

PGA	:	peak horizontal ground acceleration coefficient
Ss	:	0.2-sec period spectral acceleration coefficient
S_1	:	1.0-sec period spectral acceleration coefficient
Fpga	:	site coefficient for peak ground acceleration
Fa	:	site coefficient for 0.2-sec period spectral acceleration
Fv	:	site coefficient for 1.0-sec period spectral acceleration

- Step.2: Form spectrum by connecting the plotted points with the following two (2) formulas.
  - Csm= As+(SDS-As)(Tm/T0) (if 0≤Tm≤Ts)
  - Csm= SD1/T (if Ts≤Tm

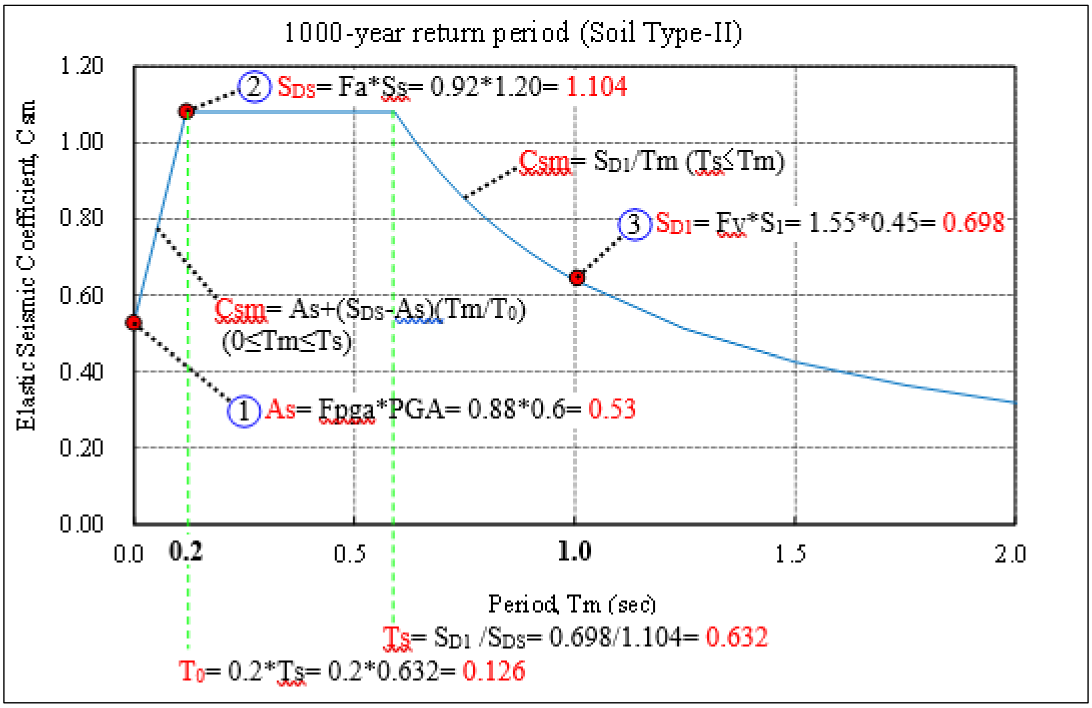


Figure 5.2-3 Design Acceleration Response Spectrum for the Design

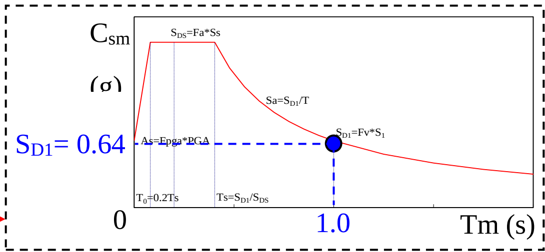
5.2.2 Analysis Requirements and Physical Modeling

(1) Seismic Performance Zone (BSDS-Article 3.7)

Since “S_{D1}” of the bridge site was 0.698 (g), seismic performance zone (SZ) of the site was categorized as SZ-4, as shown in **Table 5.2-1**.

Table 5.2-1 Seismic Performance Zone

Acceleration Coefficient, S_{D1}	Seismic Zone
$S_{D1} \leq 0.15$	SZ-1
$0.15 < S_{D1} \leq 0.30$	SZ-2
$0.30 < S_{D1} \leq 0.50$	SZ-3
$0.50 < S_{D1}$	SZ-4



(2) Analysis Requirements (for Multi-span Bridges) (BSDS-Article 3.2)

1) Regular Bridge Requirements

Sample Bridge satisfied all the regular bridge requirements shown in **Table 5.2-2**. The detail of requirement assessment is as follows.

- Number of spans: 3
- Bridge skew angle: 90 degrees
- Maximum span length ratio: 1.0:1.0
- Maximum pier stiffness ratio: 1.0:1.0

Table 5.2-2 Regular Bridge Requirements

Parameter	Value				
Number of Spans	2	3	4	5	6
Maximum subtended angle for a curved bridge	90°	90°	90°	90°	90°
Maximum span length ratio from span to span	3	2	2	1.5	1.5
Maximum bent/pier stiffness ratio from span to span, excluding abutments	-	4	4	3	2

2) According to "minimum analysis requirements for seismic effects" shown in **Table 5.2-3**, either "single-mode elastic method (SM)" or "uniform load elastic method (UL)" has only to be taken for seismic analysis. However, "multimode elastic method (MM)" was applied in order to clarify a design procedure of BSDS using the most typical analysis method in the Philippines.

Table 5.2-3 Minimum Analysis Requirements for Seismic Effects

Seismic Zone	Single-Span Bridges	Multispan Bridges					
		Other Bridges		Essential Bridges		Critical Bridges	
		Regular	Irregular	Regular	Irregular	Regular	Irregular
1	No seismic analysis required	*	*	*	*	*	*
2		SM/UL	SM	SM/UL	MM	MM	MM
3		SM/UL	MM	MM	MM	MM	MM/TH
4		SM/UL	MM	MM	MM	MM/TH	MM/TH

Where,

*	=	no seismic analysis required
UL	=	uniform load elastic method
SM	=	single-mode elastic method
MM	=	multimode elastic method
TH	=	time history method

3) Applied Analysis Methodologies

Three (3) dimensional response spectra analysis method (multimode elastic method) was applied for the seismic analysis under the following conditions.

a) Application Point of Superstructure Mass

Application point of superstructure mass was set at centroid of the superstructure. As shown in **Figure 5.2-4**, height of the application point is approximately1.8m from top surface of pier copings.

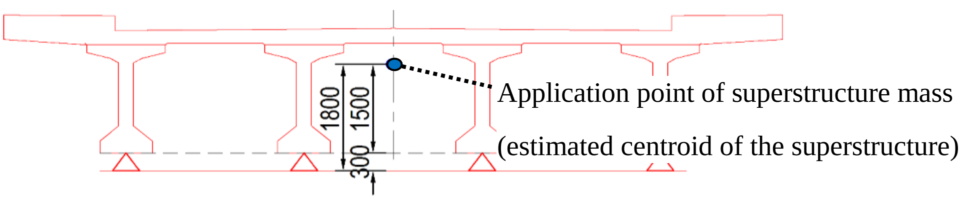
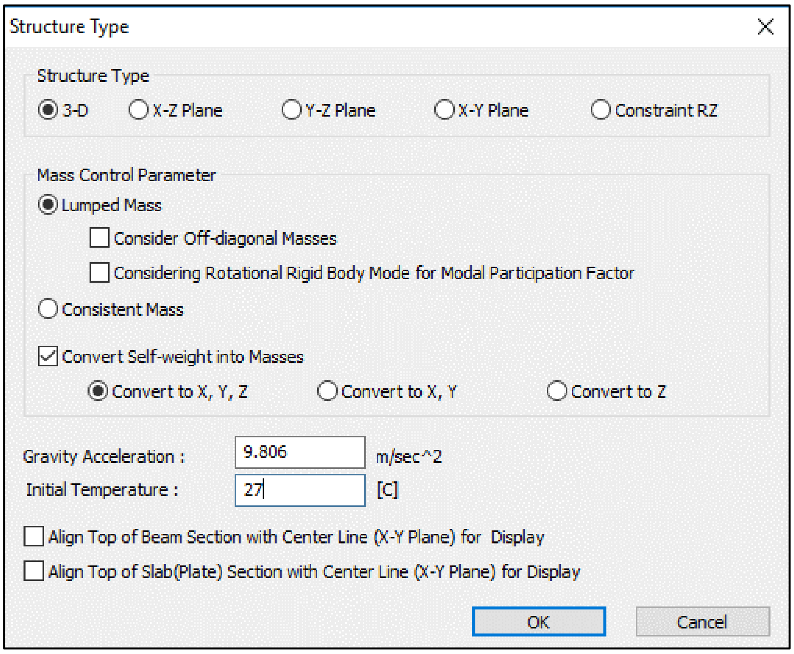


Figure 5.2-4 Application Point of Superstructure Mass

In dynamic analysis, it is important to define all the considered deadload that act on the structure during earthquake into equivalent mass. Some of commercial software has able to convert automatically by assigning each dead load (e.g. self-weight, nodal load, beam load, etc.) into equivalent masses. First option is to convert self-weight into mass as shown in **Figure 5.2-5** a and other deadload in **Figure 5.2-5** b. Other option is by defining it manually and assigning according to its actual location.



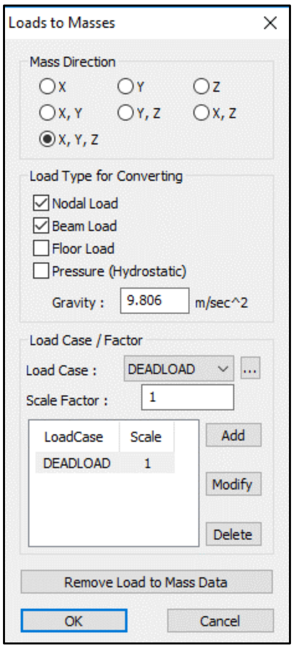


Figure 5.2-5 a) Convert self-weight to mass b) Convert other types of deadload to mass

b) Modeling of Bearings (Boundary Conditions at Bearings)

Degrees of freedom of movable and fixed bearings were modeled under the conditions shown in **Figure 5.2-6**.

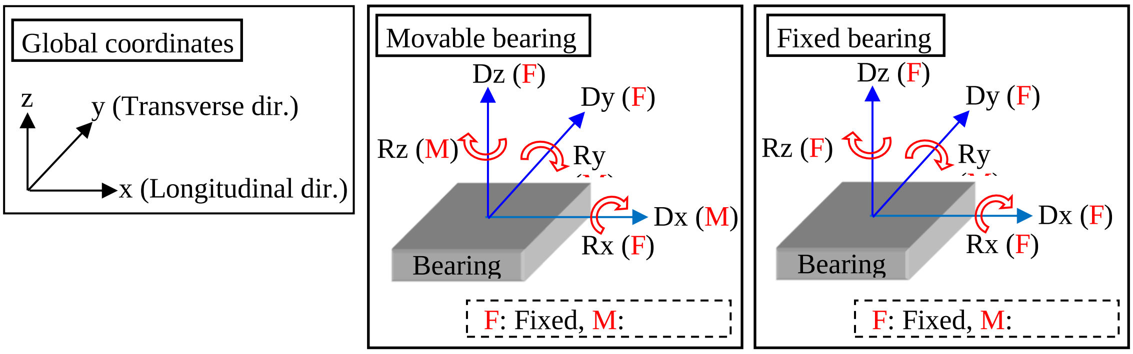
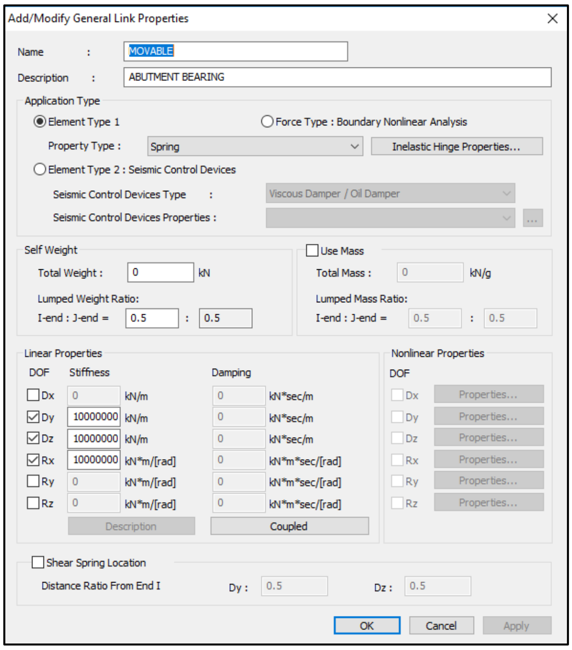


Figure 5.2-6 Degrees of freedom of Bearings

Bearing was modelled as a linear spring consider rigid in restrained direction by assigning high spring stiffness value in **Figure 5.2-7** a for movable bearing at the abutment and **Figure 5.2-7** b for fixed bearing at Piers.



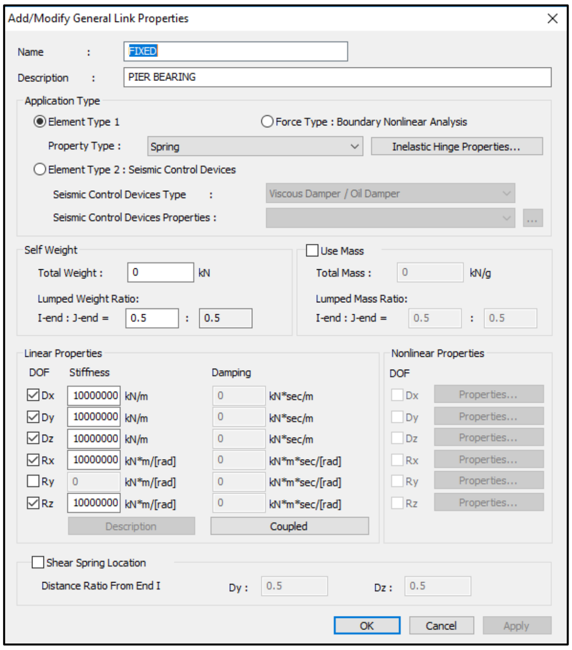


Figure 5.2-7 a) Movable bearing b) Fixed bearing

c) Pier/Column Stiffness in Analysis (BSDS-Commentary-C4.5.3)

"A moment of inertia equal to one-half that of the uncracked section" was adopted as "cracked section stiffness" in bridge analysis in the consideration of nonlinear effects which decrease stiffness. Image of cracked section stiffness is illustrated in **Figure 5.2-8** .

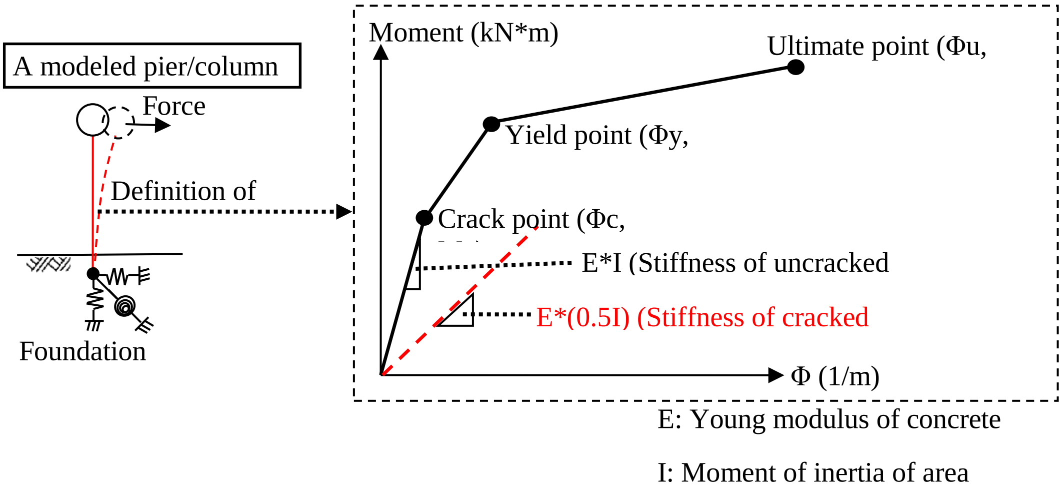
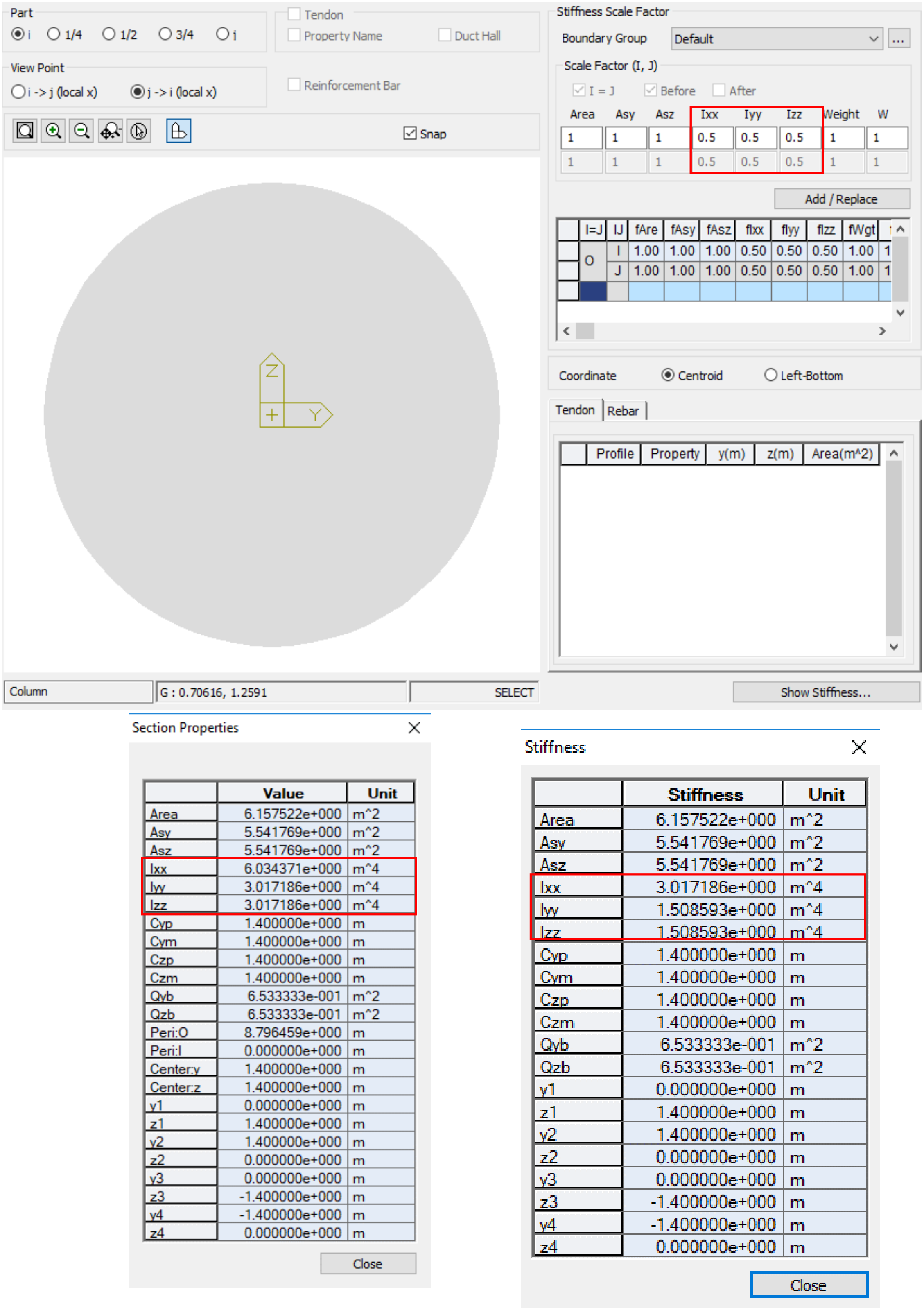


Figure 5.2-8 Image of Cracked Section Stiffness of Piers/Columns



Section Properties

	Value	Unit
Area	6.157522e+000	m^2
Asy	5.541769e+000	m^2
Asz	5.541769e+000	m^2
Ixx	6.034371e+000	m^4
Iyy	3.017186e+000	m^4
Izz	3.017186e+000	m^4
Cyp	1.400000e+000	m
Cym	1.400000e+000	m
Czp	1.400000e+000	m
Czm	1.400000e+000	m
Qyb	6.533333e-001	m^2
Qzb	6.533333e-001	m^2
Peri:O	8.796459e+000	m
Peri:I	0.000000e+000	m
Center:y	1.400000e+000	m
Center:z	1.400000e+000	m
y1	0.000000e+000	m
z1	1.400000e+000	m
y2	1.400000e+000	m
z2	0.000000e+000	m
y3	0.000000e+000	m
z3	-1.400000e+000	m
y4	-1.400000e+000	m
z4	0.000000e+000	m

Stiffness

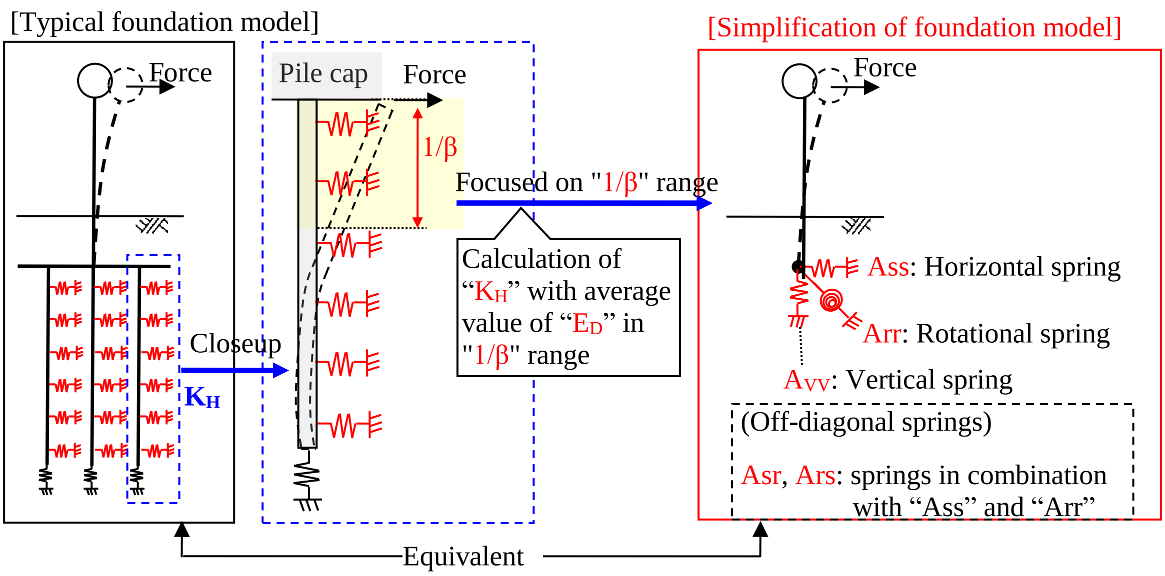
	Stiffness	Unit
Area	6.157522e+000	m^2
Asy	5.541769e+000	m^2
Asz	5.541769e+000	m^2
Ixx	3.017186e+000	m^4
Iyy	1.508593e+000	m^4
Izz	1.508593e+000	m^4
Cyp	1.400000e+000	m
Cym	1.400000e+000	m
Czp	1.400000e+000	m
Czm	1.400000e+000	m
Qyb	6.533333e-001	m^2
Qzb	6.533333e-001	m^2
y1	0.000000e+000	m
z1	1.400000e+000	m
y2	1.400000e+000	m
z2	0.000000e+000	m
y3	0.000000e+000	m
z3	-1.400000e+000	m
y4	-1.400000e+000	m
z4	0.000000e+000	m

Figure 5.2-9 a) Stiffness factor b) Uncracked section properties c) Cracked section properties

d) Dynamic Spring Properties of Pile Foundation (BSDS-Article 4.4.3)

Focusing on "1/β" range of pile foundation, which is the effective range of "K_H", foundation structure was modeled as group of springs in one node as shown in **Figure 5.2-10**.

Figure 5.2-10 Dynamic Spring Property of Pile Foundation a) Discrete pile model b) Lumped Spring model



In BSDS, for pile foundation there are two recommended modelling for analysis to determine the natural vibration periods under seismic load. It's either of the following:

a) Pile cap is modeled as a vertical element with piles represented by vertical, horizontal, and rotational springs lumped at the end node supports. The pile system is represented by foundation spring constants with properties considering all piles in the group. This is called as the simplified foundation model as shown in **Figure 5.2-10** b.

b) Another type of pile foundation model is called discrete model. In this model, the pile foundation is modeled using discrete elements representing the pile cap and pile body with corresponding stiffness and material properties. Vertical and horizontal springs are used at the nodal points in the piles to represents the ground resistance as shown in **Figure 5.2-10** a. This model is recommended when designing pile foundation using plastic hinging forces from columns.

If the effect of foundation on analyses is focused on "1/β" range, which is the effective range of "KH", foundation structure can be modeled as group of springs in one node as shown in **Figure 5.2-10**. If this method is applied, "KH" should be calculated with the average value of "ED" in "1/β" range,
"(ED) β". After the calculation of "KH" with "(ED) β", β can be obtained with the following equation.

$$\boxed{\beta = \sqrt[4]{\frac{K_H * D}{4 * E * I}}}$$

Then, if there's no pile projection over the ground surface, spring properties of a pile can be determined with the following equations; spring properties of a pile with rigid connection at the head

$$K1 = 4*E*I*\beta^3 \text{ (kN/m)}$$
$$K2 = K3 = 2*E*I*\beta^2 \text{ (kN/rad)}$$
$$K4 = 2*E*I*\beta \text{ (kN*m/rad)}$$
$$Kv = a*Ap*E/L \text{ (kN/m)}$$

Where,

K1, K3 : radical force (kN/m) and bending moment (kN*m/m) to be applied on a pile head when displacing the head by a unit volume in a radical direction while keeping it from rotation.

K2, K4 : radical force (kN/rad) and bending moment (kN*m/rad) to be applied to on a pile head when rotating the head by a unit volume while keeping it from moving in a radical direction.

Kv : axial spring constant of a pile

a : modification factor; with CCP, a= 0.031*(L/D)-0.15

L : pile length (m)

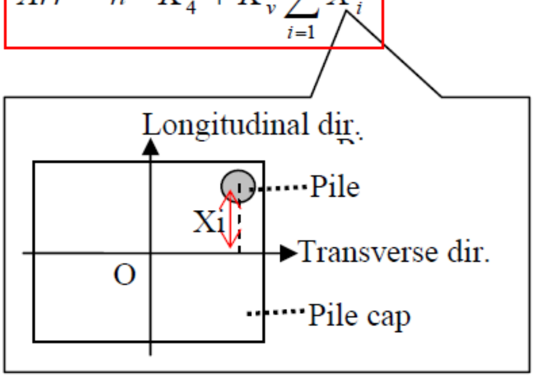
D : pile diameter (m)

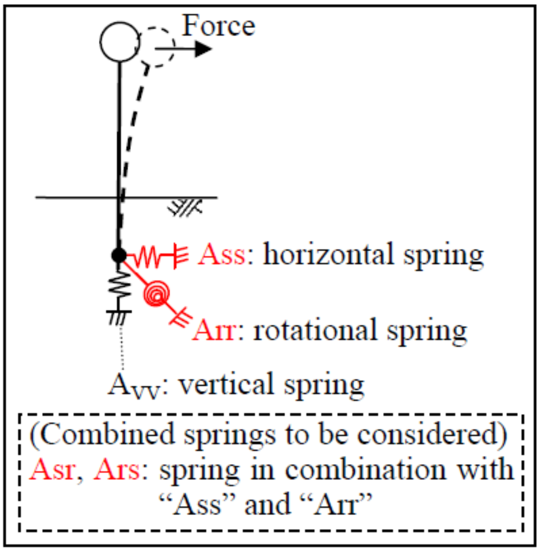
Ap : net cross-sectional area of a pile (mm2)

E : Young’s modulus of elasticity of the pile (kN/mm2)

Finally, spring properties of entire pile foundation can be determined with the following equations.

$$Ass = n * K_1$$
$$Asr = Ars = -n * K_2$$
$$Arr = n * K_4 + K_v \sum_{i=1}^{n} X_i^2$$





Note: the above equations can be applied only when there’re no battered piles.

Where,

Ass : horizontal spring property of the foundation structure (kN/m)

Asr, Ars : spring properties of the foundation structure in combination with “Ass” and “Arr” (kN/rad)

Arr : rotational spring property of the foundation structure (kN*m/rad)

n : number of piles in the foundation structure (nos)

Xi : X-coordinate of the i-th pile head (m)

Determination process of spring properties of foundation for bridge seismic analyses can be summarized with the following flowchart.

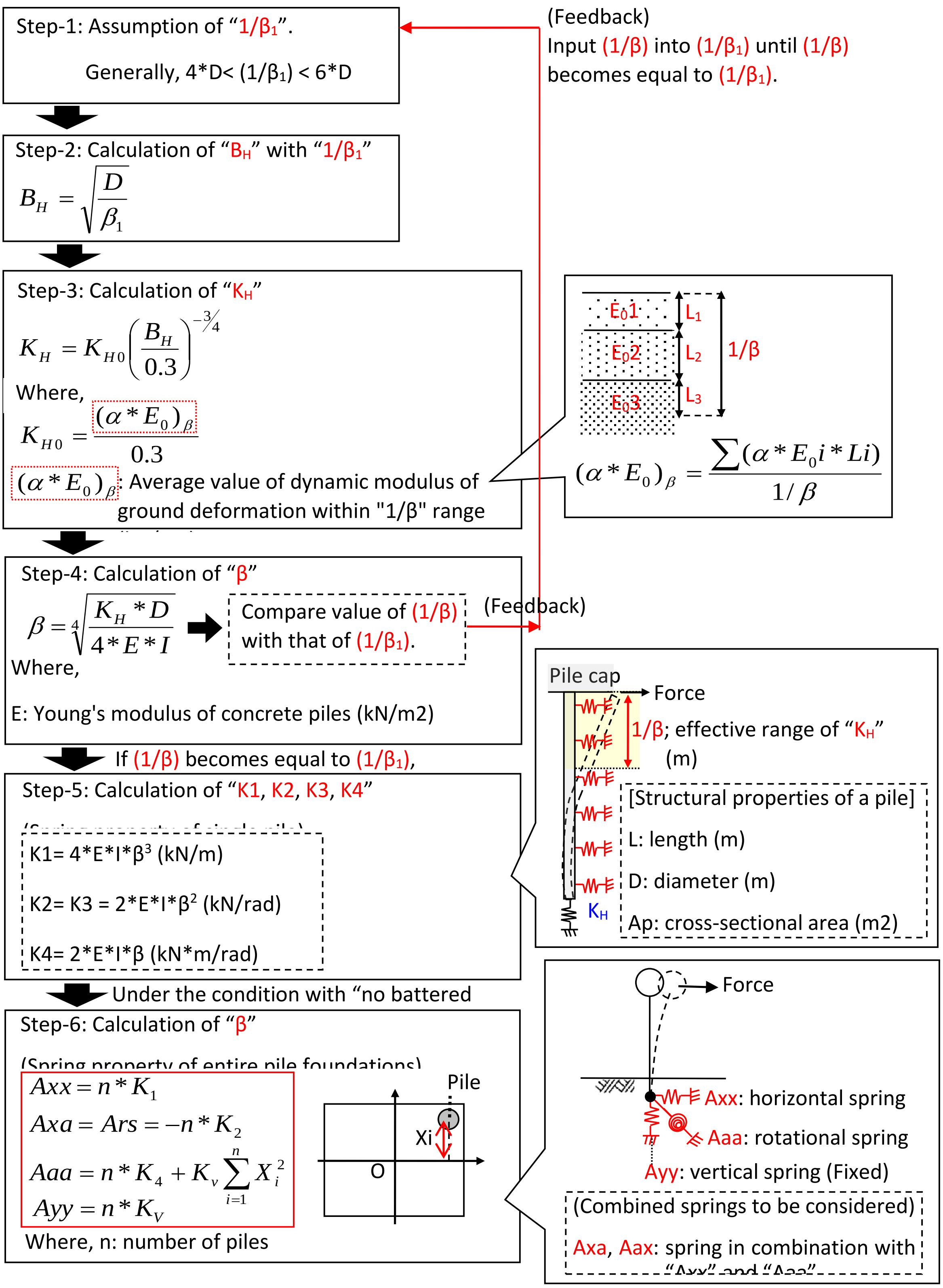


Figure 5.2-11 Determination Process of Dynamic Spring Property of Pile Foundation

In these examples, the simplified dynamic analysis model “lumped spring model was adopted during the analysis.

From borehole data at Pier foundation the following average N-value based on soil layer was obtained and the corresponding soil spring stiffness was calculated according to **Figure 5.2-11**.

FOR PIER 1

Layer symbol	Layer type	Layer thickness Li (m)	N-value	Vsi (m/s)	Cv	VsD (m/s)	γt (kN/m3)	G_D (kN/m2)	νD	E_D (kN/m2)
Ac	Clay	12.00	17	257	0.8	205	18.0	77188	0.5	231564
GFW	Clay	1.00	50	292	0.8	233	20.0	110793	0.5	332379

FOR PIER 2

Layer symbol	Layer type	Layer thickness Li (m)	N-value	Vsi (m/s)	Cv	VsD (m/s)	γt (kN/m3)	G_D (kN/m2)	νD	E_D (kN/m2)
Ac	Clay	11.00	15	247	0.8	197	18.0	71281	0.5	213843
GFW	Clay	1.00	50	292	0.8	233	20.0	110793	0.5	332379
GF	Sand	1.00	50	295	0.8	235	20.0	112704	0.5	338112

Pile spring stiffness, P1
Longitudinal/Transverse Direction

Type	Stiffness	Unit
Ass	3,771,748	(kN/m)
Asr,Ars	-4,774,365	(kN/rad)
Arr	37,961,700	(kN*m/rad)
Avv	3,236,400	(kN/m)

Pile spring stiffness, P2
Longitudinal/Transverse direction

Type	Stiffness	Unit
Ass	3,519,762	(kN/m)
Asr,Ars	-4,559,278	(kN/rad)
Arr	37,594,500	(kN*m/rad)
Avv	3,236,400	(kN/m)

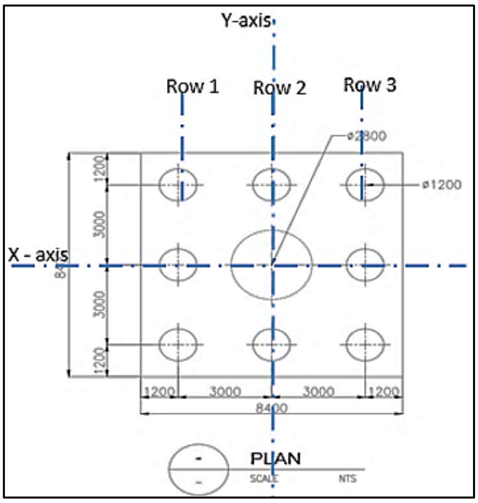


Figure 5.2-12 Piles foundation plan

The computed spring stiffness in this example both directions are same since the configuration of pile foundation as well as the number of piles is the same as shown in **Figure 5.2-12**.

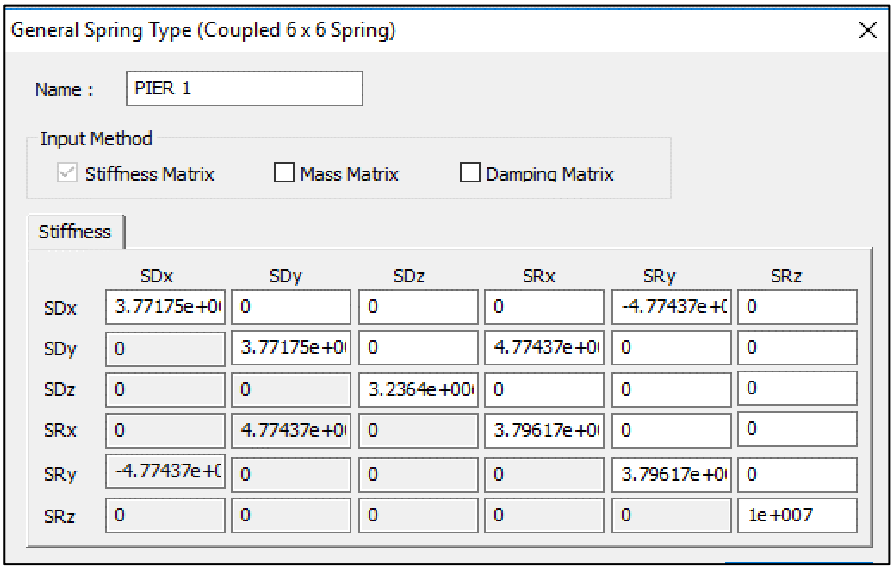
Consideration of off diagonal spring stiffness (Asr, Ars) was also employed in modelling of spring foundation as shown in equation below.

$$\begin{bmatrix} F_x \\ F_y \\ F_z \\ M_x \\ M_y \\ M_z \end{bmatrix} = \begin{bmatrix} K_{xx} & 0 & 0 & 0 & K_{Ryx} & 0 \\ 0 & K_{yy} & 0 & K_{Rxy} & 0 & 0 \\ 0 & 0 & K_{zz} & 0 & 0 & 0 \\ 0 & K_{Rxy} & 0 & R_x & 0 & 0 \\ K_{Ryx} & 0 & 0 & 0 & R_y & 0 \\ 0 & 0 & 0 & 0 & 0 & R_z \end{bmatrix} * \begin{bmatrix} U_x \\ U_y \\ U_z \\ \theta_x \\ \theta_y \\ \theta_z \end{bmatrix}$$

Where:

$Ass = K_{xx} = K_{yy}, \ and\ K_{zz}$ in $k\ N/m$
$Arr = \mathrm{R}_x = R_y$ and R_z in $kN.m/r\ ad.$
$Ars, Asr = -K_{Rxy} = K_{Ryx}$ in $k\ N/r\ ad.$

Applying to the analysis model by means of assigning a couple of springs as shown in **Figure 5.2-13**. See also C.4.4.3 Section 4.4.3 in BSDS 2013 for more detail.



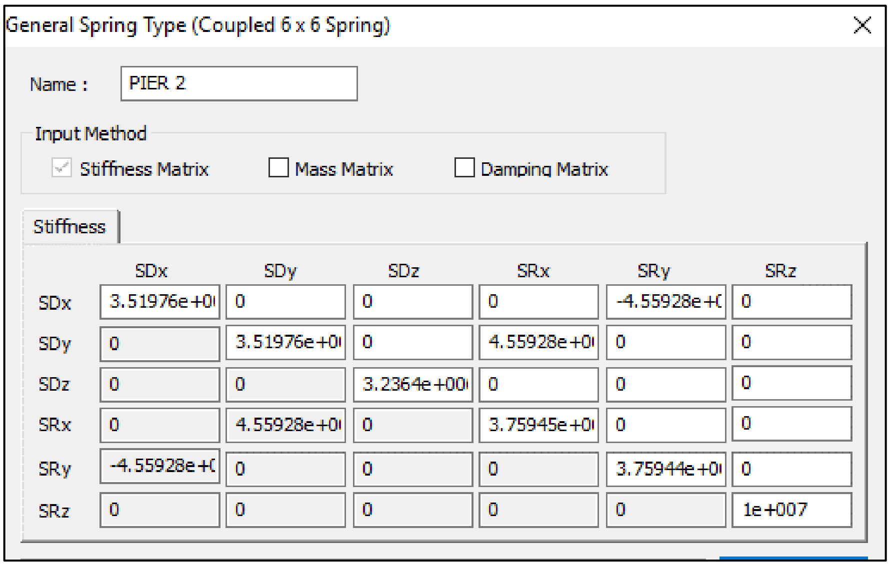


Figure 5.2-13 Soil Spring stiffness input in Midas Civil

e) Damping of Structures (BSDS-Article 4.5.4)

In dynamic analysis, damping of the structural members was given under the following conditions.

- Damping method: strain energy proportional damping
- Damping ratio: - Concrete: 2%
- Foundation: 20% (desirable value for “ground type II”)

Bridge physical model with lumped type foundation spring model in this example as shown in **Figure 5.2-13**.

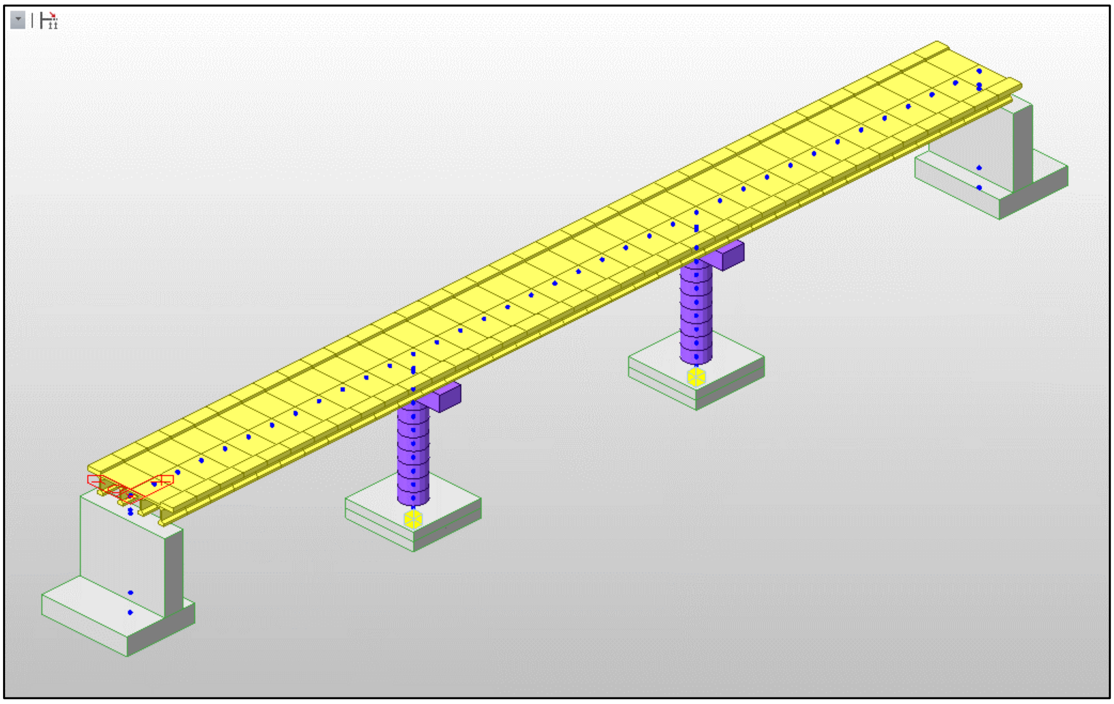


Figure 5.2-14 Dynamic Analysis Model of example Bridge in MIDAS Civil

5.2.3 Analysis Loading Model

5.2.3.1 Deadload

Deadload of the structure are composed of self-weight and superimposed deadload. In this example, superimposed deadload are composed of the following:

Wpost	=	24.3	kN
Wrail	=	134.4	kN
Wsidewalk	=	655.2	kN
Wwsurfsce	=	290	kN
We_diaphragm	=	166	kN
Wi_diaphragm	=	177	kN
Wshearblock	=	52	kN
Wblock_pier	=	10	kN
Wsi	=	**1509**	**kN**

5.2.3.2 Inertia effect of Live load During earthquake

BSDS C.4.4.1

"Generally, the inertia effects of live loads are not included in the analysis; however, the probability of a large live load being on the bridge during an earthquake should be considered when designing bridges with high live-to-dead load ratios which are located in metropolitan areas where traffic congestion is likely to occur". In this commentary, was mentioned the presence of live load during earthquake specially in metropolitan area like Manila. This clause also may be effective for viaduct which the probability of having vehicle at the bridge during earthquake is most likely to happen. In the analysis, it is not clear how much live load will be considered, but to be conservative 50% of live load effect was considered during earthquake as shown in **Figure 5.2-14**. This live load effect was converted into equivalent mass to perform as inertia force additional to dead load.

Wlaneload = 18.68 kN/m 50%Wlane = 9.34 kN/m Distributed throughout the span
Reaction of Governing Truck load = 1800 kN 50%Rtruck = 900kN Act at superstructure bearing support

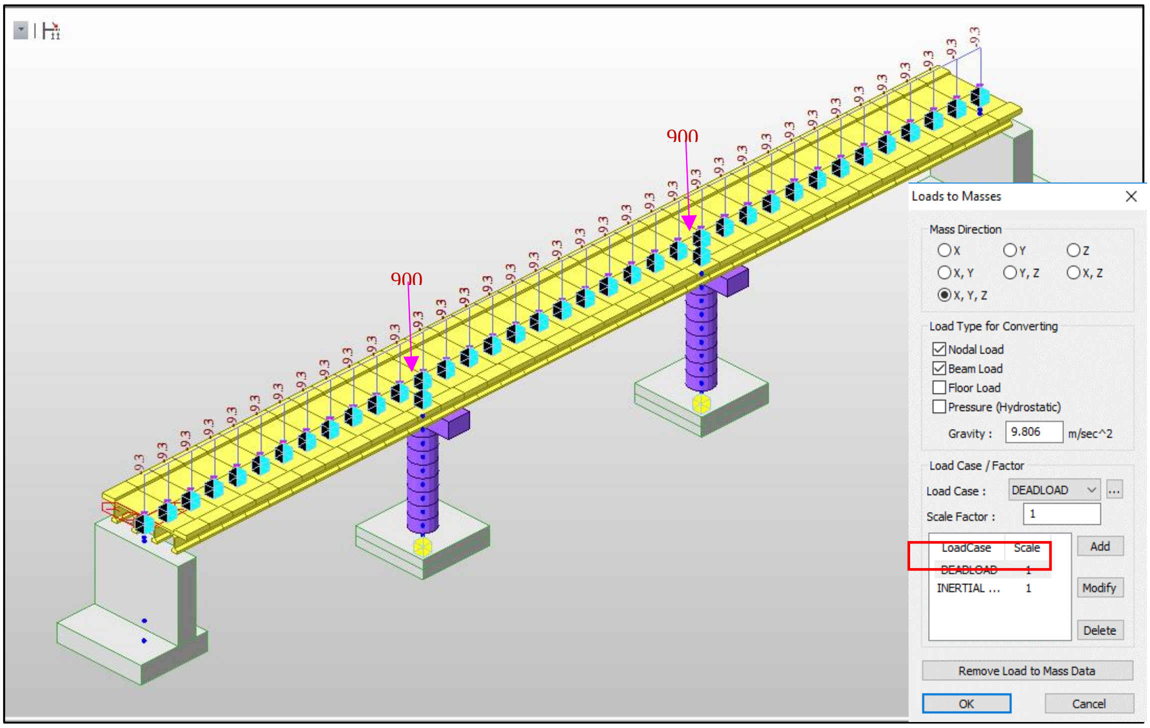


Figure 5.2-15 50% of Live load effect converted into equivalent masses

5.2.3.3 Earthquake Load

The 1000-year return period Level 2 earthquake has been employed as a design response spectrum in this example. The acceleration response spectrum as shown in **Figure 5.2-15** and the table corresponding to the curve also as shown in figure below.

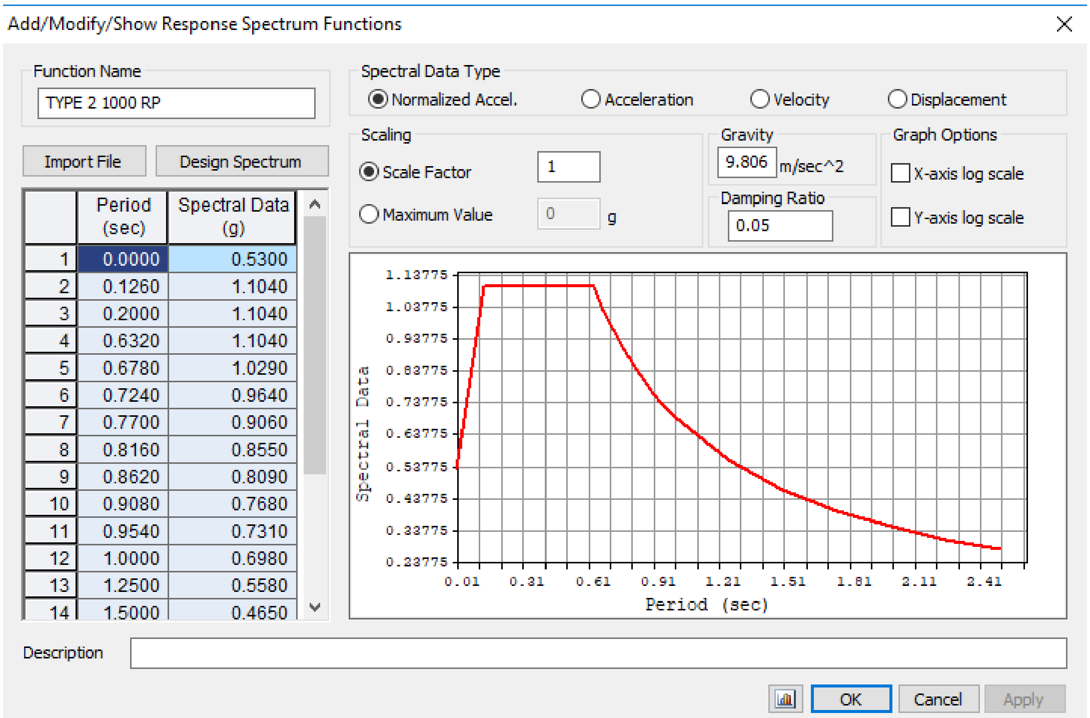


Figure 5.2-16 Design acceleration response spectrum

5.3 Modal Analysis

5.3.1 Multimode Spectral Analysis

BSDS 4.3.3

(1) The multimode spectral analysis method shall be used for bridges in which coupling occurs in more than one of the three coordinate directions within each mode of vibration. As a minimum, linear dynamic analysis using a three-dimensional model shall be used to represent the structure.

(2) The number of modes included in the analysis should be at least three times the number of spans in the model. The design seismic response spectrum as specified in Article 3.6.1 of these Specifications shall be used for each mode.

(3) The member forces and displacements may be estimated by combining the respective response quantities (moment, force, displacement, or relative displacement) from the individual modes by the **Complete Quadratic Combination (*CQC*)** method.

The multi-mode spectral analysis method is more sophisticated than single-mode spectral analysis and is very effective in analyzing the response of more complex linear elastic structures to an earthquake excitation.

Multi-mode spectral analysis assumes that member forces, moments, and displacements because of seismic load can be estimated by combining the responses of individual modes using the methods such as Complete Quadratic Combination (CQC) method and the Square Root of the Sum of the Squares (SRSS) method. The CQC method is adequate **for most bridge systems** (Wilson et al., 1981; Wilson, 2009; Menun and Kiureghian, 1998) and the SRSS method is best suited for combining responses of well-separated modes.
Application example of modal analysis and the combination rule using CQC as shown in **Figure 5.3-1**.

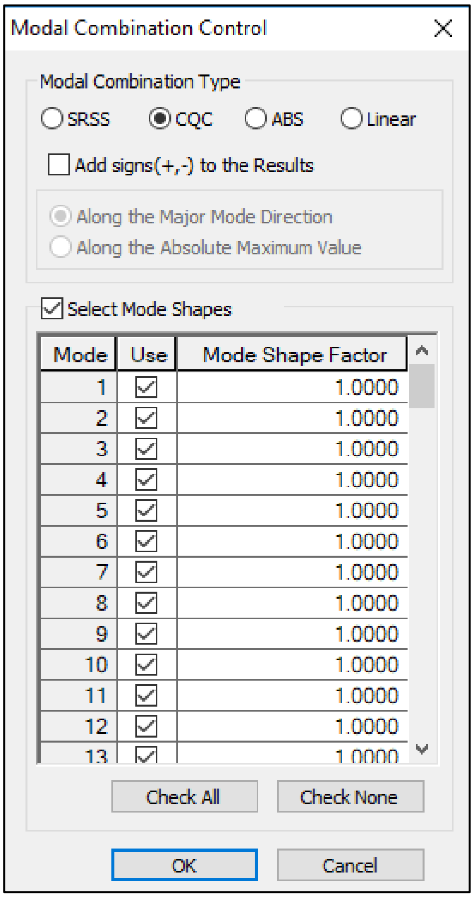


Figure 5.3-1 Modal Combination using Midas

5.3.1.1 Eigenvalue Analysis

Response values are calculated based on vibration property of the bridge and inputted seismic motion. Before calculating specific response values such as sectional forces and displacements, understanding the vibration characteristic of the target bridge must be extremely important phase because not only understanding dynamic behaviors but also dominant basic vibration mode can be understood to be utilized for static analysis. The most familiar methodology to clear this problem is eigenvalue analysis with multimode elastic method. Multi-Degree-of-Freedom and Multi-Mass-Vibration system such as bridge structure has same number of natural periods and vibration modes to number of mass. Such like that, eigenvalue analysis can be defined as calculating characteristic values of multi-mass vibration system; the following values are commonly utilized.

(1) Natural Frequency and Natural Period

Natural frequency is defined as the vibration frequency (Hz), and Natural Period is the time (seconds) for a cycle, which indicates the period of well-vibrated vibration system. Eigenvalue analysis is to obtain characteristic values of vibration system, the principal is conformed to the mentioned equation below regarding dynamic analysis in which right side member is zero. Then, damping term should be separated from eigenvalue analysis but should be considered to determine mode damping based on various damping property when response spectrum analysis or time history response analysis. Therefore:

- No effects from inputted seismic motion and its direction
- Effects from mass and structural system
- Non-linear performance of structural members not considered
- Damping coefficient not considered, but later can be considered for response spectrum analysis or time history response analysis

In eigenvalue analysis, the natural frequency ω is obtained without consideration of damping factor, using the following equation. Where, the natural period T_n is the inverse number of the natural frequency.

$$\left|[K]-\omega^2[M]\right|=0$$

$[K]$: *Stiffness matrix,* $[M]$: *Mass matrix*

(2) Participation factor and Effective mass

The participation factor at "j" th mode can be obtained by following the equation. The standard coordination "qj" that is the responses of the mode with larger participation factor become larger and commonly the participation factor has both positive and negative values.

$$\beta_j=\{\Phi_j\}^T[M]\{L\}/\overline{M}_j$$

β_j: *Model participation factor,* $\{\Phi_j\}$: *Mode matrix,* $[M]$: *Mass matrix,*

$\{L\}$: *Acceleration distribution vector:* $\{\ddot{Z}\}=\ddot{z}\{L\}$: $\{\ddot{Z}\}$: *Acceleration vector,* $\ddot{z}$:*Ground motion acceleration,* $\overline{M}_j$: *Equivalent mass*

From the participation factor, the effective mass at "j" the mode can be obtained by the following equation and have always positive value and the summation of effective mass of all of the vibration modes must conform to total mass of the structure. This effective mass indicates "vibrating mass in all of mass". In most seismic design codes, it is stipulated that the **sum of the effective modal masses included in an analysis** should be **greater than 90%** of the total mass. This will ensure that the **critical modes** that **affect the results** are included in the design. In this example, the mass participation is tabulated in cumulative order as shown in **Figure 5.3-1**.

(3) Natural Vibration Mode (Mode Vector)

Natural vibration mode, what is called as mode vector, indicated the vibration shape at any mode based on dynamic equation of n-freedom system, which is very important factor because it is required in all the terms consisting of dynamic equation such as mass, damping and stiffness matrix. Generally, standard vibration mode vector {Φj} can be obtained by modal coordination which is transformed from displacement vector $\{u\}$ under ratio constant condition; then, coupling parameters are disappeared; n-freedom problem can be treated as "n" of mono-freedom systems. Such the analytical method is called and model analysis method

Longitudinal   Transverse dir.

MODAL PARTICIPATION MASSES PRINTOUT												
Mode No	TRAN-X		TRAN-Y		TRAN-Z		ROTN-X		ROTN-Y		ROTN-Z	
	MASS(%)	SUM(%)	MASS(%)	SUM(%)	MASS(%)	SUM(%)	MASS(%)	SUM(%)	MASS(%)	SUM(%)	MASS(%)	SUM(%)
1	94.77	94.77	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.10	0.00	0.00
2	0.00	94.77	82.16	82.16	0.00	0.00	3.51	3.51	0.00	0.10	0.01	0.01
3	0.00	94.77	0.00	82.16	3.59	3.59	0.00	3.51	0.23	0.32	0.00	0.01
4	0.01	94.78	0.00	82.16	0.33	3.92	0.00	3.51	64.36	64.68	0.00	0.01
5	0.00	94.78	0.01	82.17	0.00	3.92	0.03	3.54	0.00	64.68	66.49	66.50
6	0.00	94.78	0.00	82.17	66.64	70.56	0.00	3.54	0.60	65.27	0.00	66.50
7	0.00	94.78	0.00	82.17	1.12	71.68	0.00	3.54	9.05	74.32	0.00	66.50
8	0.00	94.78	7.35	89.53	0.00	71.68	2.43	5.97	0.00	74.32	0.01	66.51
9	0.00	94.78	0.00	89.53	9.98	81.66	0.00	5.97	0.24	74.56	0.00	66.51
10	0.05	94.84	0.00	89.53	0.46	82.12	0.00	5.97	3.60	78.16	0.00	66.51
11	0.00	94.84	0.05	89.58	0.00	82.12	1.67	7.63	0.00	78.16	5.10	71.61
12	0.00	94.84	3.23	92.81	0.00	82.12	62.39	70.02	0.00	78.16	0.01	71.62
13	0.02	94.86	0.00	92.81	0.51	82.63	0.00	70.02	0.00	78.16	0.00	71.62
14	0.18	95.04	0.00	92.81	12.72	95.35	0.00	70.02	0.00	78.16	0.00	71.62
15	4.07	99.11	0.00	92.81	0.64	96.00	0.00	70.02	2.23	80.39	0.00	71.62
16	0.00	99.11	0.02	92.83	0.00	96.00	0.15	70.16	0.00	80.39	16.64	88.26
17	0.00	99.11	0.00	92.83	0.02	96.02	0.00	70.16	0.00	80.39	0.00	88.26
18	0.05	99.16	0.00	92.83	0.00	96.02	0.00	70.16	7.58	87.97	0.00	88.26
19	0.00	99.16	4.83	97.66	0.00	96.02	3.18	73.34	0.00	87.97	0.03	88.29
20	0.00	99.16	0.00	97.66	0.80	96.82	0.00	73.34	0.01	87.98	0.00	88.29

Total modal participation mass to be considered (requirement: over

Figure 5.3-2 Mass Participation

(a) Eigenvalue analysis option

The subspace iteration method is an effective method widely used in engineering practice for the solution of eigenvalues and eigenvectors of finite element equations.

- This technique is suited for the calculations of few eigenvalues and eigenvectors of large finite element system.
- Starting iteration vectors will be stablished first.

$$q = \min\{2p, p+8\} \leq No.\ \text{of Mass Degrees of freedom}$$

$$d = q * \left(\frac{q+1}{2}\right)$$

Where:

q = number of iteration vectors

p = number of eigenvalues and vectors to be calculated
d = subspace dimension

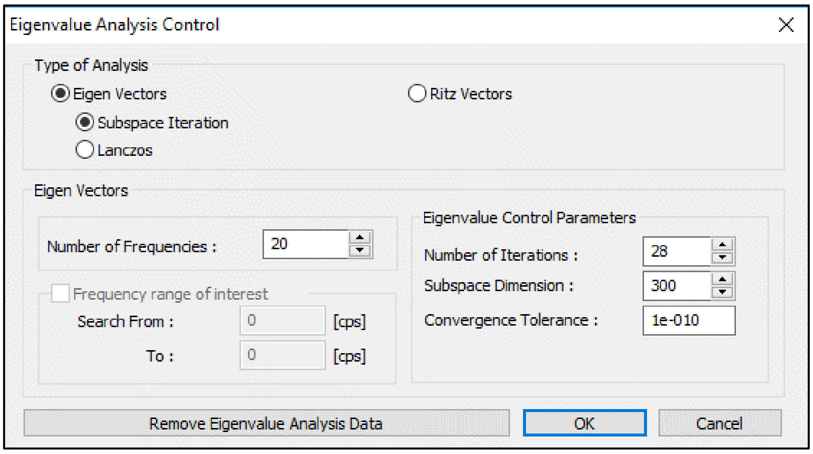
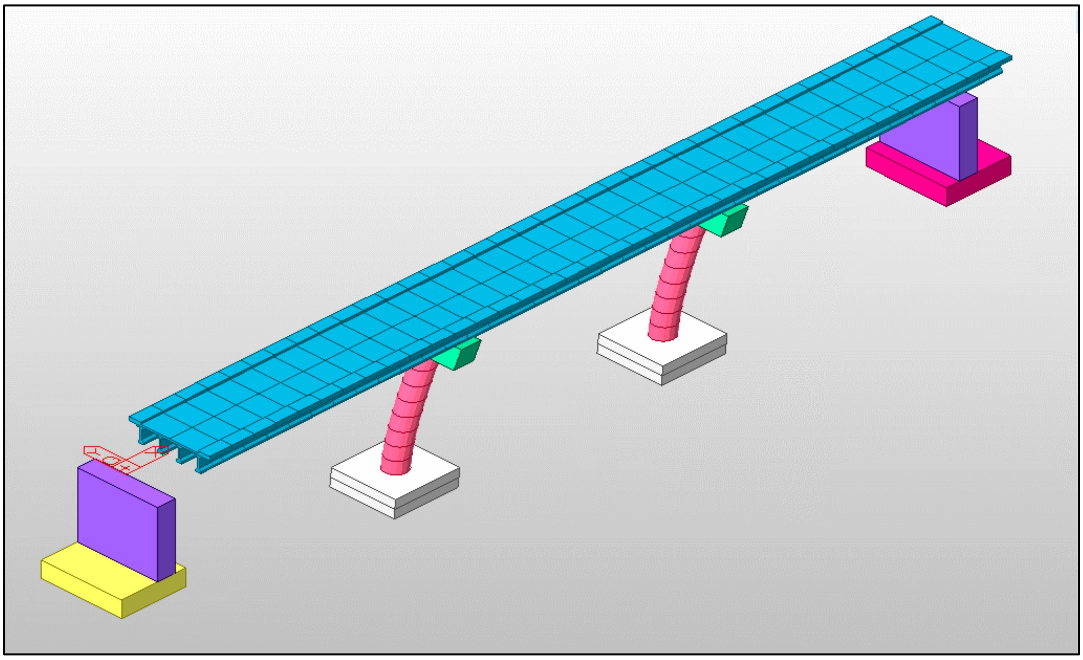


Figure 5.3-3 Eigenvalue Analysis option

Fundamental modes of vibration of the example bridge in two orthogonal direction as shown in **Figure 5.3-2** a and **Figure 5.3-2** b.

a. Longitudinal Direction, Tn = 1.15 secs.



b. Transverse Direction, Tn = 0.69 sec.

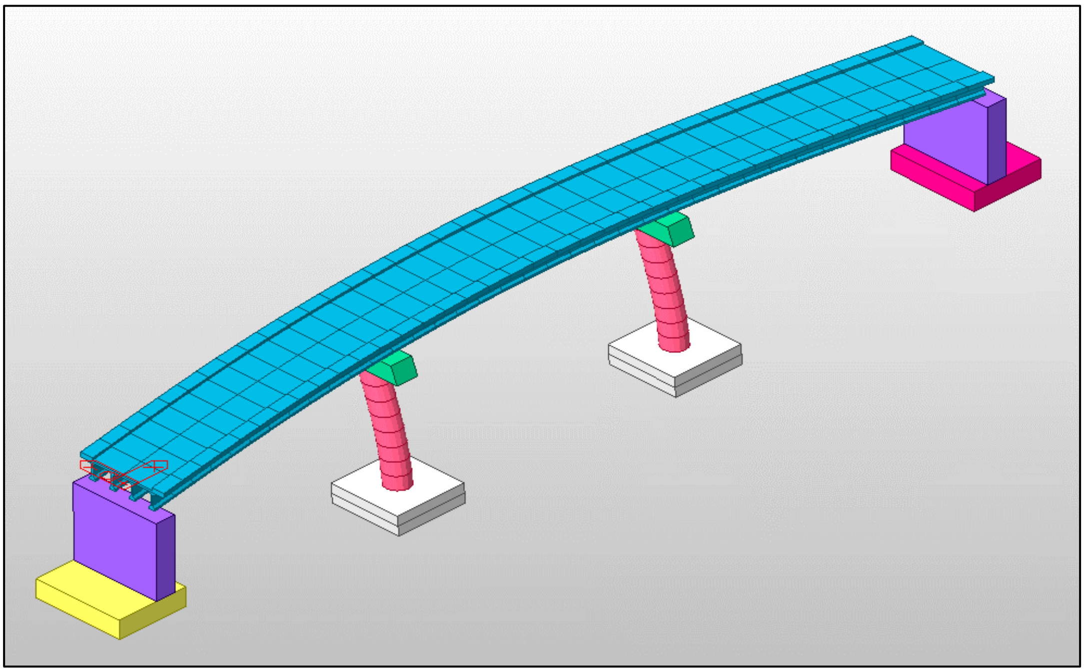


Figure 5.3-4 Natural period of Bridge

5.3.2 Bridge Response

Results from seismic analysis of example bridge Pier bottom as shown below.

Table 5.3-1Pier Bottom Force Response

Elem	Load	Part	Axial (kN)	Shear-y (kN)	Shear-z (kN)	Torsion (kN*m)	Moment-y (kN*m)	Moment-z (kN*m)
412	TYPE 2 PGA - X(R	J[642]	323.67	0.00	7117.23	0.00	90013.56	0.00
413	TYPE 2 PGA - X(R	J[643]	326.66	0.00	7095.62	0.00	89545.46	0.00
412	TYPE 2 PGA - Y(R	J[642]	0.00	4144.03	0.00	3225.38	0.00	51399.06
413	TYPE 2 PGA - Y(R	J[643]	0.00	4181.11	0.00	3536.09	0.00	51863.05

5.4 Elastic Time History (Direct Integration) Analysis

In BSDS Article 4.3.4 mention the used of "Time History Method" in bridge analysis specially for very critical and irregular bridge as specified by DPWH.
Time history analysis is a step-by-step analysis of the dynamic response of a structure to a specified loading that may vary with time. Time history analysis is used to determine the seismic response of a structure under dynamic loading of representative earthquake as discussed in chapter 4 of this guideline. In this chapter, time history analysis using direct integration method of bridge will be discuss.
Mathematical model of example bridges to be used in this analysis are the same as from previous model in Chapter 5.1 except that the seismic load in this model is a transient load. In BSDS following was defined in time history load:

BSDS ***4.3.4.2 Acceleration Time Histories***

"*1.0 Developed time histories shall have characteristics that are representative of the seismic environment of the site and the local site conditions.*

2.0 Response-spectrum-compatible time histories shall be used as developed from representative recorded motions. Analytical techniques used for spectrum matching shall be demonstrated to be capable of achieving seismologically realistic time series that are similar to the time series of the initial time histories selected for spectrum matching.

3.0 Where recorded time histories are used, they shall be scaled to the approximate level of the design response spectrum in the period range of significance. Each time history shall be modified to be response-spectrum compatible using the time-domain procedure.

*4.0 **At least three response-spectrum-compatible time histories** shall be used for **each component** of motion in representing the Level 2 EGM design earthquake (ground motions having seven percent (7%) probability of exceedance in 75 years). **All three orthogonal components (x, y, and z) of design motion shall be input simultaneously** when conducting a nonlinear time-history analysis. The design actions shall be taken as the **maximum response** calculated for the three ground motions in **each principal direction**.*

*5.0 **If a minimum of seven-time histories are used for each component of motion, the design actions may be taken as the mean response calculated for each principal direction**.*

*6.0 **For near-field sites (D < 10 km),** the recorded horizontal components of motion that are selected should represent a near-field condition and should be transformed into principal components before making them response-spectrum-compatible. The **major principal component should then be used to represent motion in the fault-normal direction** and the **minor principal component should be used to represent motion in the fault-parallel direction**."*

In this example, seven (7) pairs of time history site specific ground motion has been used as shown in **Figure 5.4-1**.

There are also several methods used in the time history response analysis including the modal analysis, direct integration and the complex response method but the appropriate method shall be chosen based on the purpose of analysis during verification of seismic performance as discussed in Chapter 4 of this guideline. In this example, Direct integration by means of Newmark Integration (Linear acceleration method) has been employed for the analysis. The response may be calculated using spectrally matched input motions applied in two directions simultaneously. In general, the methods of time history analysis are summarized as shown in **Figure 5.4-1** as explained in chapter 4 of this guideline.

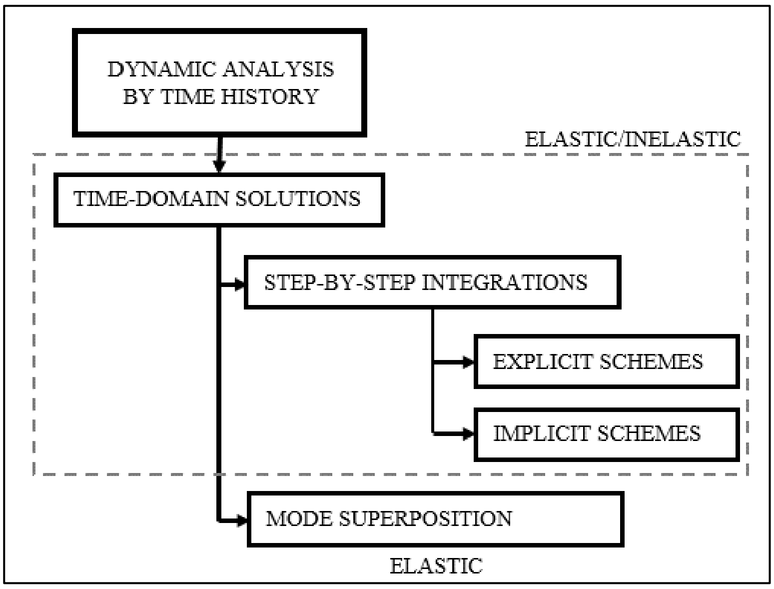
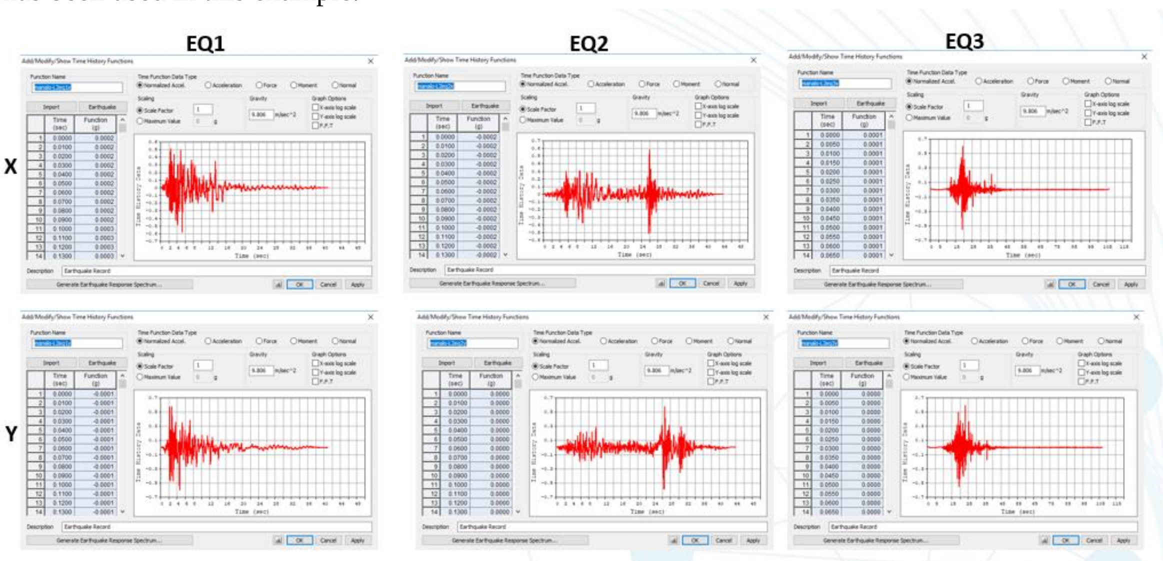


Figure 5.4-1Time History Analysis Methods

5.4.1 Time History ground motions

The generated site-specific ground motions based on actual ground characteristics and site conditions has been used in this example.



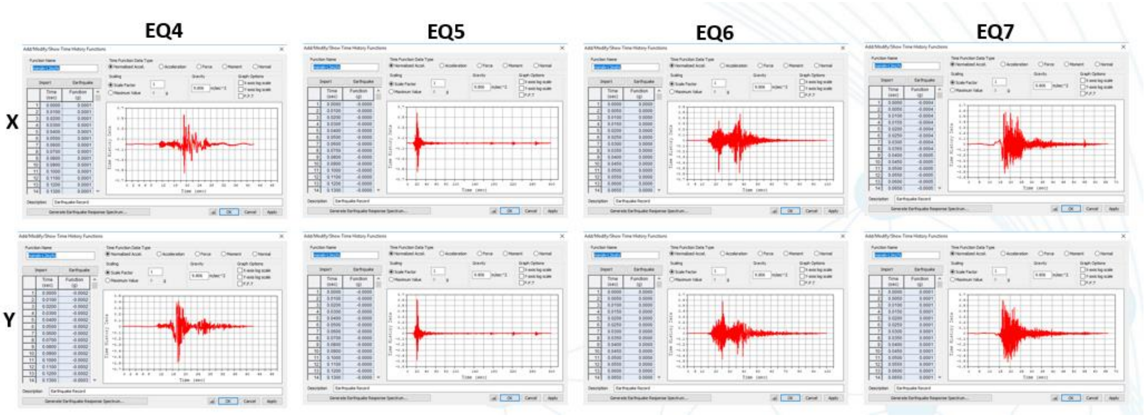


Figure 5.4-2 Seven Pairs of spectrally Matched acceleration time history

Using the mathematical model of sample bridge in Error! Reference source not found., Followings are the example of time history analysis performed in Midas Civil software

(1) Definition of dynamic load cases

As shown in **Figure 5.4-2**, first load case is defined as EQ1. Load type is transient since it is a time domain loadings and analysis type are linear using direct integration by Newmark Integration method.

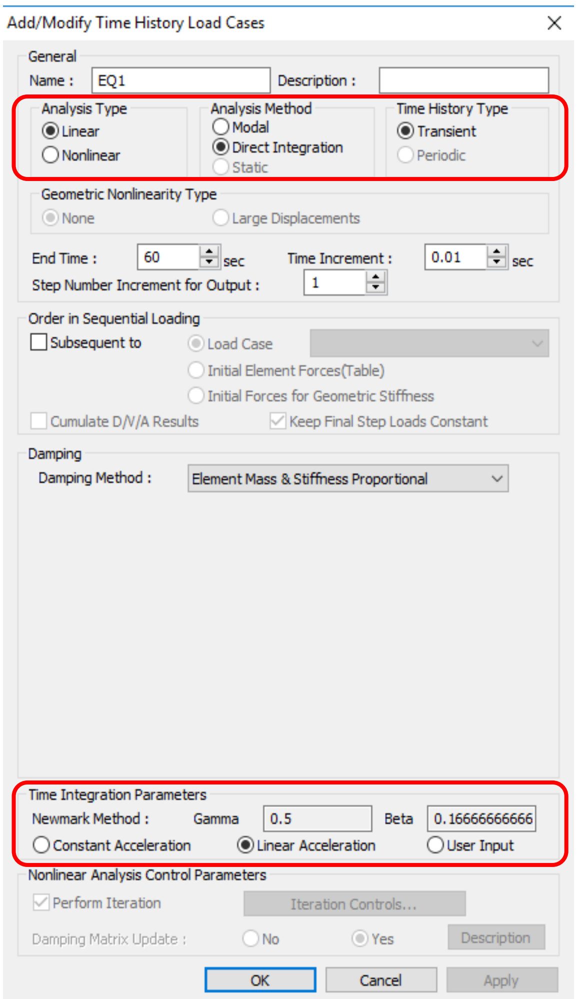
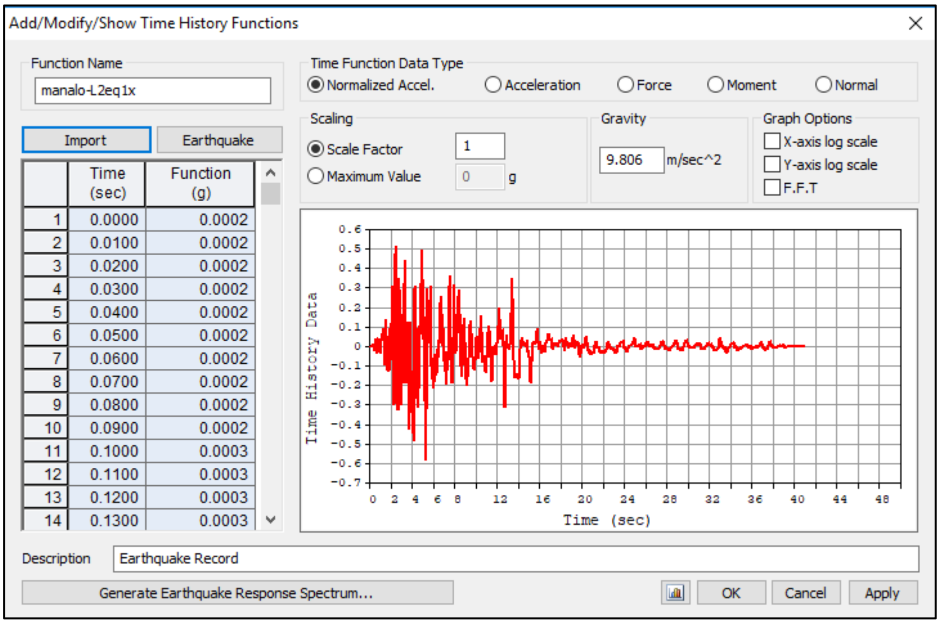


Figure 5.4-3 Load cases definition

(2) Definition of time forcing function (input acceleration time history)

Seven pairs of spectrally matched acceleration time history in **Figure 5.4-1** has been inputted in the analysis model as shown on **Figure 5.4-3**.



	Time (sec)	Function (g)
1	0.0000	0.0002
2	0.0100	0.0002
3	0.0200	0.0002
4	0.0300	0.0002
5	0.0400	0.0002
6	0.0500	0.0002
7	0.0600	0.0002
8	0.0700	0.0002
9	0.0800	0.0002
10	0.0900	0.0002
11	0.1000	0.0003
12	0.1100	0.0003
13	0.1200	0.0003
14	0.1300	0.0003

Figure 5.4-4 Example input ground motion

(3) Input Ground Acceleration (Fault parallel and Fault Normal Direction)

Selected earthquake ground motion has been assigned according to its principal orthogonal direction simultaneously with respect to the direction of source and the principal axis of the bridge. In this example, only the two components at horizontal direction (fault normal (EQx) and fault parallel (EQy)) earthquake has been applied. The angle of excitation according to bridge position with respect to the nearest source (Line Fault) in this case is approximately 45° as shown in **Figure 5.4-5.**

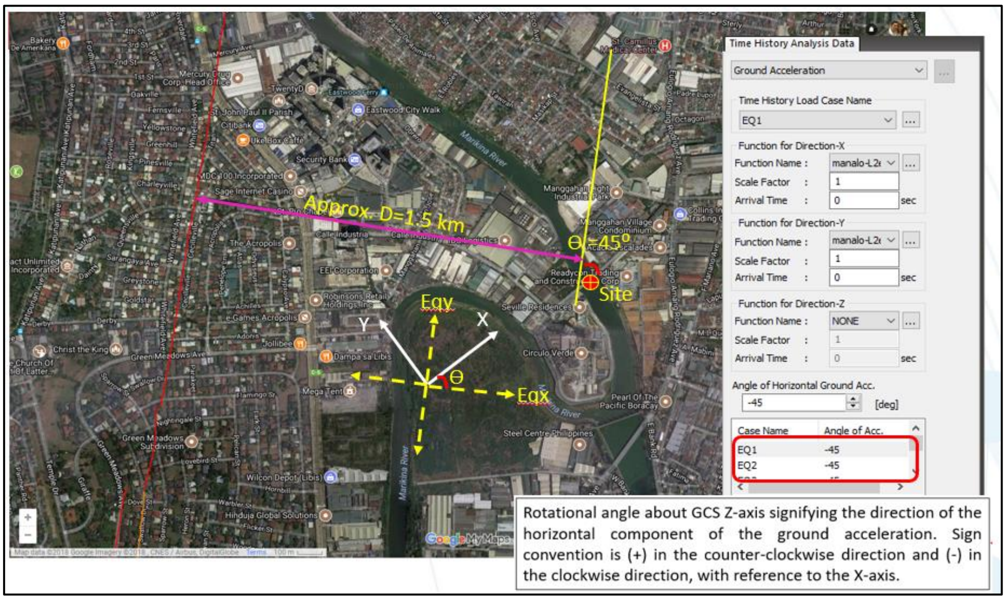


Figure 5.4-5 Excitation angle of earthquake based on the nearest source

(4) Damping

The damping of a structure is related to the amount of energy dissipated during its motion. It could be assumed that a portion of the energy lost because of the deformations and thus damping could be idealized as proportion to the stiffness of the structure. Another mechanism of energy dissipation could be attributed to the mass of the structure and thus damping is idealized as proportion to the mass of the structure. In time history analysis, damping is very important to consider during the analysis. In BSDS Commentary C.4.5.4 mention that "*Damping may be neglected in the calculation of natural frequencies and associated nodal displacements.* ***The effects of damping should be considered where a transient response is sought.***

Suitable damping values may be obtained from field measurement of induced free vibration or by forced vibration tests. In lieu of measurements, the following values recommended in AASHTO may be used for the equivalent viscous damping ratio:

- ***Concrete construction: two percent (2%)***
- *Welded and bolted steel construction: one percent (1%)*
- *Timber: five percent (5%)*

In this example, The Rayleigh damping in a direct integration method has been use as explained in chapter 4 of this guideline. The values of a_0 and a_1 determined by only two major modes, which are incorporated in $C = a_0M + a_1K$ to compute a damping matrix. With the equation of motion in a matrix format, direct integration is executed for each time step. Application of this type into the analysis model as shown in **Figure 5.4-5**

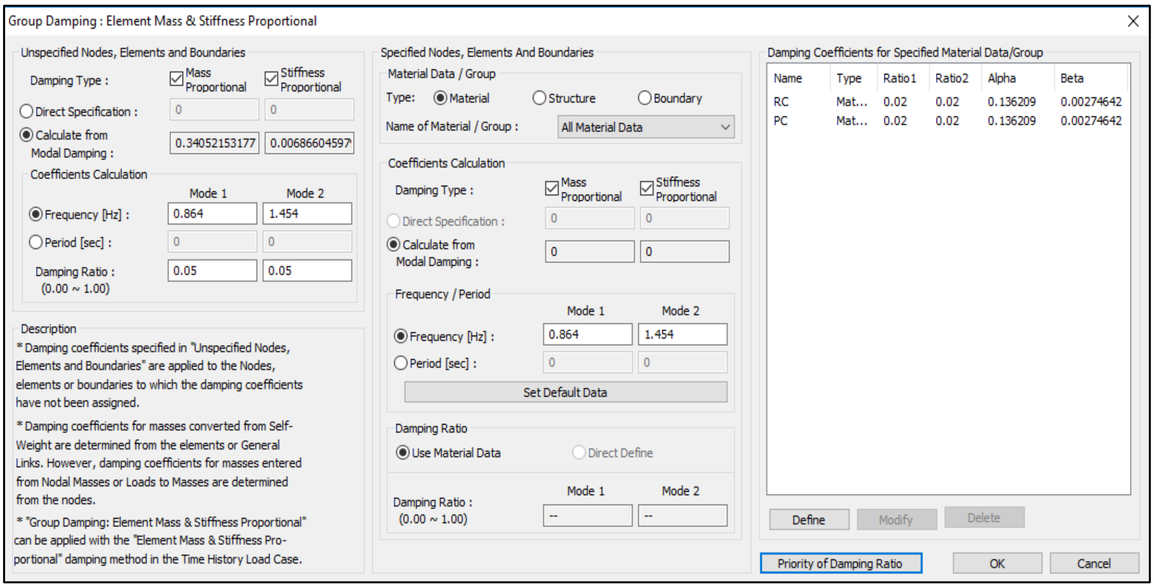


Figure 5.4-6 Damping Coefficient Calculation using Midas civil

(5) Time History Analysis Results

a) Time History Response

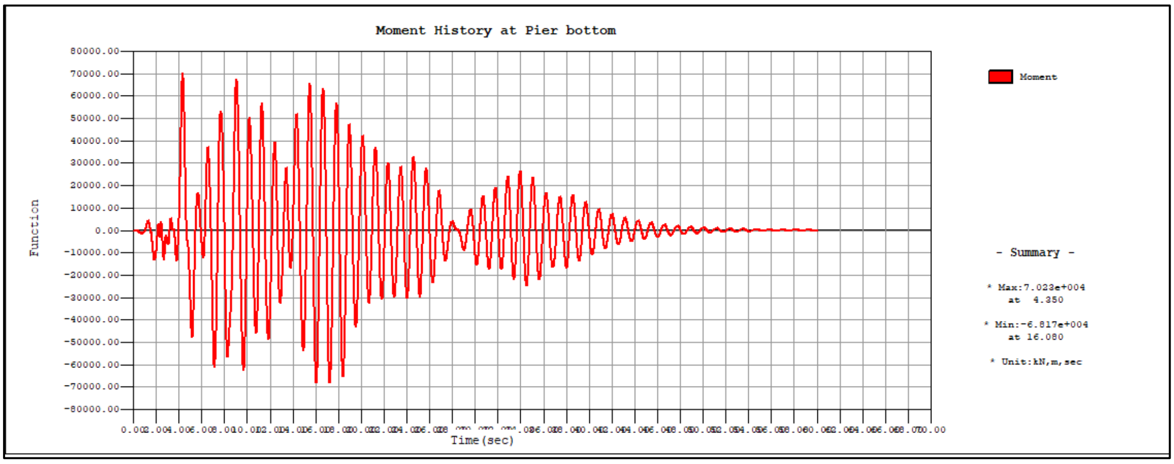


Figure 5.4-7 a) Moment (My) Response of Pier bottom due to Eq1

b) Force Response

Design actions were taken as the mean response due to seven pairs of ground motions as explained in BSDS 4.3.4.2.

Table 5.4-1 Force Response

SAMPLE BRIDGE								
PIER NO.:		**1**	**FORCES**					
Elem	Load	Part	Axial (kN)	Shear-y (kN)	Shear-z (kN)	Torsion (kN*m)	Moment-y (kN*m)	Moment-z (kN*m)
412	EQ1(ma	J[642]	288.93	5182.82	5476.84	3733.92	70233.34	58972.4
412	EQ2(ma	J[642]	450.08	4943.08	7451.77	3783.43	108068.83	60436.09
412	EQ3(ma	J[642]	289.19	6843.97	6084.9	5655.11	90919.15	89701.26
412	EQ4(ma	J[642]	318.65	5061.04	8165.82	4041.32	96915.87	64480.69
412	EQ5(ma	J[642]	297.87	3233.44	5486.41	2869.91	80894.25	45224.67
412	EQ6(ma	J[642]	414.11	4121.29	10333.81	3291.48	124001.9	52294.22
412	EQ7(ma	J[642]	334.52	5978.16	5917.66	4150.15	83399.67	67361.69
	MEAN:		**341.907143**	**5051.97143**	**6988.17286**	**3932.18857**	**93490.43**	**62638.71714**
SAMPLE BRIDGE								
PIER NO.:		**2**	**FORCES**					
Elem	Load	Part	Axial (kN)	Shear-y (kN)	Shear-z (kN)	Torsion (kN*m)	Moment-y (kN*m)	Moment-z (kN*m)
413	EQ1(ma	J[643]	219.93	5240.69	5448.89	4342.05	69862.9	59580.55
413	EQ2(ma	J[643]	325.11	4994.96	7414.77	4188.39	107509.25	60996.72
413	EQ3(ma	J[643]	252.16	6908.45	6052.12	5914.37	90430.25	90409.2
413	EQ4(ma	J[643]	326.22	5125.7	8124.28	4364.46	96403.7	65124.97
413	EQ5(ma	J[643]	335.39	3273.9	5458.12	2813.26	80459.42	45599.37
413	EQ6(ma	J[643]	410.16	4163.44	10281.85	3448.46	123341.47	52718.63
413	EQ7(ma	J[643]	296.77	6000.29	5884.94	4810.29	82954.56	67878.07
	MEAN:		**309.391429**	**5101.06143**	**6952.13857**	**4268.75429**	**92994.50714**	**63186.78714**
MAX FORCES:			**341.907143**	**5101.06143**	**6988.17286**	**4268.75429**	**93490.43**	**63186.78714**

c) Displacement Response of Pier top

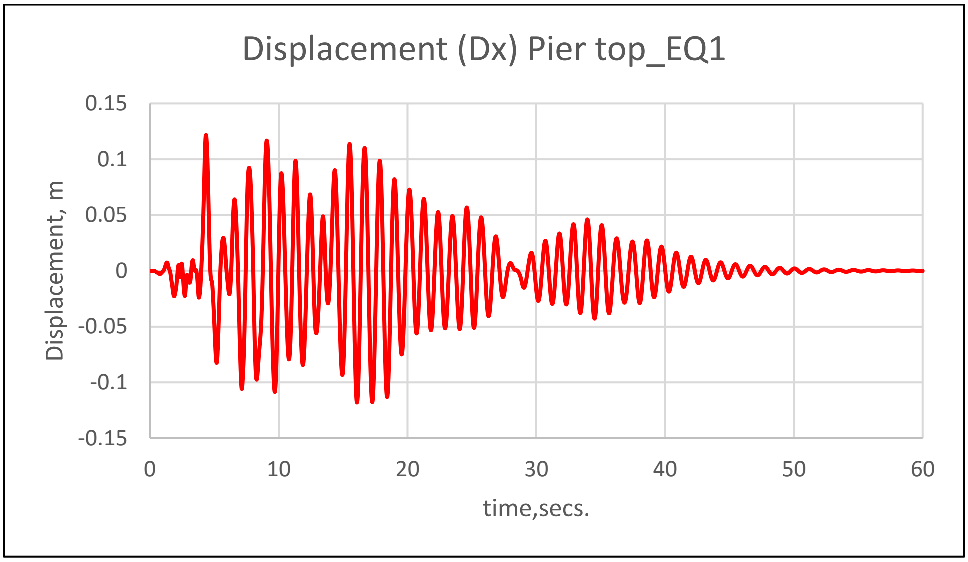


Figure 5.4-8 b) Pier top Displacement response due to EQ1

The design displacement at any direction were taken as the mean response of bridge due to seven pairs of earthquakes as shown in **Table 5.4-2**.

Table 5.4-2 Design Displacement At Pier Top

Node	Load	DX (m)	DY (m)	DZ (m)	RX (rad)	RY (rad)	RZ (rad)
224	EQ1(max)	0.121684	0.102286	0.00011	0.012822	0.014288	0.001674
224	EQ2(max)	0.187435	0.104567	0.000171	0.012379	0.022053	0.001696
224	EQ3(max)	0.15734	0.1552	0.00011	0.017458	0.018419	0.002535
224	EQ4(max)	0.167794	0.111639	0.000121	0.012887	0.019657	0.001812
224	EQ5(max)	0.140006	0.078113	0.000113	0.00828	0.016411	0.001286
224	EQ6(max)	0.214616	0.090488	0.000157	0.010142	0.025144	0.001475
224	EQ7(max)	0.14447	0.116174	0.000127	0.014156	0.017002	0.001861
MEAN:		**0.161906**	**0.108352**	**0.00013**	**0.012589**	**0.018996**	**0.001763**

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CHAPTER 6: SEISMIC DESIGN OF PIER

Chapter 6 Seismic Design of Pier

6.1 Flowchart

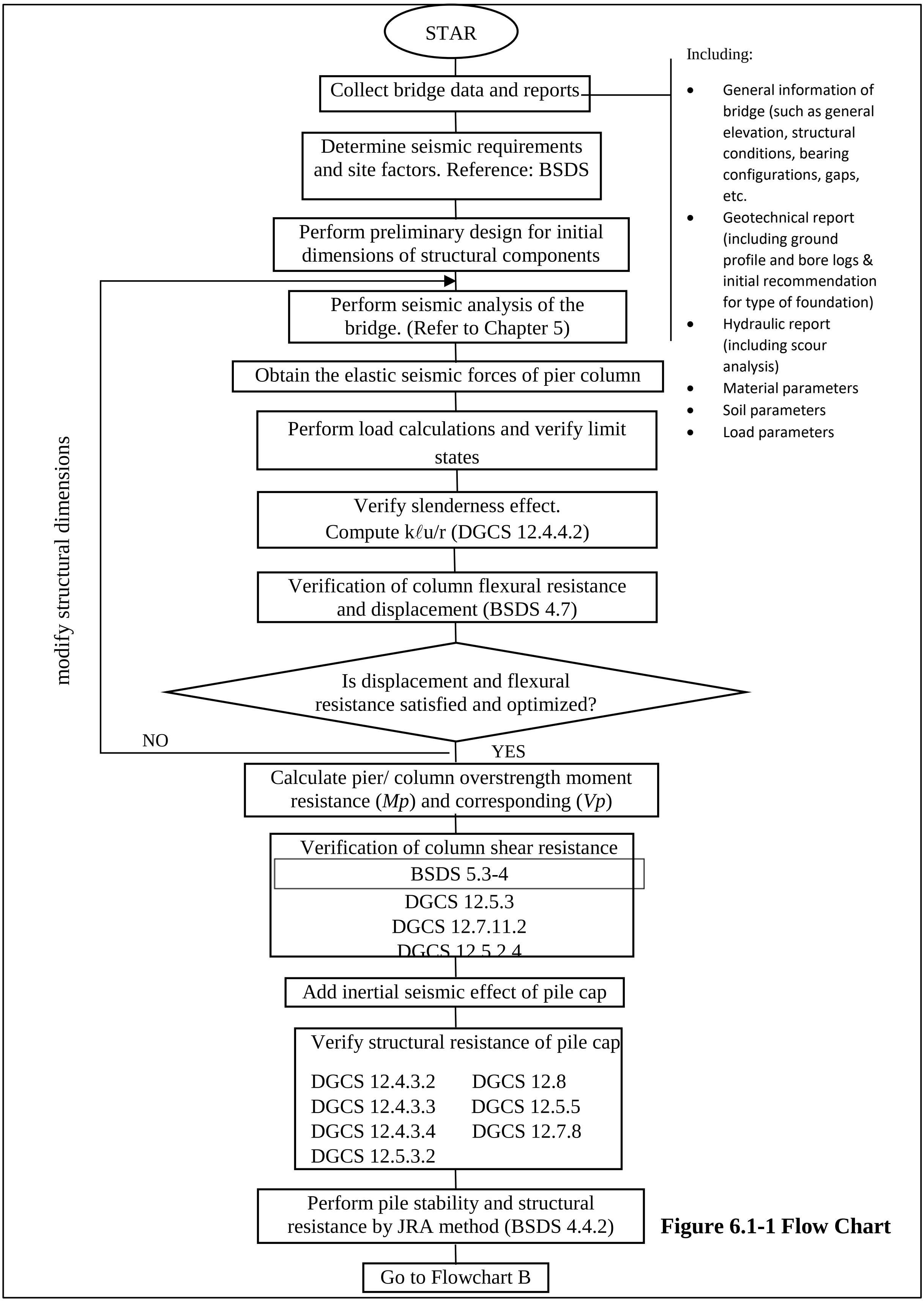


Figure 6.1-1 Flow Chart

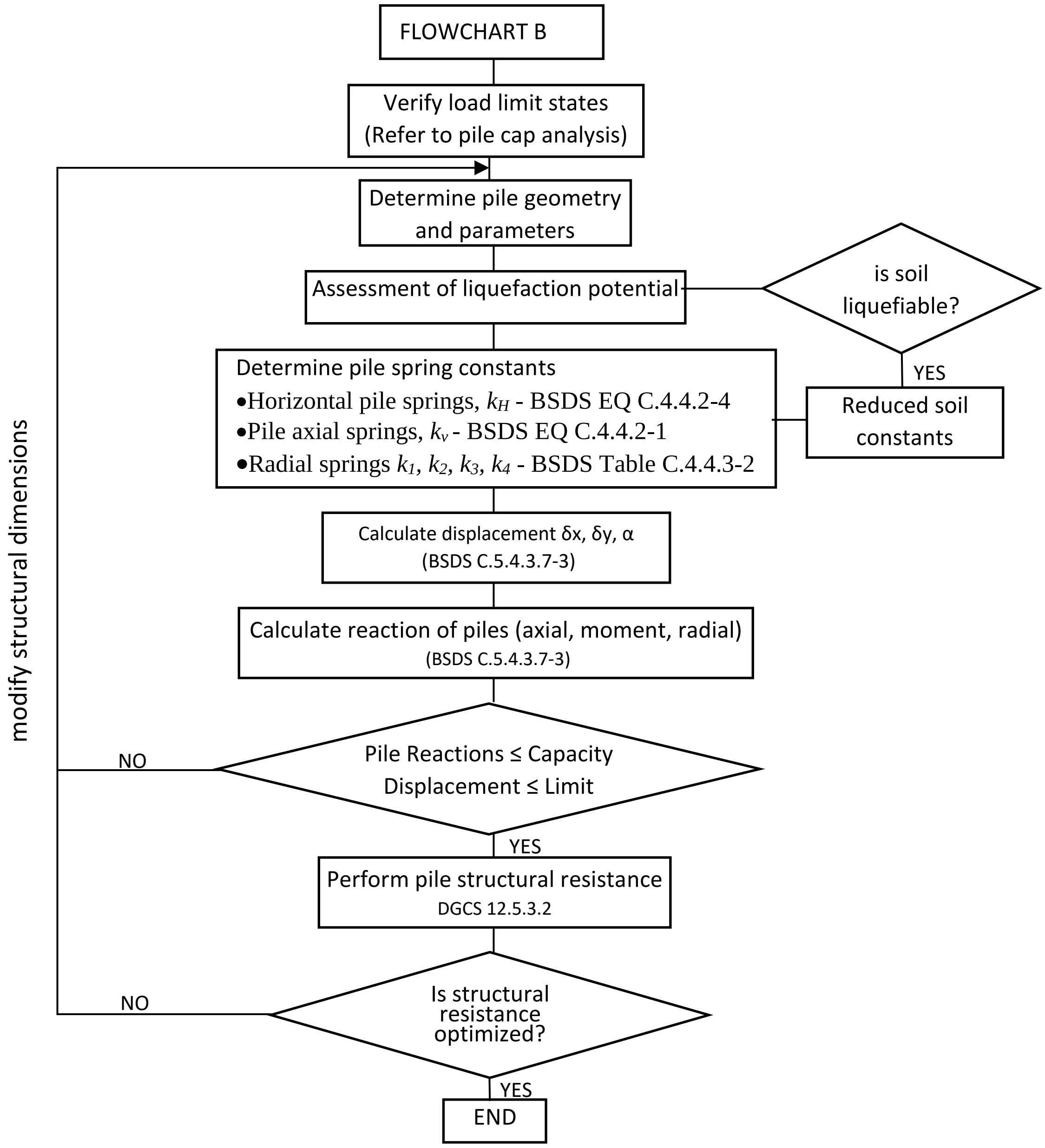


Figure 6.1-2 Flowchart B

6.2 GENERAL DESIGN CONDITIONS & CRITERIA

6.2.1 Bridge General Elevation & Cross section

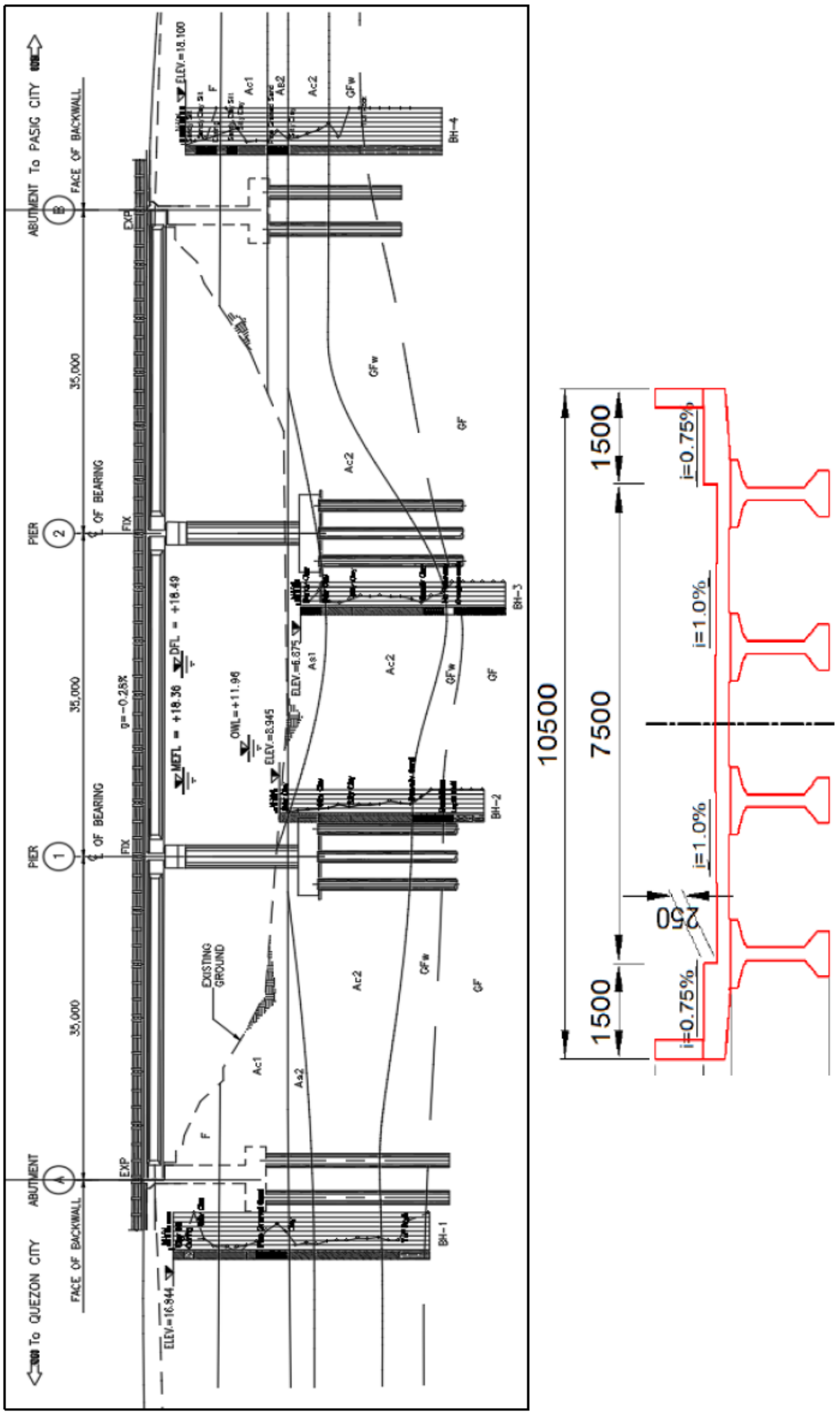


Figure 6.2-1 Bridge General Elevation & Cross Section

6.2.2 Pier Geometry and Location Map.

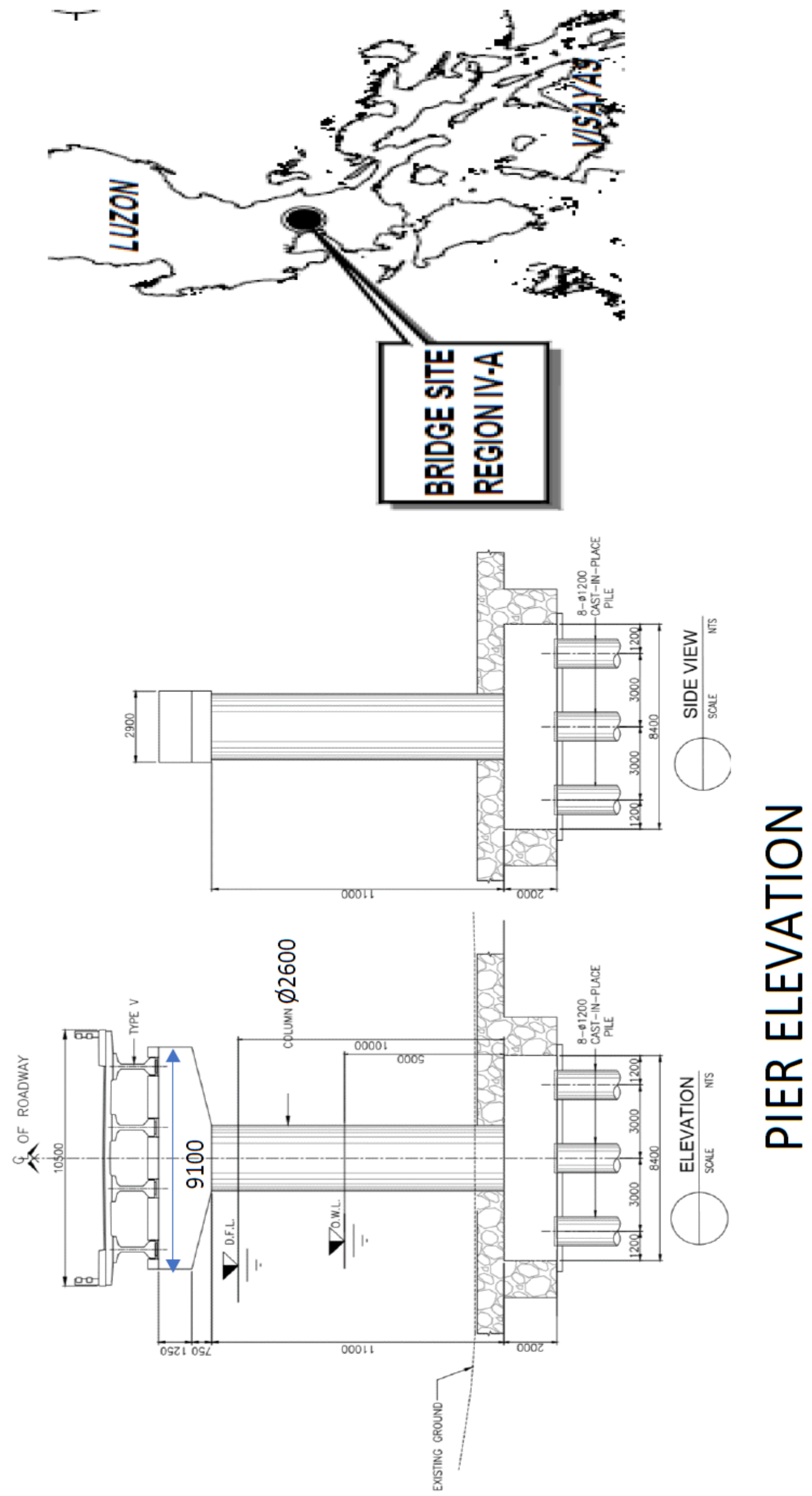


Figure 6.2-2 Pier Geometry and Location Map

6.2.3 Structural Conditions

- Two lane carriageway; total width = 10.50m
- 3 – 35m continuous ASSTHO girders- Type V
- Bearing restraints: M-F-F-M (F=fixed, M=movable)
- Regular bridge (non – skewed bridge)
- Pier type: single column on cast in place concrete pile
- Abutment type: cantilever type on cast in place concrete pile

6.2.4 Seismic Design Requirements and Ground Conditions

Bridge Operational Classification =	OTHERS
Earthquake Ground Motion =	Level 2
Ground Type =	3
Seismic Performance Level =	3
Seismic Performance Zone =	4
Peak Ground Acceleration =	0.6g

6.2.5 Site Factors

Site Factors:	
Fpga =	0.88
Fa =	0.92
Fv =	1.55
As =	0.53

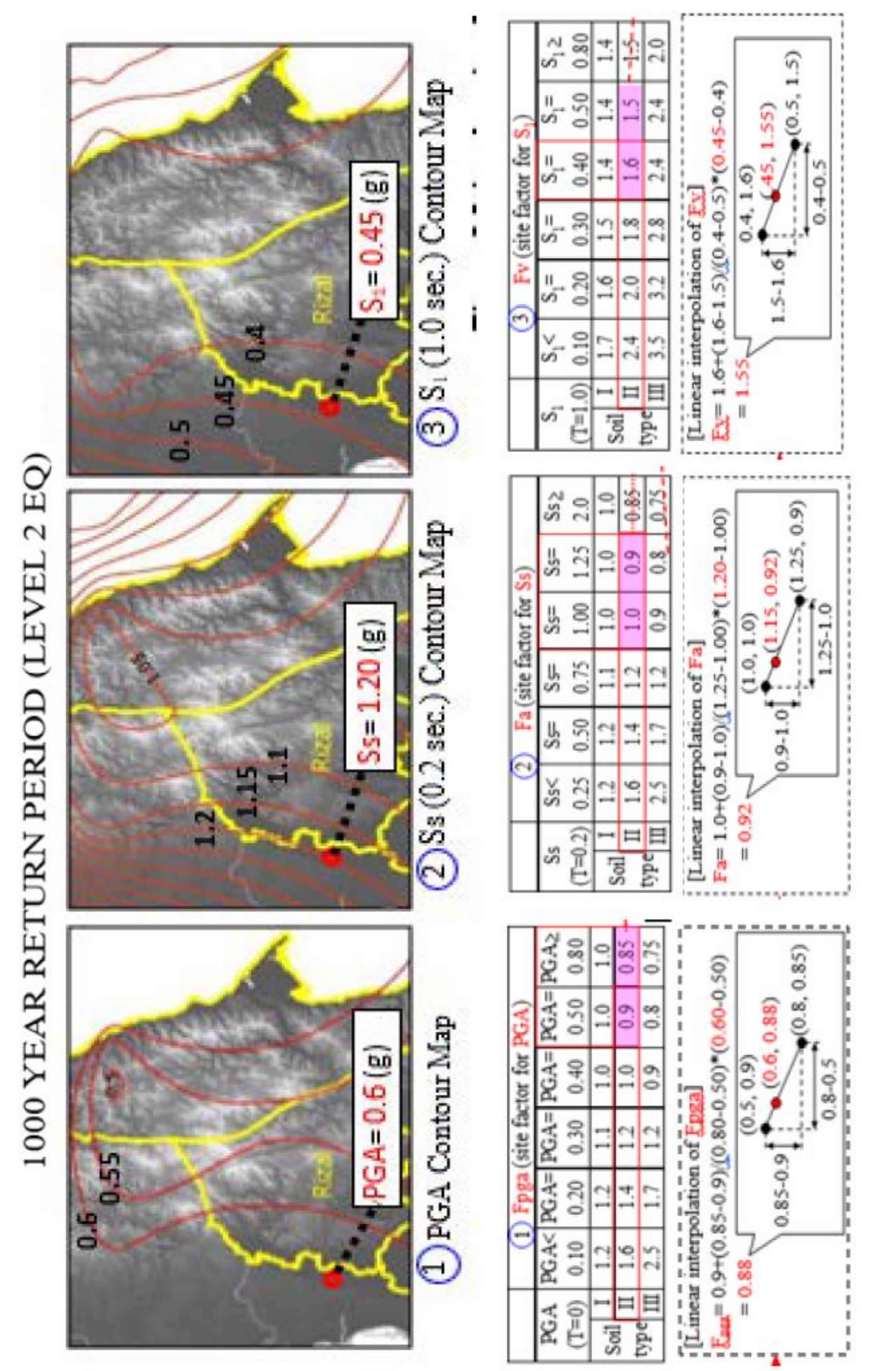


Figure 6.2- 3 Site Factors

6.2.6 Borelogs (not to scale)

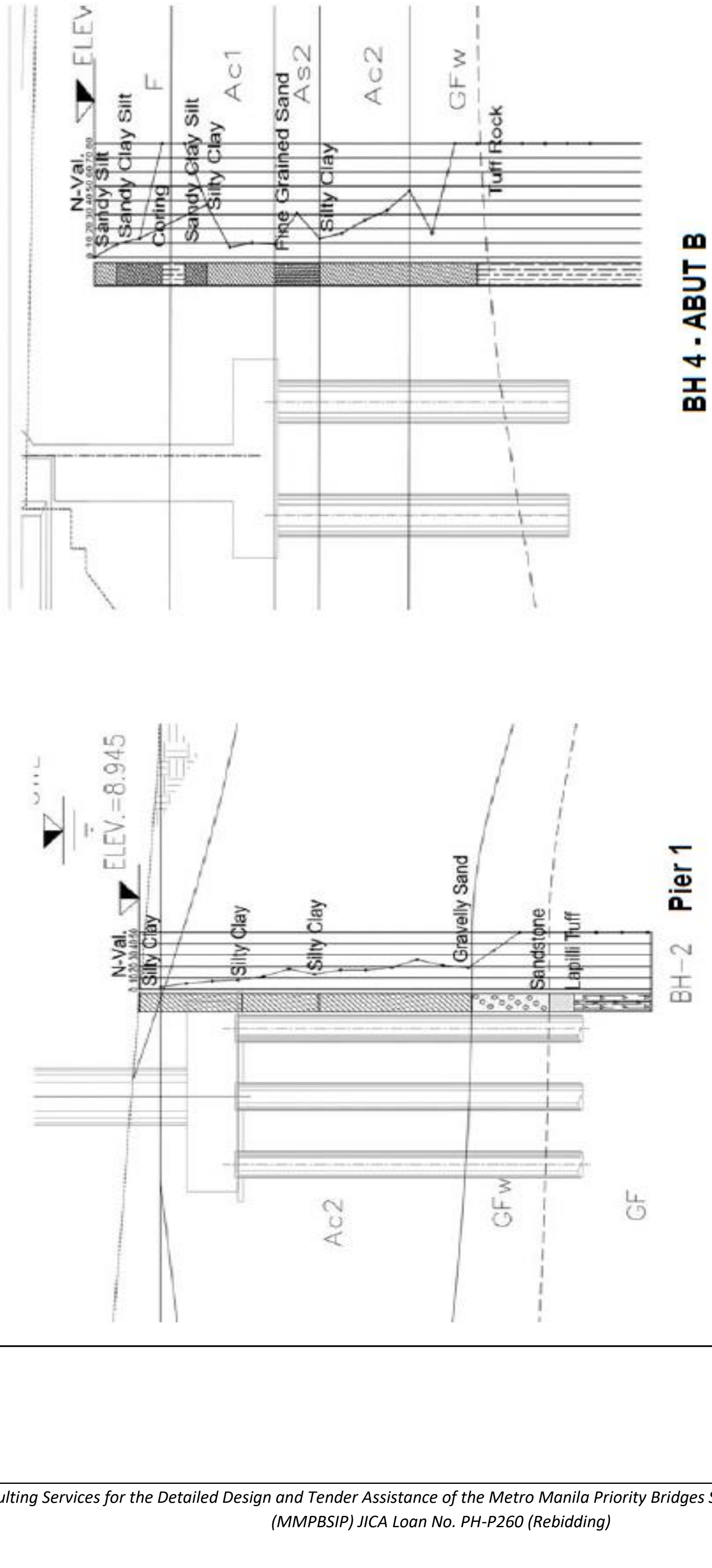


Figure 6.2- 4 Bore Logs of Pier 1 and Abutment

6.2.7 Hydrology and Hydraulics Data

100 years Return Period

Discharge: 3100 m^3/s

Water Level, DFL: 18.50m

Velocity: 4.12 m/s

Freeboard: 0.0m (no consideration)

Drainage Area: 2,360 sqm

Computed scour depth: 6.67m

6.2.8 Design Loads

1. Permanent Loads	
DC =	Dead load pertaining to structural and non-structural components
DW =	Dead load pertaining to future wearing surface
EH =	Horizontal earth pressure
ES =	Earth surcharge
EV =	Vertical pressure from earth fill

2. Transient Loads	
EQ =	Earthquake
LL/IM =	Vehicular Load/Impact Load
LS =	Live load Surcharge
WA =	Water Load
FR =	Friction Load
BF =	Braking Force Load

For Seismic load analysis: Refer to BSDS

Load Combinations and factors: Refer to DGCS 10.0

6.2.9 Material and Soil Property

Conc. Compressive strength @ 28days, f'c	28	MPa
Reinforcing steel (ASTM 615), fy	415	MPa
Unit weight of concrete, δc	24	kN/m^3
Unit weight of concrete, $\Upsilon soil$	19	kN/m^3
Unit weight of rock, Υr	20	kN/m^3

6.3 LOAD CALCULATIONS

6.3.1 Seismic Loads

Obtain the elastic seismic forces of column at its base due to earthquake loadings in longitudinal (Long'l EQ) and transverse (Trans. EQ) directions from the bridge seismic analysis.

Loads = Long'l EQ + 50% LL
Trans. EQ + 50% LL

LC1 = 1.0*Long'l EQ + 0.3*Trans. EQ
LC2 = 1.0*Trans. EQ + 0.3*Long'l EQ

Calculate the orthogonal forces and inelastic hinging effect with Modification Factor, R.

Modified elastic forces(LC moment /R)

ELASTIC FORCES						
Loadings		LONGITUDINAL		TRANSVERSE		AXIAL
	Load Type	SHEAR -z	MOM -y	SHEAR -y	MOM -z	Fx
Loads	Long'l. EQ.	7117.00	90014.00	0.00	0.00	324.00
	Transv. EQ.	0.00	0.00	4181.00	51863.00	0.00
	LC1	7117.00	90014.00	1254.30	15558.90	324.00
	LC2	2135.10	27004.20	4181.00	51863.00	97.20
		V_L	M_L	V_T	M_T	
LC moment/R	LC1	7117.00	30004.67	1254.30	5186.30	324.00
	LC2	2135.10	9001.40	4181.00	17287.67	97.20
R =	3					

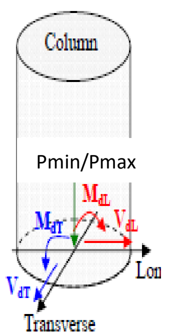
Note: For regular/normal bridges, forces are absolute values that is without regards to its sign because the seismic can act in either direction.

BSDS 3.8: RESPONSE MODIFICATION FACTOR, R

Table 3.8.1-1 Response Modification Factors – Substructures

Substructure	Operational Category		
	OC-I (Critical)	OC-II (Essential)	OC-III (Others)
Wall-type piers – larger dimension	1.5	1.5	2.0
Reinforced concrete pile bents			
• Vertical piles only	1.5	2.0	3.0
• With batter piles	1.5	1.5	2.0
Single columns	1.5	2.0	3.0
Steel or composite steel and concrete pile bents			
• Vertical piles only	1.5	3.5	5.0
• With batter piles	1.5	2.0	3.0
Multiple column bents	1.5	3.5	5.0

Calculate the combine seismic design forces considering the bi-axial response (for circular column).

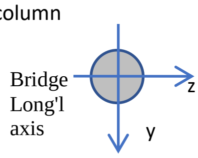


SEISMIC DESIGN FORCES due to combination of biaxial response:				P_{EQ}
EQ_LC1	Mlong=MdL = sqrt $(M_L{}^2 + M_T{}^2)$ =	30449.59	kN-m	324.0
	Vlong=VdL = sqrt $(V_L{}^2 + V_T{}^2)$ =	7226.68	kN	
EQ_LC2	Mtrans=MdT = sqrt $(M_L{}^2 + M_T{}^2)$ =	19490.73	kN-m	97.2
	Vtrans=VdT = sqrt $(V_L{}^2 + V_T{}^2)$ =	4694.62	kN	

Commentary

•Refer to Chapter 5 for the seismic analysis of the bridge and the results of the elastic seismic forces.
• BSDS C4.4.1 (2) : EQ loads + 50% LL (as equivalent inertial mass) for metropolitan bridges.

<<<local axis of column



•For regular/normal and short bridges, it is likely the Long' EQ will produce in z-direction while Trans EQ will produce in y-direction.

•BSDS 3.8. The sample bridge is OC III, Mod. Factor for single column, R=3

•The column is circular which is same capacity in any direction. To simplify the investigation, the load components are combined to obtain vector sum. This procedure not applicable for tied, rectangular or wall shape pier column.

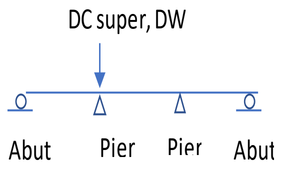
6.3.2 Permanent Loads

Commentary

a. superstructure

DC_super = R_{DL} = Deadload reaction of Pier 1 (top of pier cap) from superstructure permanent loads (including diaphragms, sidewalks, railings, slab, girders, etc. except future wearing surface).

DC_super = 5600 kN



DW = future wearing surface DW = 300 kN

b. substructure

unit weight of concrete Υ_c = 24kN/m^3

DC_cap =((9.1m x 2.9m x 2m)-(6.85m^3)) x Υ_c = DC_cap = 1102.5 kN

Vol of the tapered section = 1/2(0.75m*2.9m*3.15m)*[2] = 6.85m^3

DC_col= (πDcol^2/4) x 11m x Υ_c = DC_col = 1401 kN

where Dcol = 2.6mØ Hcol = 11m

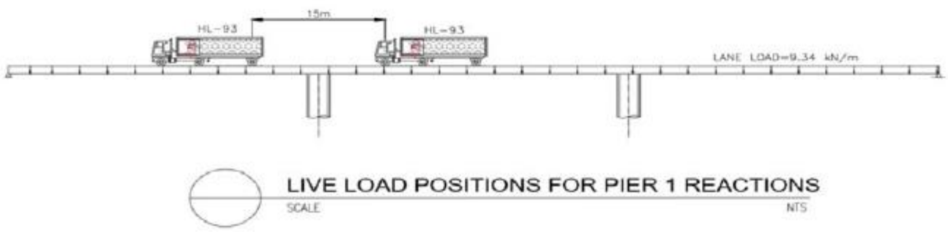
•Note: Other loads were obtained basically by manual calculations.

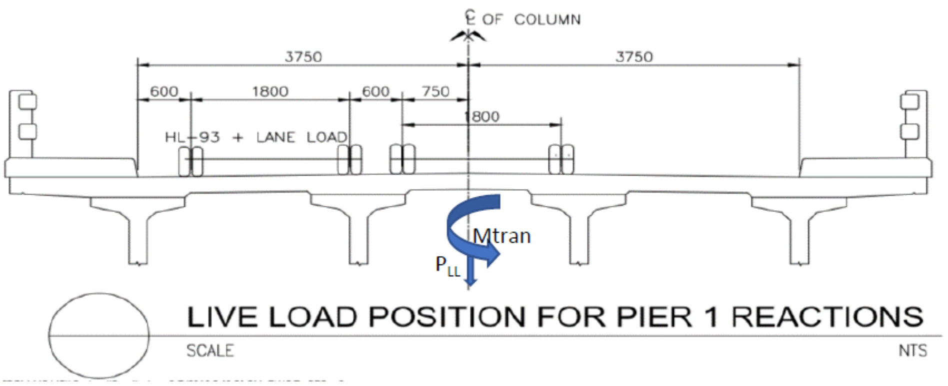
• Refer to Typical cross section of Pier for the geometry and dimensions.

6.3.3 Vehicular Loads

Note: There may be several positions of Trucks in the carriageway, however in this exercise, the combination of (2-trucks + lane)90% is assumed to produce severe effects to the bent.

a. Vehicular Truck + Lane Loads





• In actual design practice, several trials in the positioning of vehicular trucks may be performed to arrive results that will produce severe effects to Pier.

•As shown, it is presumed this case will produce worst effect to pier.

Commentary

From live load analysis, the Reaction of tandem of 2-HL93 =		528.00	kN
	the Reaction Lane Load =	360.00	kN
Reaction of tandem of 2-HL 93 =	90% (Rxn) =	475	kN
Reaction of Lane load =	90% (Rxn) =	324	kN
Reaction per wheel line (2-HL 93)		237.60	kN
Reaction per wheel line (Lane load)		162.00	kN
Impact (not to be applied to Lane Load and footing),IM		n/a	
Factor for multiple presence (m) on 1-lane		1.20	
Factor to multiple presence on (m) 2-lane		1.00	
No. of lanes		2	

<u>For bottom of Column</u>

Mtrans = (Reaction per wheel (2-HL 93 +IM) + Lane)x L.A. x m	2409.16	kN-m
Pmax = Reaction ((2-HL + IM)+ Lane Load) x m x No. of tandems of 2-HL93	1912.03	kN-m

b. <u>Vehicular Braking Force, BF</u>

Note: Braking force shall be taken as the greater of :

25% (HL 93) =	25%(145kN + 145kN + 35kN) =	81.25	kN
25% (Tandem)=	25%(110kN+ 110kN) =	55.00	kN
5% (HL 93 + Lane Load) =	5%(145kN +145kN+35kN+Total Length *9.34)	65.29	kN
5% (Tandem) + Lane Load) =	5%(110 +110+Total Length *9.34)	60.04	kN

Therefore horizontal force BF = greater of the vehicular BF x no. of lanes x m 162.50 kN

<u>For bottom of Column</u>

V_{LONG} = BF /no. of Column per pier	for single column-pier	162.50	kN
M_{LONG} = V_{LONG} (L.A.= height of column+cap thickness)		2112.50	kN-m

<u>For bottom of footing</u>

Mlong = Vlong (L.A.= height of column+cap thickness+ftg thickness)	2437.50	kN-m

•The superstructure is fixed to pier therefore the vehicular load will produce braking force.

6.3.4 Stream flow, WA

a. <u>During Ordinary Water Level (OWL)</u>

Uniform distributed pressure, $P = 5.14 \times 10^{-4} C_d V^2$

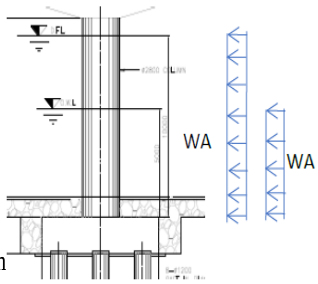
Drag coeff. C_d =	0.7	for circular shape
Velocity of water,V=	3.00	(refer to hydraulic report), m/s

• DGCS 10.12.3

• The OWL is always present hence its effect is combined with Extreme Event 1

H_{OWL} =	5	height of ord. water level, m
P=	0.0032	MPa
Dcol =	2.6	m
Ftg depth =	2	m
WA =	8.42	kN/m
Acol =	5.31	m^2
H_sub =	5.00	m

WA = P x width (Dcol)



For bottom of Column

Vtrans = WA x H_{OWL} =	42.10	kN
Mtrans = Vtrans * H_{OWL} /2 =	105.24	kN-m
BF(buoyant force) = Acol x H_sub x ϒw =	-265.33	kN

b. During Design Flood Level (DFL) 100 year period

Uniform distributed pressure, $P = 5.14 \times 10^{-4}\ C_d\ V^2$

Drag coeff.	0.7	for circular shape
Velocity of water,	4.12	(refer to hydraulic report), m/s
H_{DFL} =	10	height of design flood level, m
P=	0.0061	MPa
Ftg depth =	2	m
WA =	15.88	kN/m
H_sub =	10.00	m

WA = P x width (Dcol)

For bottom of column

Vtrans = WA x H_{DFL} =	158.79	kN
Mtrans = Vtrans * H_{DFL} /2 =	793.96	kN-m
BF(buoyant force) = Acol x H_sub x ϒw =	-530.66	kN

For bottom of footing

Vtrans = WA x H_{DFL} =	158.79	kN
Mtrans = Vtrans * H_{DFL} /2 + ftg thk	1111.54	kN-m

Commentary

- The footing is intentionally protected with non-erodible rock to address scour issues.
- The effect of DFL is combined with Strength I.

6.3.5 TU, Shrinkage & Creep, Settlement

These force effects are excluded in this example. In the actual design, the effects of these forces must be considered if applicable.

6.3.6 Summary of unfactored loads

Loads at bottom of column

Loads	Pmax	Pmin	Vlong	Vtrans	Mlong	Mtrans
	kN	kN	kN	kN	kN-m	kN-m
DC_super	5600	5600	0	0	0	0
DC_cap	1102.5	1102.5	0	0	0	0
DC_col	1401	1401	0	0	0	0
DW	300	300	0	0	0	0
LL+IM	1912	0	0	0	0	2409
BF	0	0	163	0	2113	0
WA_owL	-265.33	-265.33	0	42	0	105
WA_dfL	-530.66	0.00	0	159	0	794
EQ_LC1	324.0	0.0	7227		30450	
EQ_LC2	97.2	0.0		4695		19491

Commentary

• The Pmax practically refers to condition when the structure is already completed, while Pmin refers to construction stages.

6.3.7 Load modifiers and factors

$Q = \Sigma \eta_i \Upsilon_i Q_i$ Total factored force effect, Q

where $\eta_i = \eta_D * \eta_R * \eta_I > 0.95$ Strength Limit State (maximum value)

$\eta_i = 1/(\eta_D * \eta_R * \eta_I) \leq 0.95$ Strength Limit State (minimum value)

η_i = load modifier

Υ_i = load factors

η_D = factor relating to Ductility

η_R = factor relating to Redundancy

η_I = factor relating to Operational Importance

Q_i = Force effects

Ductility				***For the Strength Limit State***
	η_D =	≥	1.05	for non-ductile components and connections
	η_D =	=	1.00	for conventional designs and details complying AASHTO Specs
	η_D =	≥	0.95	for components and connections that additional ductility enhancign measures are specified beyond required by AASHTO
	η_D =	=	1.00	***For all other Limit State***
Redundancy				***For the Strength Limit State***
	η_R =	≥	1.05	for non-redundant members
	η_R =	=	1.00	for conventional levels of redundancy, foundation elements where Ø already accounts for redundance as specified in Art 15.2)
	η_R =	≥	0.95	for exceptional levels of redundancy beyond girder continuity and torsionally-closed cross section.
	η_R =	=	1.00	***For all other Limit State***

•DGCS 10.3

•ϒi load factors specified in DGCS Table 10.3-1 and 10.3-2

Operational Importance				
				For the Strength Limit State
	ηI =	≥	1.05	for critical or essential bridges
	ηI =	=	1.00	for typical bridges
	ηI =	≥	0.95	for relatively less importance bridges
	ηI =	=	1.00	***For all other Limit State***

Table 10.3-1 Load Combination and Load Factors

Load Combination / Limit State	DC DD DW EH EV ES EL PS CR SH	LL IM CE BR PL LS	WA	WS	FR	TU	TG	SE	EQ	Use one of these at a time: BL	CT	CV
STRENGTH-I (Unless noted)	γ_p	1.75	1.00	-	1.00	0.50/1.20	0.0	γ_{SE}	-	-	-	-
STRENGTH-II	γ_p	1.35	1.00	-	1.00	0.50/1.20	0.0	γ_{SE}	-	-	-	-
STRENGTH-III	γ_p	1.35	1.00	-	1.00	0.50/1.20	0.0	γ_{SE}	-	-	-	-
STRENGTH-III	γ_p	-	1.00	1.4	1.00	0.50/1.20	0.0	γ_{SE}	-	-	-	-
STRENGTH-IV	γ_p	-	1.00	-	1.00	0.50/1.20	-	-	-	-	-	-
EH, EV,ES,DW,	1.5											
DC ONLY												
STRENGTH-V	γ_p	1.35	1.00	0.40	1.00	0.50/1.20	0.0	γ_{SE}	-	-	-	-
EXTREME EVENT - I	γ_p	γ_{EQ}	1.00	-	1.00	-	-	-	1.00	-	-	-
EVENT - I												
EXTREME EVENT - II	γ_p	0.5	1.00	-	1.00	-	-	-	-	1.00	1.00	1.0 0
EVENT - II												
SERVICE - I	1.00	1.00	1.00	0.30	1.00	1.00/1.20	0.0	γ_{SE}	-	-	-	-
SERVICE - II	1.00	1.3	1.00	-	1.00	1.00/1.20	-	-	-		-	-
SERVICE - III	1.00	0.8	1.00	-	1.00	1.00/1.20	0.0	γ_{SE}	-	-	-	-
SERVICE - IV	1.00	-	1.00	0.70	1.00	1.00/1.20	-	1.0	-	-	-	-
FATIGUE – 1 LL, IM, & CE ONLY	-	1.50	-	-	-	-	-	-	-	-	-	-
FATIGUE – II LL, IM, & CE ONLY	-	0.75	-	-	-	-	-	-	-	-	-	-

Table 10.3-2 Load Factors for Permanent Loads, γ_p

Type of Load	Load Factor: Max	Min
DC: Component and Attachments	1.25	0.90
DD: Downdrag	1.80	0.45
DW: Wearing Surfaces and Utilities	1.50	0.65
EH: Horizontal Earth Pressure		
Active	1.50	0.90
At-Rest	1.35	0.90
EL: Locked-in Erection Stresses	1.00	1.00
EV: Vertical Earth Pressure		
Retaining Walls and Abutments	1.35	1.00
Rigid Buried Structure	1.30	0.90
Rigid Frames	1.35	0.90
Flexible Buried Structures other than Metal Box Culverts	1.95	0.90
Flexible Metal Box Culverts	1.50	0.90
ES: Earth Surcharge	1.50	0.75

Load modifiers :

a. for strength limit (maximum values) $\eta_i =$ 1.05*1.00*1.0 = 1.05

$$Max=\eta_i\ [1.25DC+1.5DW+ 1.75(LL+BR+IM+LS+PL)+1.0WA\ +.10FR\ ])]$$

b. for strength limit (minimum values) $\eta_i =$ 1/(1.05*1.00*1.0) = 0.95

$$Min=\eta_i[0.90DC+0.65DW+0.75\ ES+1.75(LL+BR+IM+LS+PL)+1.0WA+1.0FR]$$

c. for all other limits state $\eta_i =$ 1.0

Commentary

- Load Factors: DGCS Table 10.3-1 and 10.3-2
- In this exercise, the bridge is classified as typical concrete bridge, non-ductile and conventional level of redundancy

6.3.8 Summary of factored and load combinations

STRENGTH 1		
Pmax =ηi [1 .25 (DC_super+DC_cap+DC_col)+1.5(DW) +1.75(LL + IM) +1.0(WA_dfL]	Pmax =	14064.51 kN
Pmin =ηi[0.9 (DC_super+DC_cap+DC_col)+0.65(DW) + 1.0(WA_owl)]	Pmin =	6861.68 kN
Vlong =ηi[1.25 (DC_super+DC_cap+DC_col)+1.5(DW) +1.75(BR)]	Vlong =	298.59 kN
Vtrans =ηi[1.25 (DC_super+DC_cap+DC_col)+1.5(DW) +1.0(WA_dfL)]	Vtrans =	166.73 kN
Mlong =ηi[1.25 (DC_super+DC_cap+DC_col)+1.5(DW) +1.75(BR)]	Mlong =	3881.72 kN-m
Mtrans =ηi[1.25 (DC_super+DC_cap+DC_col)+1.5(DW) +1.75(LL + IM) +1.0WA_dfL]	Mtrans =	5220.79 kN-m

- Modifier, ηi (Strength Limit)
  $\eta_D \geq$ 1.05
  $\eta_R \geq$ 1.00
  $\eta_I \geq$ 1.00
  for max. values
  ηi = 1.05 x 1.00 x 1.00
  ηi = 1.05
  for min. values
  ηi = 1/(1.05 x 1.0 x 1.0)
  ηi = 0.95

EXTREME EVENT 1		
Pmax =ηi[1.25 (DC_super+DC_cap+DC_col)+1.5(DW) +0.5(LL + IM) + 1.0(WA_owl)+ 1.0EQ]	Pmax =	11594.06 kN
Pmin =ηi[0.9 (DC_super+DC_cap+DC_col)+0.65(DW)+1.0(WA_owl)+1.0EQ]	Pmin =	7027.82 kN
Vlong =ηi[1.25 (DC_super+DC_cap+DC_col)+1.5(DW) +0.5(BR) +1.0EQ]	Vlong =	7307.93 kN
Vtrans =ηi[1.25 (DC_super+DC_cap+DC_col)+1.5(DW) +1.0(WA_owl) +1.0EQ]	Vtrans =	4736.71 kN
Mlong =ηi[1.25 (DC_super+DC_cap+DC_col)+1.5(DW) +0.5(BR)+1.0EQ]	Mlong =	31505.84 kN-m
Mtrans =ηi[1.25 (DC_super+DC_cap+DC_col)+1.5(DW) +0.5(LL + IM) +1.0WA_owl+1.0EQ]	Mtrans =	20800.55 kN-m

SERVICE 1		
Pmax =ηi[1.0 (DC_super+DC_cap+DC_col)+1.0(DW) +1.0(LL + IM)+1.0(WA_owl)]	Pmax =	10050.20 kN
Vlong =ηi[1.0 (DC_super+DC_cap+DC_col)+1.0(DW) +1.0(BR)]	Vlong =	162.50 kN
Vtrans =ηi[1.0 (DC_super+DC_cap+DC_col)+1.0(DW) +1.0(WA_owL)]	Vtrans =	42.1 kN
Mlong =ηi[1.0(DC_super+DC_cap+DC_col)+1.0(DW) +1.0(BR)]	Mlong =	2112.50 kN-m
Mtrans =ηi[1.0 (DC_super+DC_cap+DC_col)+1.0(DW) +1.0(LL+ IM) +1.0WA_owl]	Mtrans =	2514.40 kN-m

From load combinations above, it shows Extreme Event 1 is critical at LC1 combination (Long'L EQ direction) ...!!!!

Commentary

6.3.9 Verification of slenderness effect

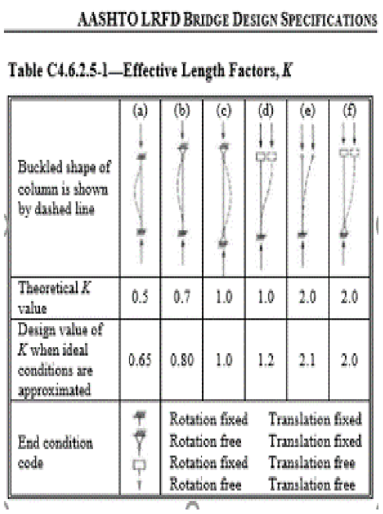

AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

Table C4.6.2.5-1—Effective Length Factors, K

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Design value of K when ideal conditions are approximated	0.65	0.80	1.0	1.2	2.1	2.0
End condition code	Rotation fixed Translation fixed Rotation free Translation fixed Rotation fixed Translation free Rotation free Translation free					

if kLu/r>=22 compute magnification factor

if kLu/r < 22 neglect slendersness effects

if kLu/r<=100 proceed to approximate evaluation of slenderness effect

Directions	LONG'L	TRANS.	
K	2.1	2.1	
Lu , m	11.0	11.0	clear height
Diameter, m	2.60		
$Ig=(\pi D^4)/64$, m^4	2.24		for circular column
$Ag = \pi D^2/4$, m^2	5.31		for circular column
r=sqrt(Ig/Ag), m	0.65		radius of gyration
kLu/r	35.54	35.54	
Ec, kN/m^2	27,000,000.00		
Cm	1		for all cases
ϕ_K	0.75		for concrete
β_d	0		
$EcIg = (Ec\ Ig/2.5)/(1+\beta_d)$, $kN\text{-}m^2$	24,214,015.80		
$P_e = \pi^2 EcIg/(kLu)^2$, kN	447,406.36		
Pu=factored axial load,kN	11,594.06		Pmax
$\delta_b = \delta_s = Cm/(1-(Pu/\phi_K P_e)) >=1$			

Moment magnification factor $\delta b = \delta s =$ 1.036

•DGCS 12.4.4.2

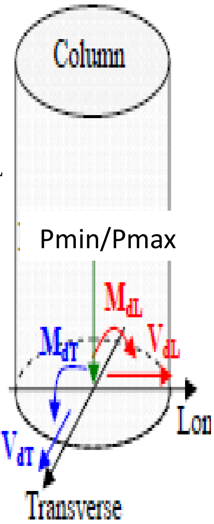
• AASHTO Table C.4.6.2.5.1 - effective Length Factor, K.

6.4 Verification of Column Flexural Resistance and Displacement

6.4.1 Verification of column resistance

DESIGN FORCES OF COLUMN

	Pmax =	11594.06	kN	
	Pmin =	7027.82	kN	
LC1	Vlong =	7307.93	kN	$=Vd_L$
LC1	Mlong =	32633.39	kN-m	$=Md_L$
	Vtrans =	4736.71	kN	
	Mtrans =	21544.97	kN-m	



Investigate :

Dcol = 2600 mm

Longitudinal bars = 76-36Ø

ρ_s = 1.44%

Mr = Flexural resistance of the column

Mr = Ø Mn where: Ø = 0.9 flexural resistance factor for column (either spiral or tied column)

Mn = nominal flexural resistance, kN-m to be obtained from "Interaction Diagram"

Mn = **37,000.00** kN-m

Mr = Ø Mn = **33,300.00** kN-m

Md = Mlong = 32,633.39 kN-m

c/d = **Mr/Md = 1.02 Ok in Long'l direction !!!**

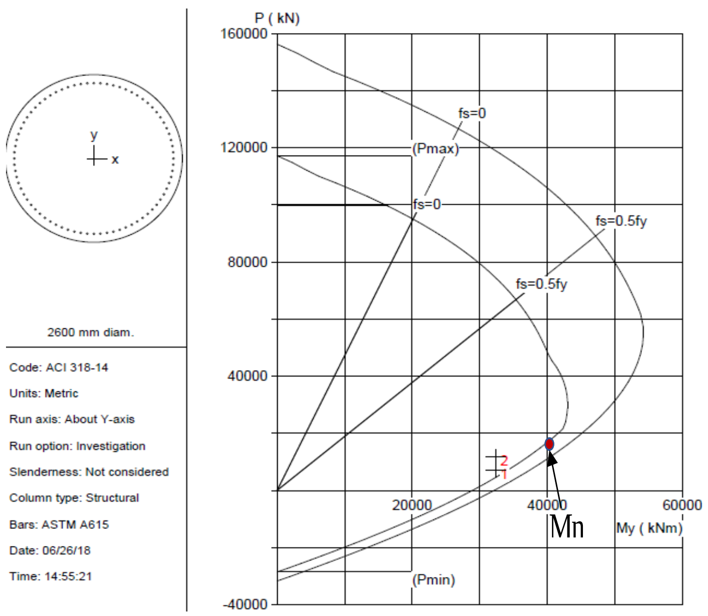
Verify min. and max. longitudinal reinf.

ZONE 1	No seimic requirement
ZONE 2	$0.01 < \rho_s < 0.06$
ZONE 3 an 4	$0.01 < \rho_s < 0.04$

For Zone 3 and 4:

ρ_{s_min} = **OK, satisfied with min. reinf. !!!**

ρ_{s_max} = **OK, satisfied with max. reinf. !!!**



Pmin = 7027.82 kN

Pmax = 11594.06 kN

Mong = 32336.51 kN-m

Commentary

- It shows the LC1 (Longitudinal direction) is the governing demand forces.
- Design moments are magnified due to slenderness effect.
- To avoid congestion of reinf. issues the ideal max ρs is 2.0% to 2.8%. The designer may resize the column. However, the displacement of column needs to satisfy. BSDS 4.7
- Mn is read from interaction diagram.
- DGCS 12.7.11
- The column is critical at Pmin or when in tension.

				Commentary
Column: pier1		Engineer: MFT		
f'c = 28 MPa	fy = 415 MPa	Ag = 5.30929e+006 mm^2	76 #11 bars	
Ec = 24870 MPa	Es = 200000 MPa	As = 76490 mm^2	rho = 1.44%	
fc = 23.8 MPa	e_yt = 0.002075 mm/mm	Xo = 0 mm	Ix = 2.24e+012 mm^4	
e_u = 0.003 mm/mm		Yo = 0 mm	Iy = 2.24e+012 mm^4	
Beta1 = 0.846954		Min clear spacing = 61 mm	Clear cover = 113 mm	
Confinement: Spiral				
phi(a) = 0.85, phi(b) = 0.9, phi(c) = 0.75				

6.4.2 Verification of column displacement

a. <u>Allowable displacement requirement:</u>

.•BSDS 4.7

•The column need to satisfy the limit of displacement.

$$\Delta allow = 0.25\ Ø\ Mn/PdL$$

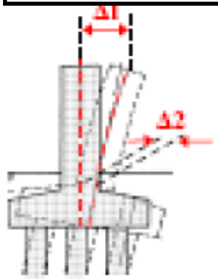
where:
PdL= factored axial dead load on top of column, N

DL on top of column	unfactored, N	factor	factored PdL, N
DC_super =	5,600,000.00	1.25	7,000,000.00
DC_cap =	1,102,500.00	1.25	1,378,125.00
DW =	300,000.00	1.25	375,000.00
Total =PdL =	**7,002,500.00**		**8,753,125.00**

Δallow = 0.25ØMn/PdL= **<u>951 mm</u>**

b. <u>Actual design displacement</u>:

Definition of **Δe (analysis result)**



Δe = Δ1- Δ2 (m)
Where
Δ1 = Displacement at pier column top (m)
Δ2 = Displacment at pier column base (m)

$$\Delta actual = Rd * \Delta e$$

where:

Δ – displacement of the point of contraflexure in the piers/columns relative to the point of fixity for the foundation, mm

Δe - displacement calculated from elastic seismic analysis, mm
Rd = (1-1/R)*(1.25*Ts/T) + (1/R) , if T<1.25*Ts
R= modification factor BSDS Art. 3.8
T= period of fundamental mode of vibration, sec
Ts = corner period specified in BSDS Art 3.6.2, sec

Commentary

From analysis of acceleration response spectrum (Chapter 5):

T= 1.00 sec

Ts = 0.59 sec

1.25*Ts = 0.74 sec

by inspection : T > 0.74sec

therefore Rd = 1

From bridge seismic analysis due to mass or dead load only the displacement of top and bottom of column as follows:

Longitudinal direction:	
$\Delta_{top} = \Delta_1 =$	180 mm
$\Delta_{bot} = \Delta_2 =$	6 mm
$\Delta e = \Delta_{top} - \Delta_{bot}$	174 mm

Transverse direction:	
$\Delta_{top} = \Delta_1 =$	100 mm
$\Delta_{bot} = \Delta_2 =$	6 mm
$\Delta e = \Delta_{top} - \Delta_{bot}$	94 mm

Δactual = Rd * Δe = 174 mm

Δallow = 0.25ØMn/PdL = 951 mm

c/d = 5.47

OK in long'l displacement!!!

Δactual = Rd * Δe = 94 mm

Δallow = 0.25ØMn/PdL = 951 mm

c/d = 10.12

OK in transv. displacement!!!

Therefore use Column diameter = 2600 mm

Vertical bars = 76-36Ø

ρ_s = 1.44%

6.5 Verification of Column Shear Resistance

a. Obtain the corresponding shear from overstrength moment resistance

Shear design = ***VdL=Vp=*** **3,317.24 kN**

The nominal shear resistance, *Vn* is lesser of the following:

Vn = Vc + Vs + Vp OR *Vn =0.25 f'c bv dv + Vp*

where : $V_c = 0.083*\beta*Sqrt(f'_c)*b_v*d_v$

$V_s = [\ A_v*f_y*d_v*(cot\theta + cota)sina\ ] / s$

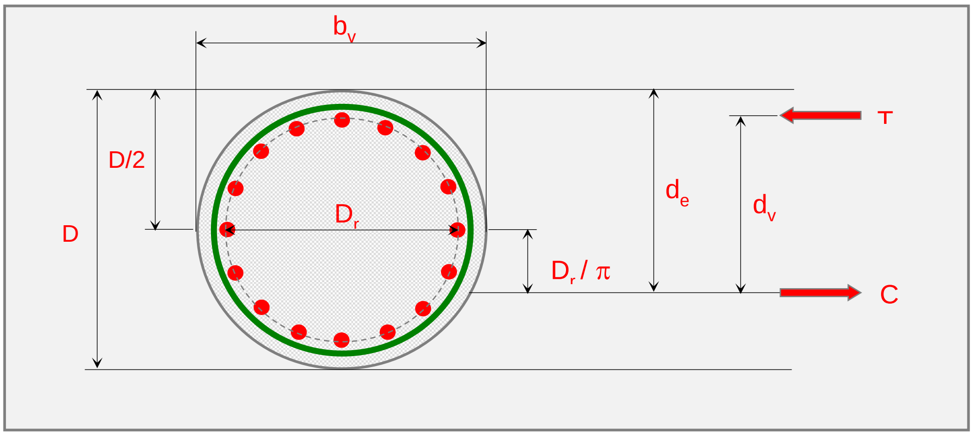
$V_p = 0.00$ *for non-prestressed concrete*

Ultimate shear resistance, $V_r = ØV_n$: $V_r = ØV_n > Vp$ where Ø = 0.9 for shear

Shear design parameters:

• BSDS 5.3.4 : The design shear Vd = Vp for single column or multiple columns. While Vd is the lesser of Vd or Vp for pile bent.

D	diameter of pier column (out to out dimension)	2600	mm
cc	concrete cover (to spiral or hoops) (Table 12.9.2-1)	100	mm
db	main bars or longitudinal bars, mm	36	mm
ds	size of spirals	20	mm
f'c	concrete compressive srrength	28	MPa
fy	yield strength of reinforcing steel	415	MPa
b_v	effective web width, mm = diameter of column	2600	mm
de	de = (D/2) + (D_r / π)	2040	mm
d_v	effective shear depth = .90 x de	1836	mm
β	factor indicating ability of diagonally cracked concrete to transmit tension	2.0	
θ	angle of inclination of diagonal compressive stresses	45	deg
α	angle of inclination of transverse reinforcement	90	deg
D_r	diameter of circle passing thru centers of the longitudinal reinforcement, mm	2324	mm
D_c	diameter of cirle measured outside the diameter of spiral/hoops, mm	2400	mm
A_c	Area of core measured outside diameter of spiral/hoops, mm^2	4521600	mm^2
A_g	Gross area of secction	5306600	mm^2



: For definition of terms:

b. Calculate the shear resistance:

From above parameters:

ds	size of spirals	20	mm
A_{sp}	area of shear reinforcement	314	mm
s	***Try spacing of transverse steel***	100	mm^2
No	***Try number of legs***	4	*No* .of legs
Total area of shear reinf for two(2) sides : Av = Av = *No**(A_{sp})		1256	mm^2

Definition of no. of legs	
leg leg	
no. of spirals	**No.of legs**
single spiral	2 legs
bundle spiral	4 legs

cot θ = 1/tan 45^0 = 1

cot α = 1/tan 90^0 = 0

sin α = sin 90^0 = 1

Commentary

- •DGCS Eq 12.5.3.2-1
- •DGCS Eq 12.5.3.2-2
- •DGCS Eq 12.5.3.2-3
- •DGCS Eq 12.5.3.2-3

•DGCS 12.5.3 for the definition of terms

•DGCS recommends the spiral type for confinement of circular columns.

Commentary

V_c =	0.00	kN	for conservative approach, assume Vc=0.0, i.e. concrete has no shear strength.
V_p=	0.00	kN	For non prestressed concrete

V_s = [Av*fy*dv*(cotθ + cotα)sinα]/s = 9571 kN

Vn is lesser of the following:

$Vn = Vc + Vs + Vp$	=	9571 kN	**Governing shear capacity!!!**
$Vn = 0.25 f'c\ bv\ dv + Vp$	=	33417.29 kN	

Vd_L = Vp= = 3317 kN

Ultimate shear resistance, V_r = 8614 kN c/d = 2.60 Vr > Vd, therefore Ok !!!

c. Verification of shear/transverse reinforcement for confinement at plastic hinges

•DGCS 12.7.11.2

Length of column end region will be the largest of the following :

a.) Max. cross- sectional dimension = 2.60 m

b.) 1 / 6 clear height = 1.83 m clear height = 11.00 m

c.) 450 mm (18 inches) = 0.45 m

therfore, **Length = 2.60** m

d. Verification of minimum required transverse reinforcement

1. The greater of the following:

$\rho_{s1} = 0.12*(f'_c/f_y)$ = 0.00810 Governs!!!

Note: for circular shape:

$\rho_{s2} = 0.45*[(A_g/A_c) - 1] *(f'_c/f_y)$ = 0.00527

2. Checking from provided confinement, where Av represent spiral leg on one (1) side

$\rho_{s\ provided} = \dfrac{4*A_v}{D_r*s}$ = 0.01081 **Ok !!!**

Maximum spacing

1. S should not be greater than 1/4 min dimension of member (=D/4) **OK!**
2. S should not be greater than 100mm **OK!**

Minimum clear spacing

1. Sc should not be less than 25mm **OK!**
2. Sc should not be less than 1.33 x aggregate size (1.33 x 25mm) **OK!**

•DGCS12.5.2.4

e. Transverse reinforcement for outside region

ds	size of spirals	20	mm
A_{sp}	area of shear reinforcement	314	mm^2
s	Try spacing of transverse steel	100	mm
No	Try number of legs	2	No.of legs

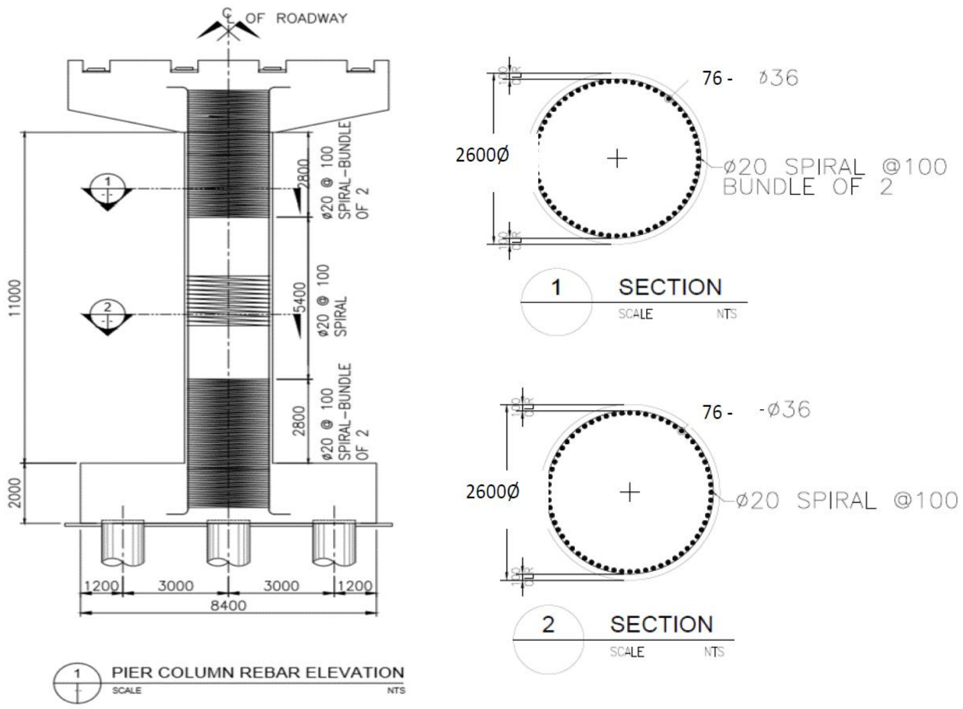
Min area of transverse reinf. Spacing:

Asp=.083xsqrtf'c x bv x S/fy 275 mm^2 **Ok !!!**

f. Summary of shear/transverse reinforcement

Location	Range, m	Spacing /Pitch of spirals, mm	diam, mm	No. of legs
End regions	2.60	100	20	4
Outside region	5.80	100	20	2

g. Column Details



Commentary

•Typical details of column. Other miscellaneous details not shown.

6.6 Verification of Pile Cap Resistance

6.6.1 Calculate the inelastic hinging forces:

a. Determine the pier/column overstrength moment resistance, Mp= ØMn

where:

Mn = from Column interaction diagram, see "Verification of column resistance " analysis.

Ø = 1.3

Mn = 37,000.00 kN-m

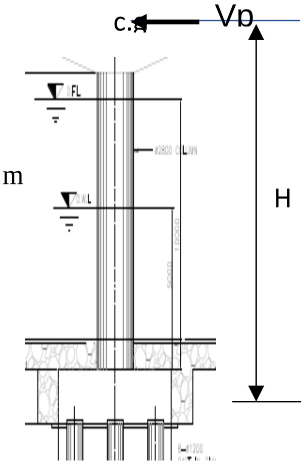
Mp= **48,100.00** kN-m

•Mn is read from interaction diagram.

Commentary

b. Calculate the corresponding plastic shear force, Vp= Mp/H

where:



H = is the height from the application of seismic inertial force from superstructure to the pier/column base

H= hcol + pilecap + c.g. = (11m + 2m + 1.5m) = 14.5 m

therefore Vp = **3,317.24** kN

c. Calculate the inertial seismic effect of pilecap weight and rock on top of pilecap.

where:

As = acceleration coeff =	0.53	
hf=height of pilecap =	2.0	m
hrock=height of rock =	1.00	m
L.A. = lever arm from pilecap base	see Fig.	

Gravity

DC_pilecap = 8.4m x 8.4 m x 2m x Υ_c = 3387 kN

DC_rock = (8.4m x 8.4 m - πDcol^2/4)x 1m x Υr = 1288 kN

Inertial seismic effect of foundation:				
Vf = shear by pilecap =	Vf =	1795.11	kN	
Vrock = shear by rock =	Vrock =	682.64	kN	
Mf = moment by pilecap =	Mf =	1795.11	kN-m	footing
Mrock = moment by rock =	Mrock =	1706.6	kN-m	rock

d. Calculate the axial forces at the footing base

Permanent Loads:

(from load analysis of column)

DC_super = R_{DL} =	DC_super =	5600	kN
DW = future wearing surface =	DW =	300	kN
DC_cap =	DC_cap =	1102.5	kN
DC_col=	DC_col =	1401	kN
DC_pilecap =	DC_pilecap =	3387	kN
DC_rock =	DC_rock =	1288	kN

Transient Loads:

Vehicular Truck + Lane Loads (from live load analysis of pier column)

Pmax = Reaction (2-HL + Lane Load) x m x no. of vehicle (**note: no IM for ftg)**

Pmax = 475kN + 324kN x 1 x 2 1598.00 kN

•Overburden weights such as overlaid rocks and the selfweight of footing to be added to inertial seismic effects.

Buoyancy Force @ OWL condition

H_sub = height of submerged

yw = unit weight of water	10	kN/m^3		
A_{coL} =	6.15	m^2	5	m
A_ftg =	70.56	m^2	2	m
Arock=	64.41	m^2	1	m

$\gamma_{coL} = A_{coL}$ * H_sub * yw =	308	kN
γ_{ftg}= A_ftg *H_sub * yw =	1411	kN
γ_{rock} = Arock * H_sub * yw	644	kN
Total	**-2363**	kN

Summary of axial loads

Loads	Unfactored, kN	Load Factors		Factored, kN	
	P	**Pmax**	**Pmin**	**Pmax**	**Pmin**
DC_super	5600	1.25	0.9	7000.00	5040.00
DC_cap	1102.5	1.25	0.9	1378.13	992.25
DC_col	1401	1.25	0.9	1751.25	1260.90
DC_pilecap	3387	1.25	0.9	4233.75	3048.30
DC_rock	1288	1.25	0.9	1610.00	1159.20
DW	300	1.5	0.65	450.00	195.00
WA (buoyancy force) ↑	-2363	1.0	1.0	-2362.80	-2362.80
LL	1598	0.5		799.00	
EQ	324	1.0		324.00	
			Total	**15,183.33**	**9,332.85**

e. Summary of inelastic hinging forces for Longitudinal direction of the bottom of footing

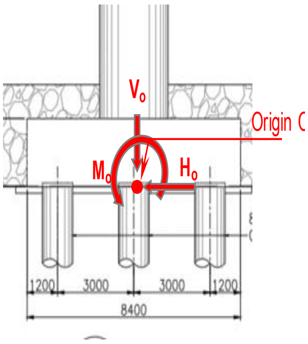
Total Mp = Mp + Mf + Mrock =Md

Total Vp = Vp + Vf + Vrock = Vd

Pmax = permanent + transient +buoyancy force

Pmin = permanent + transient +buoyancy force

Mp=M_D	**51,602**	kN-m
Vp=Vd=	**5,795**	kN
Pmax=	**15,183**	kN
Pmin =	**9,333**	kN



Note: Inelastic hinging forces to be input into : Pile Stability and Structural Resistance analysis spreadsheet for the investigation of piles.

f. Determine pile reactions from pile stabilty and structural analysis

Row 1 = F1 =	4595	kN
Row 2 = F2 =	1900	kN
Row 3 = F3 =	-1525	kN

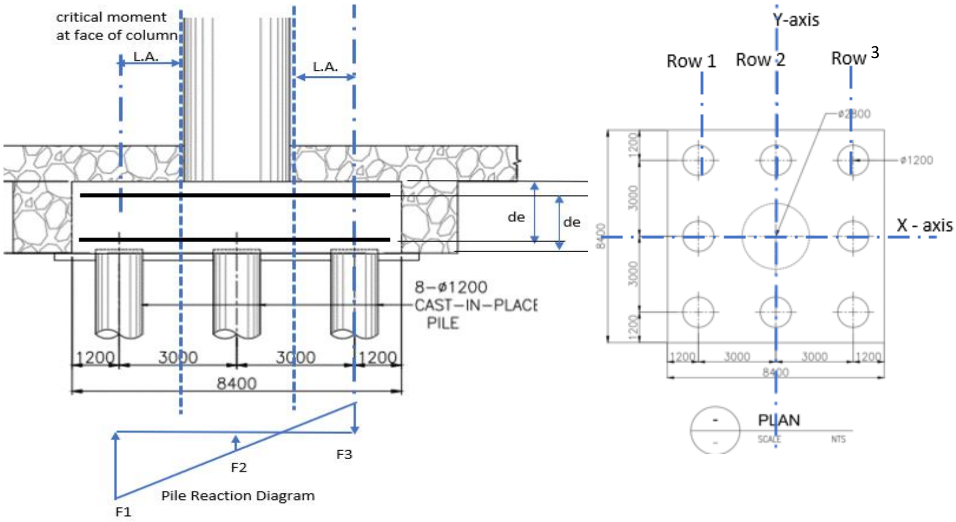
Commentary

- (-) of buoyancy is the uplift force effect of the submerged structure.

- DGCS 10.3 for the load factors.

Commentary

g. Determine design Moment and Shear of pile cap



- **Determine weight of pile cap and overburden load**

weight of part ftg = 2.8m x 8.4m x 2m x Υ_c	=	1129	kN
weight of part rock = 2.8m x 8.4m x 1m x Υ_r	=	470	kN

- **Determine design Moment for compresion side.**

F1_total =3piles x F1 -weight of ftg - weight of rock	=	12186.00	kN
Md=F1_total x L.A. =	=	19497.60	kN-m

- **Determine design Moment for tension side**

F3_total =3piles x F3 +weight of ftg + weight of rock =		-6174.00	kN
Md=F3_total x L.A. =	=	-9878.40	kN-m

6.6.2 Verification of flexure resistance

Design parameters:

Bottom main reinf. , db	36	mm	Ab = 1017
Pile embedment	100	mm	
Concrete cover	75	mm	
Clearance of reinf from pile tip	30	mm	
b=width of footing along y-axis	8400	mm	
w=width of footing along x-axis	8400	mm	
Concrete comp. strength, f'c	28	MPa	
yield strenght of steel, fy	415	MPa	

a. Flexure design on compression side

Minimum reinforcement

•DGCS 12.4.3.3

Commentary

$Mcr = \Upsilon_3 (\Upsilon_1 f_r) S_c$ flexural cracking moment

where:

fr =	3.33	MPa	(fr = 0.63*√f'c)
Υ_1 =	1.6		for all other concrete
Υ_3 =	0.67		for fy =415 Mpa reinforcement
$S_c = (b*hf^2)/6$	5.6	m^3	Section modulus
Mcr =	20,012.55	kN-m	
Mu_min =1.33*Md =	25,931.81	kN-m	
Mu_min = min(Mcr, Mu_min) =	20,012.55	kN-m	
Md=	19,497.60	kN-m	

Condition: if((Md > Mu_min), Md, Mu_min)

Therefore, Md = **20,012.55** kN-m

Compute for reinforcement

•DGCS 12.4.2.1

β1 =		0.85		
de = hf-pile embedment-clearance-db/2	de=	1852	mm	
m1=0.85*f'c/fy=	m1	0.057		
m2=2/(0.85*f'c)=	m2=	0.084	mm^2/N	
Rn= Md/(Ø b*de^2) =	Rn =	0.772	MPa	
ρ = m1*(1-sqrt(1-m2*Rn)) =	ρ =	0.00189		
As = ρ * b *de =	As=	29,416.44	mm^2	
Spacing of bars =Ab*b/As	S=	291	mm	say S_prov = 200 mm
As_prov =Ab*b/S_prov	As_prov =	42729.1	mm^2	
c=As_prov *fy/(0.85*f'c*β1*b	c=	104	mm	
a=c*β1 =	a=	89	mm	
Mn =(As_prov*fy)(de-a/2) =	Mn=	32,054.32	kN-m	

• Conservative spacing is assumed to satisfy other design requirements

• **Check net tensile strain, ξt**

ξt = 0.003*((de/c)-1) = ξt= 0.050 **Tension Controlled!!!, Reduction factor =0.9**

• **Check flexural Capacity** ØMn = **28,848.89** kN-m

• DGCS 12.4.3.4

c/d= 1.44 **Section is safe!!!**

• **Check control cracking by distribution of reinforcement**

Note: This provision applies to all members when tension in the cross section exceeds the 80% of the modulus rupture @ applicable service limit load combination.

• DGCS 12.1.1.6

• DGCS 12.4.3.4

Moment demand at Service 1 Limit	Ms =	**2,525.00**	kN-m	**<<<Service Limit Load Combination!!!**
fr = 0.52*√f'c	fr =	2.75	MPa	
	80%fr =	2.20	MPa	
fs = Ms/Sc	fs=	0.45	MPa	

checking: fs < 80%fr **Section 12.4.3.4 need not to satisfy**

Commentary

- **Check for minimum spacing of reinforcement**

• **DGCS 12.7.3.1**

For cast in place concrete, clear distance between paraller bars in a layer shall not be less than:

▪ 1.5 x nominal diam of bars	=	54	mm	satisfied the required min. spacing!!!
▪ 1.5 x max. size of aggregates	=	37.5	mm	satisfied the required min. spacing!!!
▪ 38mm	=	38	mm	satisfied the required min. spacing!!!

- **Check for maximum spacing of reinforcement (for walls and slabs)**

• DGCS 12.7.3.2

▪ s < 1.5 x hf	=	3000	mm	satisfied the required max. spacing!!!
▪ 450mm	=	450	mm	satisfied the required max. spacing!!!

b. Flexure design on tension side

• DGCS 12.4.3.3

Minimum reinforcement

Mu_min =	20,012.55	kN-m
Md=	9,878.40	kN-m
Condition: if((Md > Mu_min), Md, Mu_min)		
Therefore, Md =	**20,012.55**	kN-m

Compute reinforcement

Top main reinf. , db		28	mm	Ab =	615	mm^2
de = hf-concrete cover-db/2 =	de=	1911	mm			
m1=0.85*f'c/fy=	m1	0.057				
m2=2/(0.85*f'c)=	m2=	0.084	mm^2/N			
Rn= Md/(Ø b*de^2) =	Rn	0.725	MPa			
ρ = m1*(1-sqrt(1-m2*Rn)) =	ρ =	0.0018				
As = ρ * b *de =	As=	28,478.76	mm^2			
Spacing of bars =Ab*b/As	S=	182	mm	say S_prov =	175	mm
As_prov =Ab*b/S_prov	As_prov =	29541.1	mm^2			
c=As_prov *fy/(0.85*f'c*β1*b	c=	72	mm			
a=c*β1 =	a=	61	mm			
Mn =(As_prov*fy)(de-a/2) =	Mn=	23,052.14	kN-m		Stru	
• **Check net tensile strain, ξt**						
ξt = 0.003*((de/c)-1) =	ξt=	0.076	**Tension Controlled!!!, Reduction factor =0.9**			
• **Check flexural Capacity**	ØMn =	**20,746.92**	kN-m			
	c/d	1.04	**Section is safe!!!**			

Note: No need to investigate DGCS 12.4.3.4, 12.7.3.1 and 12.7.3.2. Already satisfied above.

6.6.3 Verification of shear resistance

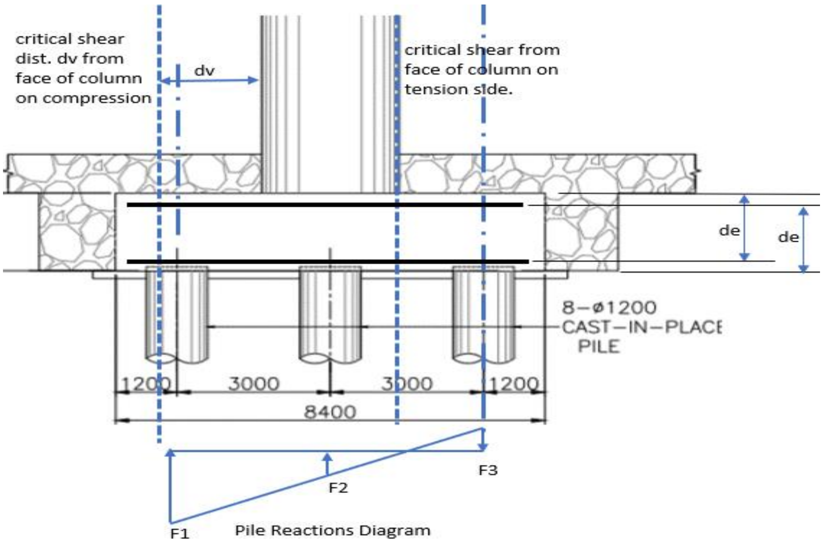
DGCS provides 3 procedures of determining shear resistance:

• DGCS 12.5.3

- Simplified procedure for non-prestressed sections
- General procedure <<<< *this procedure is applicable for design of footing and slab for larger thickness*
- Simplified procedure for prestressed and prestressed sections

• DGCS 12.5.3.3.2

Commentary



a. Check shear at tension side

Vu=Vd = F3_total =		**6174.00**	kN
Nominal shear resistance,	Vn = min(Vn1, Vn2)		
where :			
Vn1=	Vc + Vs		
Vc=	0.083*β(f'c^0.5)bv dv		
Vs=	[Av*fy*dv*(cot θ + cot α)*sinα] / S		
Vn2 =	0.25*f'c*bv*dv		
bv =	b =	8400	mm
de =		1911	mm
dv1 =	(de - a/2) =	1880	mm
dv2=	0.9 x de =	1720	mm
dv3=	0.72hf =	1440	mm
dv = max (dv1, dv2, dv3) =		1880	mm
θ=		45°	
α=		90°	

Compute for β :

$$\beta = \frac{4.8}{(1+750\varepsilon_s)}\frac{51}{(39+s_{xe})}$$

DGCS Eq 12.5.3-2
β in english units

$$\varepsilon_s = \frac{\left(\frac{|M_u|}{d_v}+0.5N_u+|V_u-V_p|-A_{ps}f_{po}\right)}{(E_sA_s+E_pA_{ps})}$$

where :

Mu =	9,878,400,000.00	N-mm	(absolute value of the factored moment, not to be taken less than IVu-VpI *dv, N-mm)
dv =	1880	mm	
Nu =	0.00	N	
Vu =	6,174,000.00	N	(absolute value)
Es =	200,000.00	MPa	
As =	29541	mm^2	
EsAs =	5,908,224,000.00	N	
EpAps =	0		
Apsfpo =	0	for non-prestressed concrete	
Vp =	0		
*IVu-VpI*dv* =	11,609,211,896.47	N-mm	
therefore *Mu* =	11,609,211,896.47	N-mm	
Ɛs =	0.00209		
Ɛs limitations =	-0.4 x 10^-3 < ε < 0.001		
therefor use Ɛs =	0.001		

The crack spacing parameter, Sxe, shall be determined as :

Sxe = Sx [35/(ag+16)], mm

where :

ag =size of aggregates	20	mm	
Sx =	350	mm	
Sxe=	340	mm	
Limitations:	300mm≤Sxe≤2025mm		
	limit to min.:	**OK!!!**	
	limit to max.:	**OK!!!**	
β =	2.670		
Vc=	18520	kN	
Vs=	0.0	kN	assume no transverse reinf.
Vn1 =	18520	kN	
Vn2 =	110564	kN	
Ultimate shear resistance = ØVn	16668	kN	
Shear demand, Vd =	6174	kN	
c/d =	2.70	**Section is safe in shear!!!**	

a. Check shear at compression side

Vu=Vd = F1_total = **12186.00** kN

Note: By inspection, approximately 1/4 of pile section is the critical shear.

Commentary

•It is a design approach not to require transverse reinforcement for footing, walls or slab, hence Vs= 0 kN.

Say about 50% of Vu will be effective ,

Reduced shear demand: Vu=Vd =50% F1_total = **6093.00** kN

Nominal shear resistance, Vn = min(Vn1, Vn2)

where :

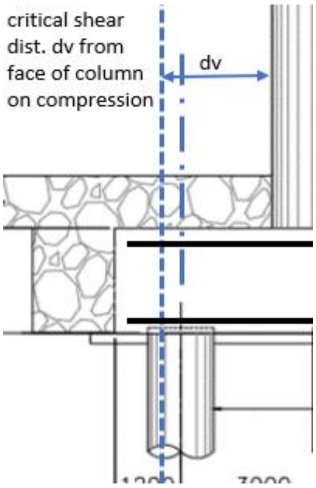
Vn1= Vc + Vs

Vc= 0.083*β(f'c^0.5)bv dv

Vs= [Av*fy*dv*(cot θ + cot α)*sinα] / S

Vn2 = 0.25*f'c*bv*dv

bv =	b	8400	mm
de =		1852	mm
dv1 =	(de - a/2)	1808	mm
dv2=	0.9 x de	1667	mm
dv3=	0.72hf	1440	mm
dv = max (dv1, dv2, dv3) =		1808	mm
θ=		45^o	
α=		90^o	



Compute for β :

$$\beta = \frac{4.8}{(1+750\varepsilon_s)} \frac{51}{(39+s_{xe})}$$

DGCS Eq 12.5.3-2

β in english units

$$\varepsilon_s = \frac{\left(\frac{|M_u|}{d_v}+0.5N_u+|V_u-V_p|-A_{ps}f_{po}\right)}{(E_sA_s+E_pA_{ps})}$$

where:

Mu =	19,497,600,000.00	N-mm	(absolute value of the factored moment, not to be taken less than IVu-VpI *dv, N-mm)
dv=	1808	mm	
Nu =	0.00	N	
Vu=	6,093,000.00	N	(absolute value)
Es=	200,000.00	MPa	
As=	42729	mm^2	
EsAs=	8,545,824,000.00	N	
EpAps =	0		
Apsfpo=	0	for non-prestressed concrete	
Vp =	0		
*IVu-VpI*dv* =	11,014,016,314.16	N-mm	
therefore *Mu* =	19,497,600,000.00	N-mm	
Ɛs =	0.00198		
Ɛs limitations =	-0.4 x 10^-3 < ε < 0.001		
therefor use Ɛs =	0.001		

Commentary

•By inspection the reaction of pile with effect in shear is about 1/4 of the cross section of pile. It is conservative to take shear 50% effective of the pile.

	Commentary
The crack spacing parameter, Sxe, shall be determined as :	

Sxe = Sx [35/(ag+16)], mm

where :			
ag =size of aggregates	20	mm	
Sx	350	mm	
Sxe=	340	mm	
Limitations:	300mm≤Sxe≤2025mm		
	limit to min.:		**OK!!!**
	limit to max.:		**OK!!!**
β =	2.670		
Vc=	17804	kN	
Vs=	0.0	kN	assume no transverse reinf.
Vn1 =	17804	kN	
Vn2 =	106290	kN	
Ultimate shear resistance = ØVn	16024	kN	
Shear demand, Vd =	6093	kN	
c/d =	2.63		**Section is safe in shear!!!**

6.6.4 Shrinkage and temperature reinforcement

•DGCS 12.7.8

Reinforcement for shrinkage and temperature stressses shall be provided near surfaces of concrete exposed to daily temperature changes and in structural mass concrete. For the pile cap, basrs shall be provided at the perimeter side of the pile cap.

Ast => 0.75MPa x (bxh)/2(b+h)fy

where:

b = least width of component section, mm 8400 mm

h=least thickness of component section, mm 2000 mm

a) Ast => 0.75MPa x (bxh)/2(b+h)fy 1.460 mm^2/mm

b) Limitations: 0.223<Ast<1.27

if Ast > 1.27, use 1.27 as the max. limit or if Ast < 0.223, use 0.223 as min. limit

therefore Ast =	1.27	mm^2/mm	***use max limit!!!***
try diameter db =	20	mm	
Ab =	314	mm^2	
Spacing, S=	247	mm	
say S_prov =	350	mm	should match Sx
: Ast=Ab/S_prov =	0.897	mm^2/mm	satisfied max. limitations!!!

where :

a) S_prov should not be greater than 3 X thickness, or 450mm

b) S_prov should not be greater than 300 for walls and footings > 450mm thk.

c) S_prov should be 300mm for other components > 900mm thick.

therefore use S = 300mm for shrinkage and temperature bars to satisfy item c)

6.6.5 Verification of two-way shear action (punching shear) for pile

Commentary

•DGCS 12.10.3.5

Factored axial Pu of pile, F1 = **4595** kN

The nominal shear resistance, V_n in N is lesser of the following :

$V_{n1} = (0.17 + 0.33/\beta_c)(f'c^{0.5} b_o d_v)$ **•DGCS Eq 12.10.3.5-1**

OR

$V_{n2} = (0.33)(f'c^{0.5} b_o d_v)$

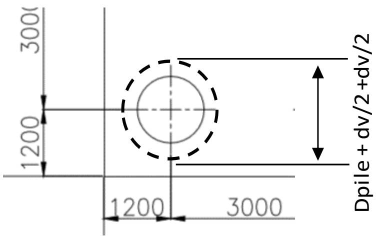
where:

β_c = ratio of long side to short side of the footing which the concentrated load or reaction is transmitted

bo = perimeter of the critical section, mm

dv = effective shear depth, mm

de =	1852	mm
a=	89	mm
hf=	2000	mm
Dpile =diam of pile, mm =	1200	mm
βc =	1	(i.e. for square footing)
dv1 =	1808	mm
dv2 =	1667	mm
dv3 =	1400	mm
dv = max (dv1, dv2, dv3) =	1808	mm
bo =	9444	mm
Vn_1=	45167	kN
$ØVn_1$=	40650	kN
OR		
Vn_2=	29810	kN
$ØVn_2$ =	26829	kN
Ult.punching resistance = ØVi	26829	kN
c/d =	5.84	**Pile is safe in punching shear!!!**



6.6.6 Verification of two-way shear action (punching shear) for column

Factored axial Pu of column **15183** kN

The nominal shear resistance, V_n in N is lesser of the following :

$V_n = (0.17 + (0.33/\beta_c)(f'c^{0.5} b_o d_v)$ OR $V_n = (0.33)(f'c^{0.5} b_o d_v)$ **DGCS Eq 12.10.3.5-1**

where:

β_c = ratio of long side to short side of the footing which the concentrated load or reaction is transmitted

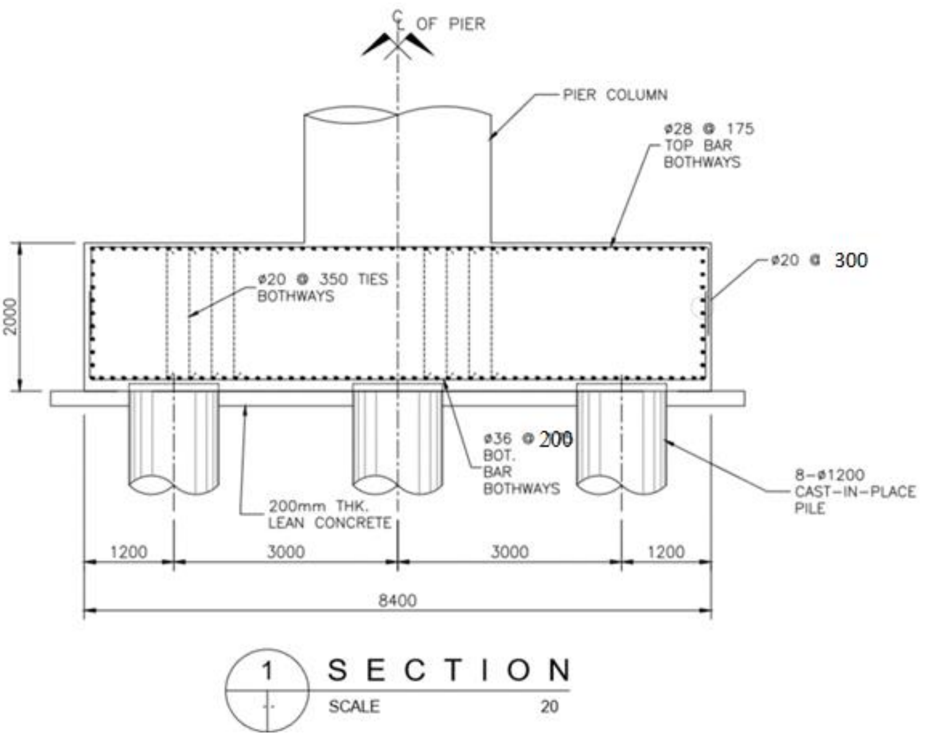
bo = perimeter of the critical section, mm

dv = effective shear depth, mm

de =	1852	mm
a=	89	mm
hf=	2000	mm
Dcol =diam of column, mm =	2600	mm

		Commentary
βc =	1 (i.e. for square footing)	
dv=	1808 mm	
bo =	13840 mm	
Vn_1=	66191 kN	
$ØVn_1$=	59572 kN	
	OR	
Vn_2=	48335 kN	
$ØVn_2$ =	43501 kN	
Ult.punching resistance = ØVı	43501 kN	
c/d =	2.87 **Column is safe in punching shear!!!**	

6.6.7 Pile cap Details



Note: In this exercise, the required reinforcement are provided both directions of same spacing and sizes where in the maximum design forces are based from Longitudinal direction which is the governing limit state. In actual practice, the designer may perform the design in the other direction to arrive to actual requirements.

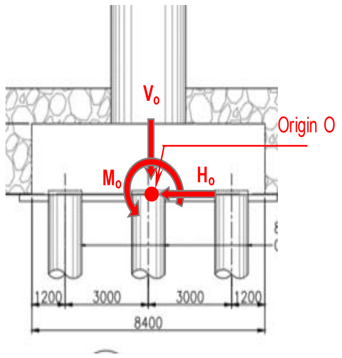
6.7 Pile Stability and Structural Resistance

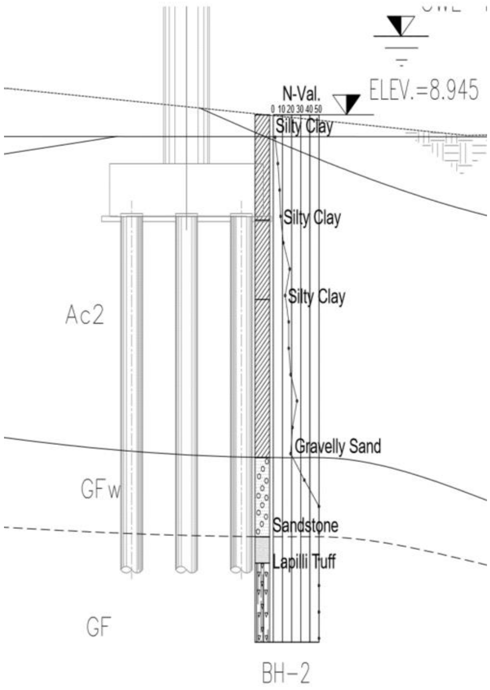
6.7.1 Determine the design forces of footing from section 6.6 Verification of Pile Cap Resistance

Design forces in due to Longitudinal Direction			
Forces acting at the bottom of pile cap/footing from inelastic forces			
Location	Vertical Load	Lateral Load	Moment
	V_o	H_o	M_o
	kN	kN	kN-m
Origin O	9350.00	5800.00	51600.00

Note: 1st trial is V_o = Pmin

Pmax 15200.0





PIER 1 - BH2

Commentary

- The verification of stability of piles is basically based on JRA method. Refer to BSDS sections 4 and 5.
- Piles shall be evaluated both Pmax and Pmin. Pmin is critical in the pull out action and flexural resistance. Pmax will govern in compression action.
- Borelog of Pier 1 is the basis of subsurface data.

6.7.2 Determine pile springs and geometric properties

a. Horizontal pile spring constant of pile (KH)

Note: The coefficient of subgrade reaction shall be determined, in principle, by using the modulus of deformation obtained from a variety of surveys and tests by considering the influence of loading width of foundations and other relevant factors:

BSDS Table C.4.4.2-1 Modulus of Deformation E_0 and *a*			
Modulus of deformation E_0 to be obtained by means of the following testing methods		*a*	
Method	Definition	Ordinary	Earthquake
Method A	A value equal to 0.5 of the modulus of deformation to be obtained from a repetitive curve of a plate bearing test using a rigid disc of 30cm. in diameter.	1	2
Method B	Modulus of deformation to be measured in the bore hole.	4	8
Method C	Modulus of deformation to be obtained by means of an unconfined or triaxial compression test of samples.	4	8
Method D	Modulus of deformation to be estimated from E_0 = 2,800*N using the N-value of the standard penetration test.	1	2

<<<< to be used

The coefficient of horizontal subgrade reaction should be obtained by using BSDS Equation C.4.4.2-4

Commentary

$$k_H = k_{HO} \left(\frac{B_H}{0.3} \right)^{-3/4}$$

BSDS EqC.4.4.2-4

where :

•BSDS section 4

k_H coefficient of horizontal subgrade reaction (kN/m^3)

k_{HO} coefficient of horizontal subgrade reaction coresponding to the value obtained by the plate bearing test using a rigid disc of diameter 0.3m, kHO = (a*E0/0.3) (kN/m3).

B_H equivalent loading width of foundation to be obtained from BSDS Table C.4.4.2-2 (m)

E_0 modulus of deformation at the design location, measured or estimated by the procedures in Table C.4.4.2-1

A_H loading area of foundation perpendicular to the load direction (m^2)

D loading width of foundation perpendicular to the load direction (m)

B_e effective loading width of foundation perpendicular to the load direction (m)

L_e effective embedment depth of a foundation (m)

$1 / b$ ground depth relating to the horizontal resistance and equal to or less than the effective embedment depth (m)

b characteristic value of foundation

EI flexural stiffness of foundation (kN-m^2)

$$K_{HP} = k_H \; A_{HP}$$

BSDS EqC.4.4.3-9

where :

K_{HP} horizontal spring constant of pile section corresponding to area A_{HP} (kN/m)

A_{HP} effective projected vertical area of the ground corresponding to pile spring K_{HP} (m2)

When analyzing the ground resistance of a pile foundation as a linear spring, the equivalent loading width B_H should take a value of $(D/b)^{1/2}$.

b. Geometric properties of piles

Select pile section:

☑Circular Section

☐Square Section

Select Pile Installation Method :

☐Driven Piles (Blow Method)

☐Driven Piles (Vibro-Hammer Method)

☑Cast-in-place RC Piles

☐Bored Piles

☐Pre-Boring Piles

☐Steel Pile Soil Cement Piles

•Cast-in-place rc piles are common use in the Philippines. Take note the Bored piles here refer to bored piles constructed according to Japan method.

Input Pile Dimension : Diameter 1.20 m
Input Number of Piles 8 piles
Input Pile Length : 13.00 m

Calculate Section Properties :

Cross-section Area 1.131 m^2
Perimeter of Pile : 3.770 m
Pile Moment of Inertia : 0.102 m^4
Pile Flexural Stiffness : 2.75E+06 $kN\text{-}m^2$

Concrete Material Properties :

Design Compressive Strength at 28th day 28 N/mm^2
Unit Density for Concrete 2400 kg/m^3
Unit weight for Reinforced Concrete 24 kN/m^3
Young's Modulus of Elasticity 2.70E+07 kN/m^2

Reinforcement Material Properties :

Minimum Yield Strength 415 N/mm^2
Ultimate Tensile Strength 620 N/mm^2
Young's Modulus of Elasticity 2.00E+08 kN/m^2

Method used to determine Modulus of Deformation : Method D ***BSDS Table C4.4.2-1***
Limit State used in determining subgrade coeff. Ordinary Condition
Coefficient to be used for estimating subgrade raction : 1 ***BSDS Table C4.4.2-1***
Unit weight of water 10 kN/m^3

Soil Layer Type	Layer Thicknes	N-Value	Unitweight γ_t	Unitweight γ'	aE_0	$(1/b_1)-d_i$	t_i	aE_0*t_i	Layer Depth	
	m	Average			kN/m^2	m	m	kN/m	m	
Clay	1.00	11	18.0	8.0	30800	3.103	1.000	30800.00		1.00
Clay	4.00	17	18.0	8.0	47600	-0.897	3.103	147687.58		5.00
Clay	4.00	22	18.0	8.0	61600	-4.897	0.000	0.00		9.00
Rock	4.00	50	20.0	10.0	140000	-8.897	0.000	0.00		13.00
Rock				-10.0						

Commentary

•The initial pile length is founded into hard strata @minimum of 1.m depth.

Assumption on Effective range of Horizontal Subgrade Reaction, $1/b_1$ 4.103 m

Note:
Typically it is 4 to 6 times as large as pile diameter

Try Between	
4D	6D
m	m
4.8	7.2

Iterate value of $1/b_1$ until becomes equal to $1/b$ Iterate

Initial Equivalent Loading Width, B_{H1} 2.219 m

Ave. value of Modulus of Deformation aE_0 within effective range of $1/b_1$ 43505.1 kN/m^2

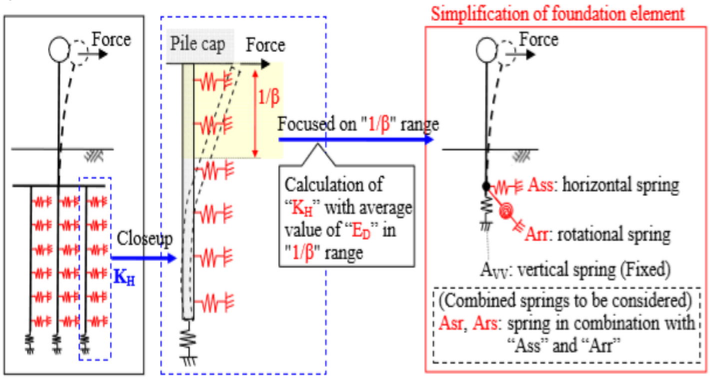
Coefficient of Horizontal Subgrade Reaction, K_H 32334.54 kN/m^3

32334.5 (based from derived equation)

Characteristic Value of Foundation, b 0.244 m^{-1}

$1/b$ 4.103 m

Horizontal Pile Spring Constant, k_H 159189.93 kN/m



GENERAL DETAILS OF SPRING CONSTANTS

c. Axial spring constant of pile (Kv)

Note :The axial spring constant KV of a single pile use for design shall be estimated from the empirical formula derived from the vertical pile loading test and results of soil test, or from load-settlement curves from vertical loading test.

$$K_V = \frac{a * A_p * E_P}{L}$$

where:

K_V axial spring constant of pile (kN/m)

a proportional coefficient (BSDS Equation C5.4.3.6-3)

Commentary

•The subgrade reactions are lumped approximatedly 4D to 6D to simplify the calculations.

Commentary

A_p	net cross-sectional area of pile (m^2)
E_p	Young's modulus of pile (kN/m^2)
L	pile length (m)
D	pile diameter (m)

Embedment Ratio, L/D 10.83

Note:

For Piles L/D<10, L/D=10

Proportional Coefficient, $\alpha = 0.031\ (L/D) - 0.15$ 0.186

Axial Spring Constant of Pile, K_V 436512.22 kN/m

d. Radial spring constrants pf pile (K1, K2, K3, K4)

Note: The radial spring constants K1 to K4 of a pile are:

K1, K3 radial force and bending moment (kN-m/m) to be applied on a pile head when displacing a unit displacement in the radial direction while keeping it from rotating (kN/m)

K2, K4 radial force and bending moment (kN-m/rad) to be applied on a pile head when rotating the head by a unit rotation in the radial direction while keeping it from moving in a radial direction (kN/rad)

Note: If the coefficient of horizontal subgrade reaction is constant irrespective of the depths and if the embedded depth of a pile is sufficiently long, the constants can be computed from BSDS Table C.4.4.3-2

Specify Limit State used in design : **During Earthquake**

Coefficient to be used, a : 2

Characteristic value of foundation, b' : 0.290 m^{-1}

Pile length above design ground surface, h : 0 m

$b'*L_e$: 3.77 Piles with semi-infinite length

Select restrictive condition of

☑Rigid Frame of Pile Head

☐Hinged Frame of Pile Head

Commentary

☑Rigid Frame of Pile Head

☐Hinged Frame of Pile Head

BSDS Table C4.4.3-2 - Hayashi Chang Theory

			Rigid		Hinged	
Radial Spring Constants of Piles, K_1 :	267724.49	kN/m	803175.46	267724.49	133862.24	133862.24
Radial Spring Constants of Piles, K_2 :	461815.26	kN-m/m	1385449.23	461815.26	0.00	0.00
Radial Spring Constants of Piles, K_3 :	461815.26	kN/rad	1385449.23	461815.26	0.00	0.00
Radial Spring Constants of Piles, K_4 :	1593229.95	kN-m/rad	1593229.95	1593229.95	0.00	0.00

6.7.3 Determine displacement and reaction force

Note: Pile reactions and displacements shall be evaluated considering the properties of the pile structure and the ground. In the displacement method, the coordinate is formed with the origin set at an arbitrary point O of the foundation. The origin O may be selected from arbitrary points, but it is recommended to coincide it with the centroid of the pile group below the pile cap/footing.

BSDS EqC.5.4.3.7-1

$$\left.\begin{aligned} A_{xx}*d_x + A_{xy}*d_y + A_{xa}*a &= H_o \\ A_{yx}*d_x + A_{yy}*d_y + A_{ya}*a &= V_o \\ A_{ax}*d_x + A_{ay}*d_y + A_{aa}*a &= M_o \end{aligned}\right\}$$

where:

H_o	lateral loads acting at the bottom of pile	dx	lateral displacement from origin O, m
V_o	vertical loads acting at the bottom of pil	dy	vertical displacement form origin O, m
M_o	moment (external force) at the origin O;	a	rotational angle of the footing at the origin O, rad

The displacements (d_x, d_y, and a) below are derived by solving BSDS Equation **C5.4.3.7-1 and C5.4.3.7-2 :**

BSDS EqC.5.4.3.7-3

$$\left.\begin{aligned} d_x &= \frac{H_o*A_{aa} - M_o*A_{xa}}{A_{xx}*A_{aa} - A_{xa}*A_{ax}} \\ d_y &= \frac{V_o}{A_{yy}} \\ a &= \frac{-H_o*A_{ax} + M_o*A_{xx}}{A_{xx}*A_{aa} - A_{xa}*A_{ax}} \end{aligned}\right\}$$

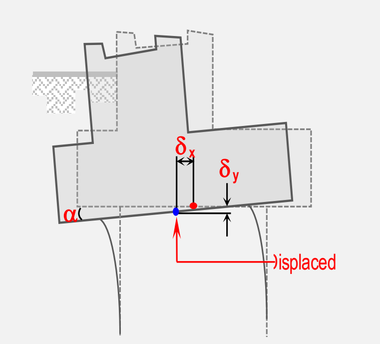
Commentary

BSDS C5.4.3.7-2 COEFFICIENTS FOR DISPLACEMENT CALCULATION								
Row	No. of Piles	x_i	q_i	*Ayy*	*Axx*	*Axa*	*Aax*	*Aaa*
1	3	3	0	1309537	803173	-1E+06	-1385446	16565520
2	2	0	0	873024	535449	-923631	-923631	3186460
3	3	3	0	1309537	803173	-1E+06	-1385446	16565520
			Sum =	3492098	2141796	-3694522	-3694522	36317499

$\cos(q_i)$	$\sin(q_i)$
1	0
1	0
1	0
1	0
1	0
1	0
1	0

a. Calculation for Displacement:

Longitudinal Displacement			
Location	Displacement		
	Lateral	Vertical	Rotational
	d_x	d_y	a
	m	m	rad
Origin O	0.0063	0.0027	0.0021



Deformed Shape

b. Calculation of Reaction:

By using the displacements at the footing origin O obtained from the results of the above calculations, the pile axial force, radial force, and moments acting on each pile head can be obtained using the following equations:

$$\left.\begin{aligned} P_{Ni} &= K_V * d_{yi}' \\ P_{Hi} &= K1 * d_{xi}' - K_2 * a \\ M_{ti} &= -K_3 * d_{xi}' + K_4 * a \end{aligned}\right\}$$

BSDS EqC.5.4.3.7-4

$$\left.\begin{aligned} d_{xi}' &= d_x * cosqi - (d_y + ax_i) * sinq_i \\ d_{yi}' &= d_x * sinq_i + (d_y + ax_i) * cosq_i \end{aligned}\right\}$$

BSDS EqC.5.4.3.7-5

Commentary

where:

d_{xi}' radial displacement at the i-th pile head, m

d_{yi}' axial displacement at the i-th pile head, m

x_i x-coordinate of the i-th pile head, m

q_i vertical axis angle from the i-th pile axis for battered pile, degree

P_{Ni} axial force of the i-th pile, kN

P_{Hi} radial force of the i-th pile, kN

M_{ti} moment as external force acting on the i-th pile head, kN-m

Summary of Pile Reaction due to Long'l Direction

Pile	Number of Piles	x_i	q_i	Axial P_{Ni}	Radial P_{Hi}	Moment M_{ti}
		m	deg.	kN	kN	kN-m
1	3	-3.00	0	-1525.35	725.00	388.27
2	2	0.00	0	1168.75	725.00	388.27
3	3	3.00	0	3862.85	725.00	388.27
	8					

$\cos(q_i)$	$\sin(q_i)$	
		-1525.4
1	0	1168.75
1	0	3862.85
1	0	
1	0	
1	0	
1	0	
1	0	

Maximum Axial Force for Capacity verification, $P_{Ni\text{-}max}$ 3862.85 kN

Minimum Axial Force for Capacity verification, $P_{Ni\text{-}min}$ -1525.35 kN

Graphing of Reaction Force and Displacement of each pile

PILE EMBEDDED IN THE GROUND (h = 0)

Depth	Rigid Pile Head Connection			Hinged Pile Head Connection		
	Deflection	Moment	Shear	Deflection	Moment	Shear
m	mm	kN-m	kN	mm	kN-m	kN
0.00	6.26	-388.27	-725.00	5.42	0.00	-725.00

Parameters			
Mt	388.272	kN-m	
Ph	725.000	kN	
E	2.70E+07	kN/m^2	
I	0.102	m^4	
b'	0.290	m^{-1}	-388.3

• In practice, the connections of piles into footing is rigid connection. Therefore, the values under the hinged head maybe ignored.

Commentary

1.00	4.31	-896.50	-316.71	3.88	-535.00	-364.86
2.00	2.67	**-1068.39**	-48.22	2.54	-767.35	-117.27
3.00	1.42	-1030.30	108.22	1.46	-800.98	36.13
4.00	0.54	-879.40	182.22	0.68	-719.08	117.51
5.00	-0.02	-684.30	200.75	0.15	-582.79	148.30
6.00	-0.33	-488.98	185.88	-0.16	-433.16	146.90
7.00	-0.46	-318.09	154.18	-0.31	-294.89	127.64
8.00	-0.48	-182.39	117.03	-0.36	-180.37	100.80
9.00	-0.43	-83.45	81.51	-0.34	-93.55	73.09
10.00	-0.35	-17.54	51.37	-0.29	-33.16	48.39
11.00	-0.26	21.58	28.03	-0.22	4.83	28.46
12.00	-0.17	40.77	11.43	-0.16	25.50	13.72
13.00	-0.11	46.39	0.70	-0.10	33.87	3.74
14.00	-0.05	43.71	-5.40	-0.06	34.30	-2.32
15.00	-0.02	36.71	-8.14	-0.02	30.22	-5.42
16.00	0.00	28.17	-8.65	0.00	24.14	-6.47
17.00	0.02	19.85	-7.84	0.01	17.70	-6.25
18.00	0.02	12.69	-6.41	0.01	11.87	-5.34
19.00	0.02	7.08	-4.80	0.02	7.11	-4.16
20.00	0.02	3.05	-3.29	0.01	3.55	-2.97
21.00	0.01	0.41	-2.04	0.01	1.11	-1.94
22.00	0.01	-1.12	-1.08	0.01	-0.40	-1.11
23.00	0.01	-1.83	-0.40	0.01	-1.19	-0.51
24.00	0.00	-2.01	0.02	0.00	-1.49	-0.11
25.00	0.00	-1.85	0.26	0.00	-1.46	0.13
26.00	0.00	-1.53	0.36	0.00	-1.27	0.25
27.00	0.00	-1.16	0.37	0.00	-1.00	0.28
28.00	0.00	-0.80	0.33	0.00	-0.72	0.27
29.00	0.00	-0.50	0.27	0.00	-0.48	0.22
30.00	0.00	-0.27	0.20	0.00	-0.28	0.17
31.00	0.00	-0.11	0.13	0.00	-0.13	0.12
32.00	0.00	0.00	0.08	0.00	-0.03	0.08
33.00	0.00	0.06	0.04	0.00	0.02	0.04
34.00	0.00	0.08	0.01	0.00	0.05	0.02
35.00	0.00	0.09	0.00	0.00	0.06	0.00
36.00	0.00	0.08	-0.01	0.00	0.06	-0.01
37.00	0.00	0.06	-0.02	0.00	0.05	-0.01
38.00	0.00	0.05	-0.02	0.00	0.04	-0.01
39.00	0.00	0.03	-0.01	0.00	0.03	-0.01
40.00	0.00	0.02	-0.01	0.00	0.02	-0.01

ho	0.536	m	-1074
l_m	2.249	m	

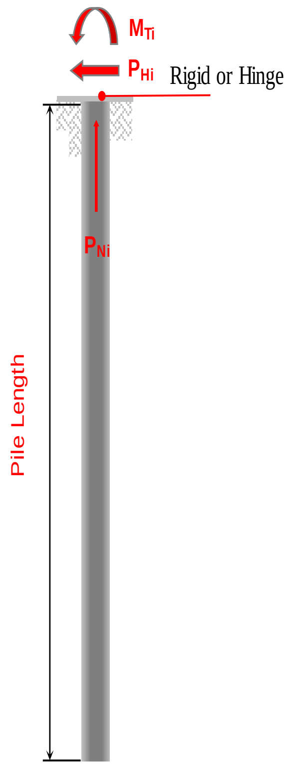
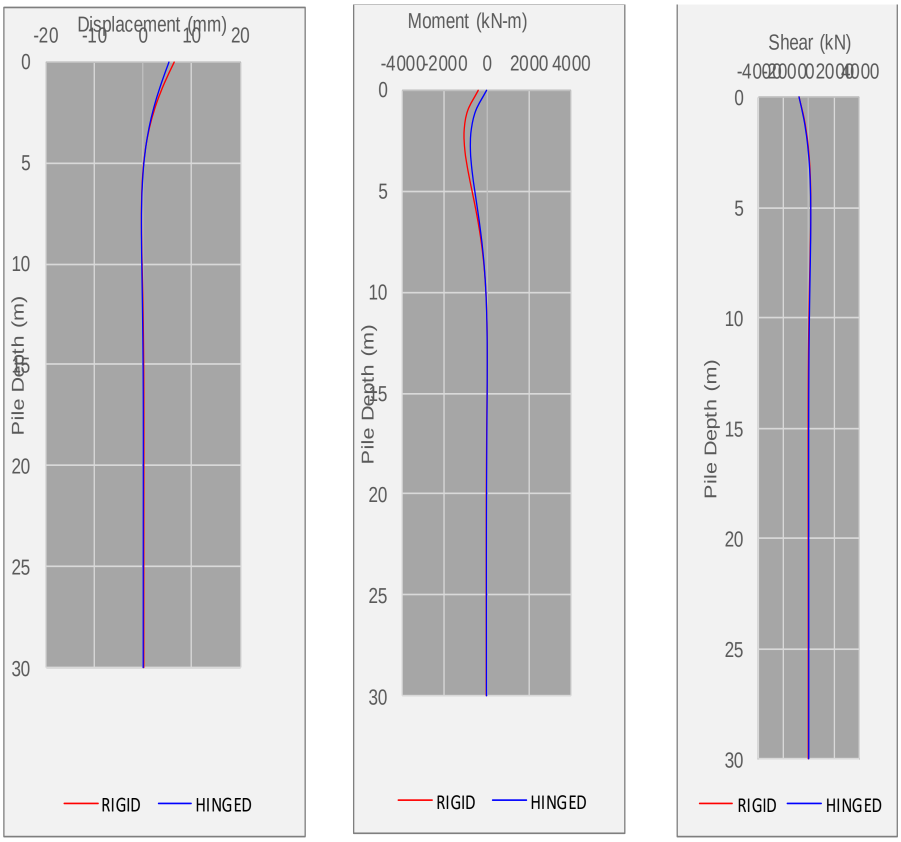


Illustration for Pile Forces :

Commentary



Graphing of Reaction Force and Displacement of each pile

6.7.4 Verification of pile stability

a. The factored resistance of piles shall be taken as:

BSDS Eq.5.4.3.3-1

$$R_R = \Upsilon(\varnothing R_n - W_s) + W_s - W$$

where:

R_R	factored resitance of pile, kN		
R_n	nominal resistance of pile, kN		
W_s	effective weight of soil replaced by pile, kN		
W	effective weight of pile and soil inside pile, kN		
$\varnothing$	resistance factor for pile under extreme event limit state	0.65	**-BSDS Article 5.4.1(5)**
Υ	modification coefficient depending on nominal bearing resist	1.00	**-BSDS Table 5.4.3.3-1**

BSDS Eq.C5.4.3.3-1

b. The nominal bearing capacity can be obtained from the empirical bearing capacity

$$R_n = q_d A_p + U \Sigma L_i f_i$$

where:

R_n nominal bearing capacity of pile, kN

A_p area of pile tip

q_d nominal end bearing resistance intensity per unit area, kN/m^2

U perimeter of pile

L_i thickness of soil layer considering shaft resistance, m

f_i maximum shaft resistance of soil layer considering pile shaft resistance, kN/m^2

c. The factored axial pull-out resistance of a single pile shall be obtained considering soil conditions and construction methods:

Commentary

BSDS Eq.5.4.3.4-1

$P_R = \emptyset P_n + W$

where:

P_R factored axial pull-out resistance of pile, kN

P_n nominal axial pull-out resistance, kN

W effective weight of pile, kN

$\emptyset$ resistance factor for pile under extreme event limit state 0.5 **-BSDS Article 5.4.1(5)**

d. Estimation of Nominal End Bearing Resistance Intensity (qd)

For Cast-in-place RC Piles : nominal end bearing resistance intensity 5000 kN/m^2

Note: On the basis of the recent results of loading tests on cast-in-place RC piles, the nominal end bearing resistance intensity may take the value of 5,000 kN/m2, when a fully hardened sturdy gravelly ground with an N value of 50 or larger and with a thickness of 5m or greater is selected as supporting layer.

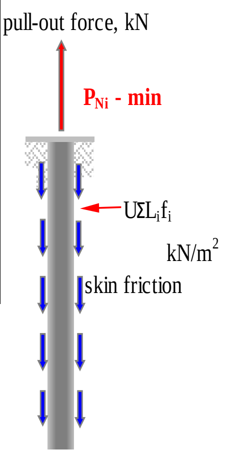
e. Estimation of Shaft Resistance Intensity fi acting on Pile Skin

Cast-in-place RC Piles

For Sandy Soil : 5N (≤ 200)

For Cohesive Soil : c or 10N (≤ 150)

N-th Layer	Soil Layer Type	Layer Thickness L_i	N-Value	g'	L_i*g'*A_p	fi	DE	U*L_i*f_i*DE
		m	Average	kN/m^3	kN	kN/m^2		kN/m^2
1	Clay	1.000	11	8.0	9.05	110	1	414.69
2	Clay	4.000	17	8.0	36.19	150	1	2261.95
3	Clay	4.000	22	8.0	36.19	150	1	2261.95
4	Rock	4.000	50	10.0	45.24	150	1	2261.95
5	Rock			-10.0			1	

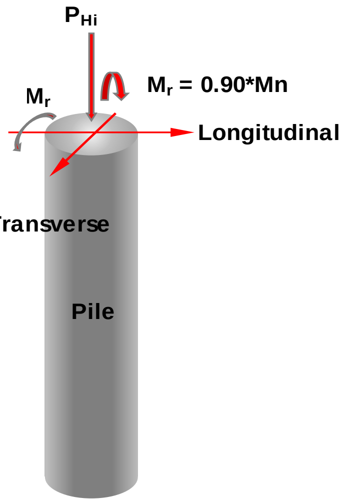


DE is the factor for liquefaction. DE=1 for no liquefaction potential in the specific site. The liquefaction analysis is calculated separately.

Effective weight of soil to be replaced by the pile, Ws	126.67 kN
Effective weight of the pile with soil inside, Wp	205.84 kN
Nominal skin friction of pile	7200.53 kN
Result of Nominal Bearing Capacity of Single Pile, R_n :	12855.40 kN
Result of Factored Resistance of Single Pile, R_R :	8150.17 kN
Result of Factored Axial Pull-out Resistance of Single Pile, P_R :	-3806.10 kN

f. Verification for Lateral Displacement at origin O

Displacement		C/D Ratio	Verification
Demand	Capacity		
mm	mm		
6.26	12	1.92	OK



g. Verification for Maximum Axial Resistance of the Pile Head

Axial Load		C/D Ratio	Verification
Demand	Capacity		
kN	kN		
3862.85	8150.17	2.11	OK

h. Verification for Maximum Axial Pull-out Resistance of the

Axial Pull-out		C/D Ratio	Verification
Demand	Capacity		
kN	kN		
-1525.35	-3806.10	2.50	OK

Commentary

BSDS pp 4-15: "In JRA, the reference displacement at the linear range is recommended to be one percent (1%) of the foundation width (<= 50mm), which is taken as the allowable displacement required from the substructure. However, under earthquake loading this value is taken as a reference and may not necessarily be adhered to and may reach as much as 5% of the foundation width."

6.7.5 Verification of pile structural resistance

Calculation for maximum moment for the design of pile

Maximum moment			
Rigid Pile Head		~~Hinged Pile Head~~	
l_m	M_m	~~l_m~~	~~M_m~~
m	kN-m	~~m~~	~~kN-m~~
2.25	-1074.25	~~2.71~~	~~-806.38~~

- The results under hinge pile head are intentionally crossed out. They are not applicable in this exercise.

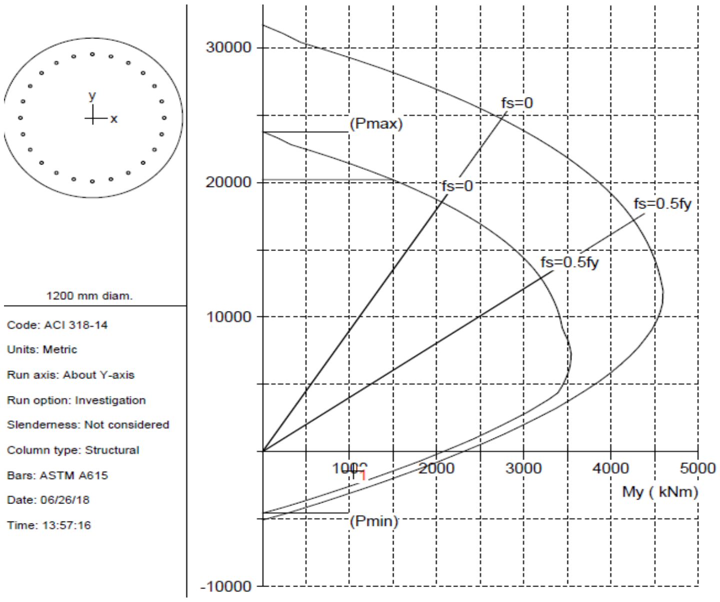
Define Reinforcement of Pile:

Commentary

Diameter of Longitudinal Bars	25	mm
Diameter of Hoops/spirals	16	mm
Reinforcement concrete cover	100	mm

a. Calculation for Pile Flexural Resistance

Maximum axial force that will result for moment capacity on the pile, P_{Hi} :	-1525.35	kN
Flexural resitance factor, Ø :	0.90	factor
Ultimate flexural resistance, M_r = ØM_n : (from inetraction diagram above)	1530.00	kN-m
Mn =	1700	kN-m



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Project: MMPBSIP CD

Column: pier 1_Rd		Engineer:	
f'c = 28 MPa	fy = 415 MPa	Ag = 1.13097e+006 mm^2	24 #8 bars
Ec = 24870 MPa	Es = 200000 MPa	As = 12232 mm^2	rho = 1.08%
fc = 23.8 MPa	e_yt = 0.002075 mm/mm	Xo = 0 mm	Ix = 1.02e+011 mm^4
e_u = 0.003 mm/mm		Yo = 0 mm	Iy = 1.02e+011 mm^4
Beta1 = 0.846954		Min clear spacing = 99 mm	Clear cover = 110 mm

Confinement: Spiral

phi(a) = 0.85, phi(b) = 0.9, phi(c) = 0.75

P-M Interaction Diagram of Pile

b. Calculation for Pile Shear Resistance

The nominal shear resistance, V_n, shall be determined by :

$V_n = V_c + V_s$ **DGCS Equation 12.5.3.2-1**

where :

$V_c = 0.083*\beta*Sqrt(f'_c)*b_v*d_v$ **DGCS Equation 12.5.3.2-3**

$V_s = [\ A_v*f_y*d_v*(cot\theta + cota)sina\ ]\ /\ s$ **DGCS Equation 12.5.3.2-4**

Commentary

b_v	effective web width, mm	1200	mm
d_v	effective shear depth, mm	810.15	mm
s	spacing of transverse reinforcement, mm	60	mm
β	factor indicating ability of diagonally cracked concrete to tra	2.0	factor
θ	angle of inclination of diagonal compressive stresses	45.0	deg
a	angle of inclination of transverse reinforcement	90.0	deg
A_v	area of shear reinforcement within a distance "s"	402.12	mm^2
D	external diameter of the pile, mm	1200	mm
D_r	diameter of pile passing the centers of the longitudinal reinfc	943	mm
Ø	shear resitance factor for normal weight concrete	0.90	factor

Definition of Parameters:

1. Calculation for d_v :

$d_v = 0.9*d_e$

2. Number of shear area within a distance "s" N = 2 pcs

3.Required spacing of transverse reinforcement = S = 60.00 mm

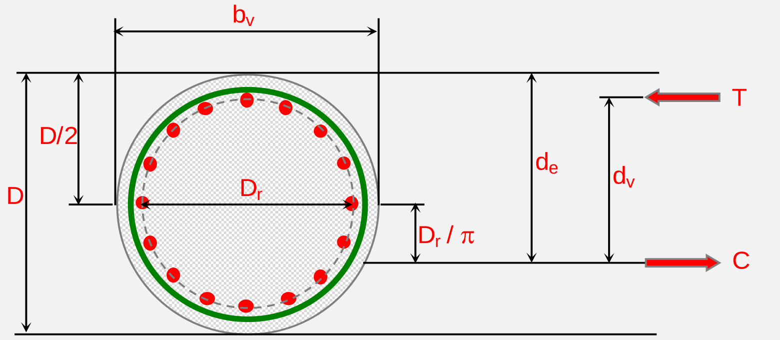


Illustration of Terms b_v, d_v and d_e for Cicular Sections

Shear strength provided by the concrete, V_c :	853.95	kN
Shear strength provided by the reinforcements. V_s :	2253.32	kN
Nominal shear resistance, V_n :	3107.27	kN
Ultimate shear resistance, $V_r = ØV_c$:	2796.54	kN

Verification for Single Pile

Flexural Resistance		C/D Ratio	Verification
Demand	Capacity		
kN-m	kN-m		
1074.25	1530.00	1.42	OK

Shear Resistance		C/D Ratio	Verification
Demand	Capacity		
kN	kN		
725.00	2796.54	3.86	OK

Commentary

c. Verification of minimum required longitudinal reinforcement

The longitudinal reinforcement shall be verified according to the following:

where :

A_a total area of longitudinal reinforcement, mm^2

A_s cross-sectional area of single longitudinal reinforcement, mm^2

A_g gross area of pile, mm^2

A. The longitudinal reinforcement shall not be less than 0.01

$$\rho_s = A_a / A_g > 0.01$$

B. The longitudinal reinforcement shall be more than 0.04 times the gross section area

$$\rho_s = A_a / A_g < 0.04$$

Pile Diameter	Bar Diameter	Number of Bars	A_s	A_a	A_g	ρ_s	Verification		
m	m	No.	m^2	m^2	m^2	ratio		A	B
			per bar	total bars	pile area			ρ_s => 0.01	ρ_s < 0.04
1.20	0.025	24	0.00049	0.01178	1.1310	0.0104		OK	OK

•DGCS 12.7.11

d. Verification of minimum required transverse reinforcement

The ratio of spiral reinforcement to total volume of concrete core masured out-to-out of spirals shall be :

A. The greater of :

$\rho_{s1} = 0.12*(f'_c/f_y)$ and $\rho_{s2} = 0.45*[(A_g/A_c) - 1] *(f'_c/f_y)$ **(for circular shape only)**

where :

A_g gross area of pile, mm^2

A_c area of core measured to the outside diameter of the spiral, mm^2

B. Checking from provided confinement, where As represent spiral leg on one(1) side

$$\rho_{s3} = \frac{4*A_s}{D_r*s}$$

where :

A_s area of shear reinforcement, mm^2 representing spiral leg on one(1) side .

D_r diameter of pile passing the centers of the longitudinal reinforcement, mm

A_c area of core

Commentary

Pile Diameter	A_g	A_c	A_s	ρ_{s2}	$\rho s1 = 0.12*(f'_c/f_y)$	@max ρ_{s1}, ρ_{s2})	ρ_{s3}	Verification
m	m^2	m^2	m^2	ratio	ratio	max.	ρ_s provided	
			Spiral leg on one(1)					
1.20	1.1310	0.79	0.000201	0.01338	0.00810	0.01338	0.01421	**OK**

c/d = 1.06

e. Verification of spacing of spirals at critical section

Maximum spacing

1. S should not be greater than 1/4 min dimension of member (=D/4)
2. S should not be greater than 100mm

Minimum clear spacing

1. Sc should not be less than 25mm
2. Sc should not be less than 1.33 x aggregate size (1.33 x 25mm)

Summary of design of Pile:

Number of Piles	8	
Length, m	13	
Diameter, mm	1200	
no. of Reinforcemnt and sizes	24	25
spacing and size Spiral Reinf.	60	16

Summary of reactions for pile cap design:

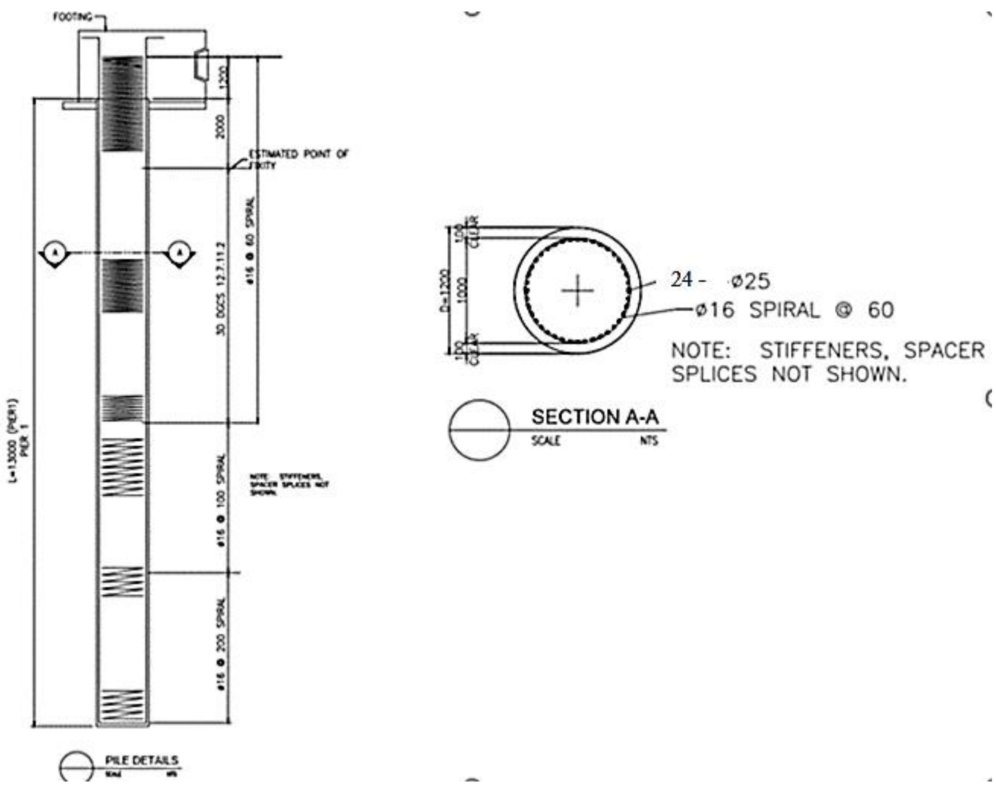
From Pmin :

Pile reactions :			
Row 1 = F1 =	-1525.35	kN	DESIGN REACTION FOR PILECAP
Row 2 = F2 =	1168.75	kN	
Row 3 = F3 =	3862.85	kN	

From Pmax :

Pile reactions :			
Row 1 = F1 =	-794.10	kN	
Row 2 = F2 =	1900.00	kN	DESIGN REACTION FOR PILECAP
Row 3 = F3 =	4594.10	kN	DESIGN REACTION FOR PILECAP

6.7.6 Pile Details



Commentary

•Typical details of piles. Miscellaneous details not shown.

CHAPTER 7: SEISMIC DESIGN OF ABUTMENT

CHAPTER 7 Seismic Design of Abutment

7.1 Flowchart

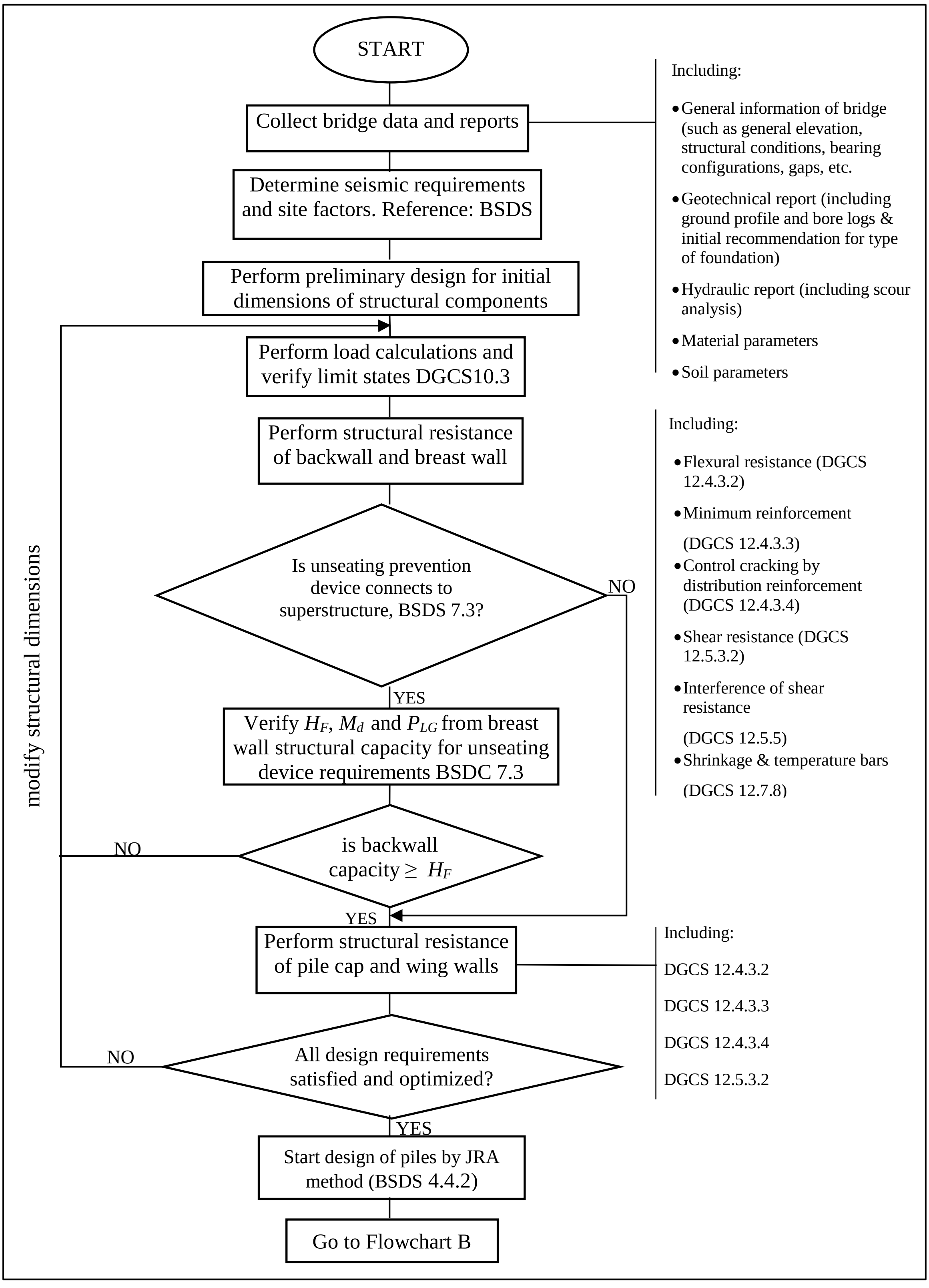


Figure 7.1-1 Flow Chart

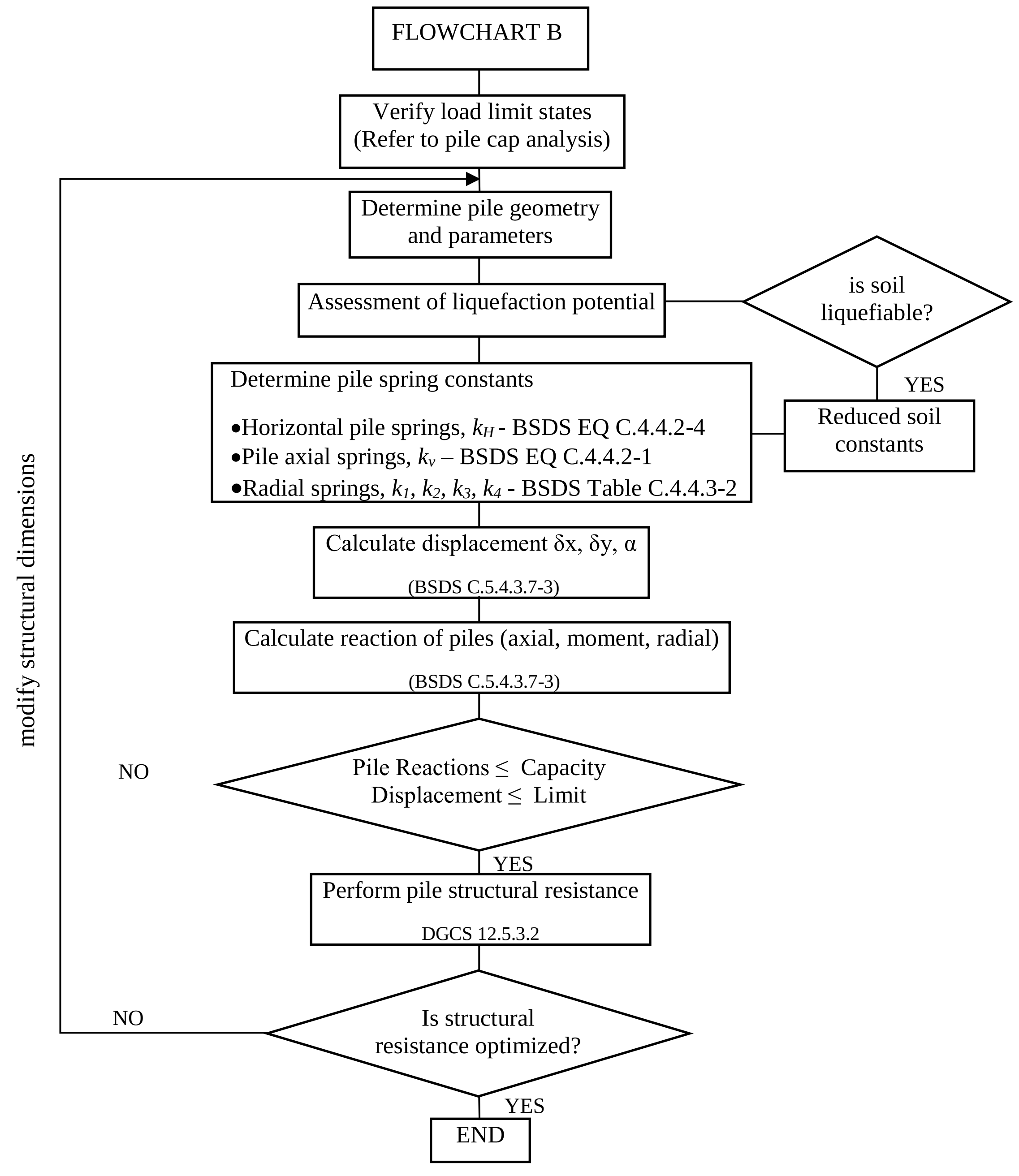


Figure 7.1-2 Flow Chart (B)

7.2 General Design Conditions & Criteria

7.2.1 Bridge General Elevation & Location Map

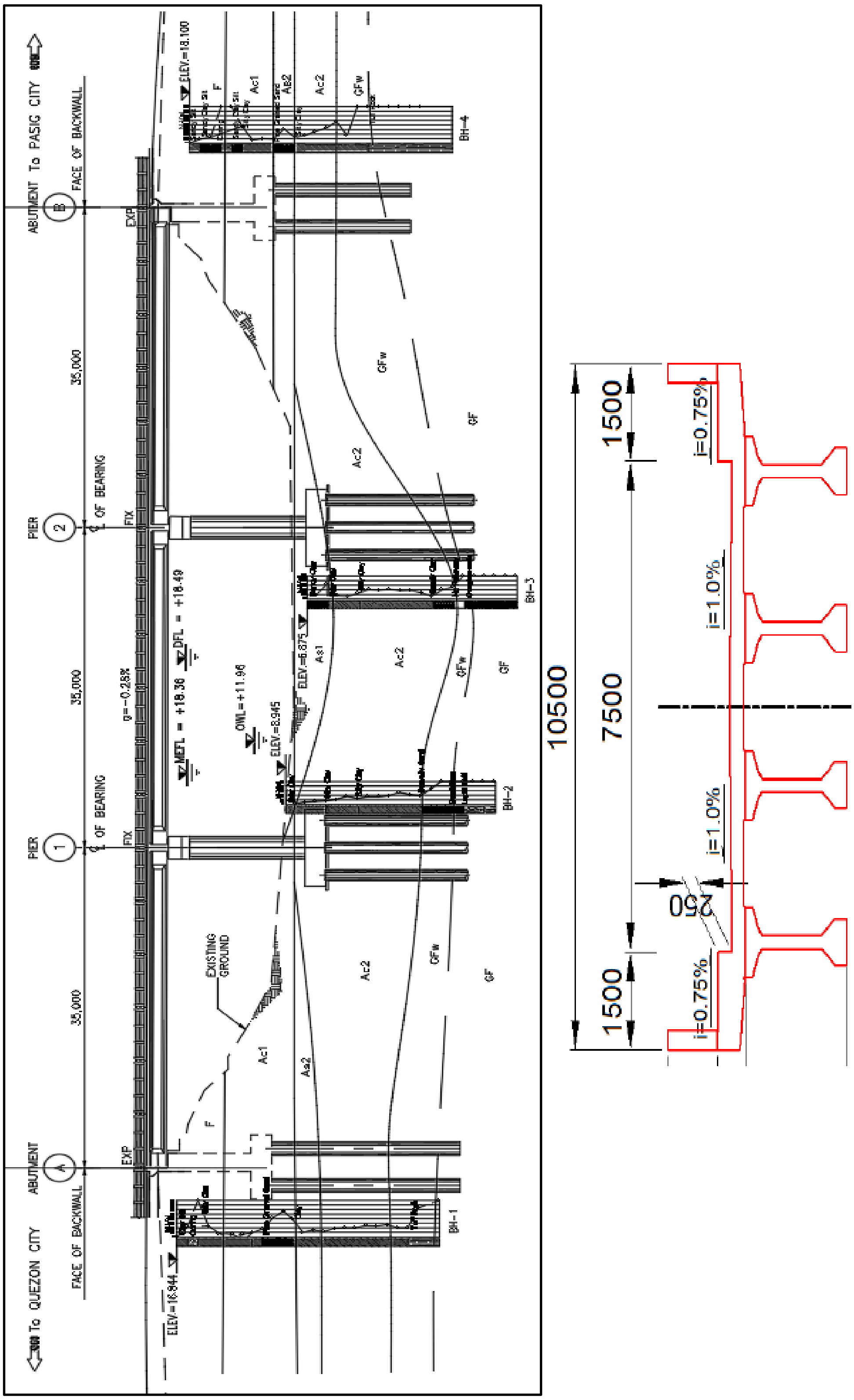


Figure 7.2-1 Bridge General Elevation

7.2.2 Structural Conditions

- Two lane carriageways; total width = 10.50m
- 3 – 35m continuous AASHTO girders- Type V
- Bearings restraints: M-F-F-M (F=fixed, M=movable)
- Regular bridge (non - skewed bridge)
- Pier type : single column on cast in place concrete pile
- Abutment type : cantilever type on cast in place concrete pile

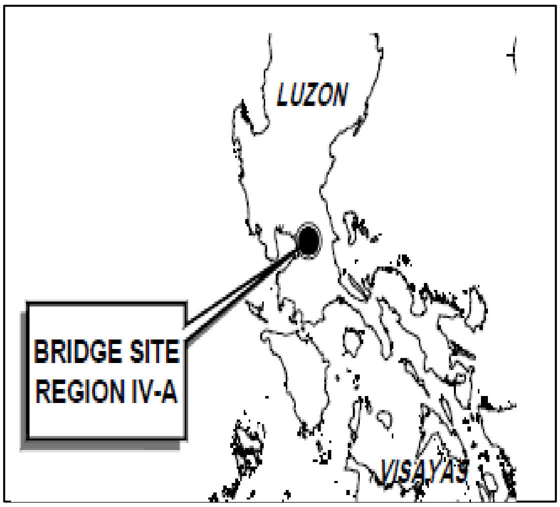


Figure 7.2-2 Location Map

7.2.3 Seismic Design Requirements and Ground Conditions

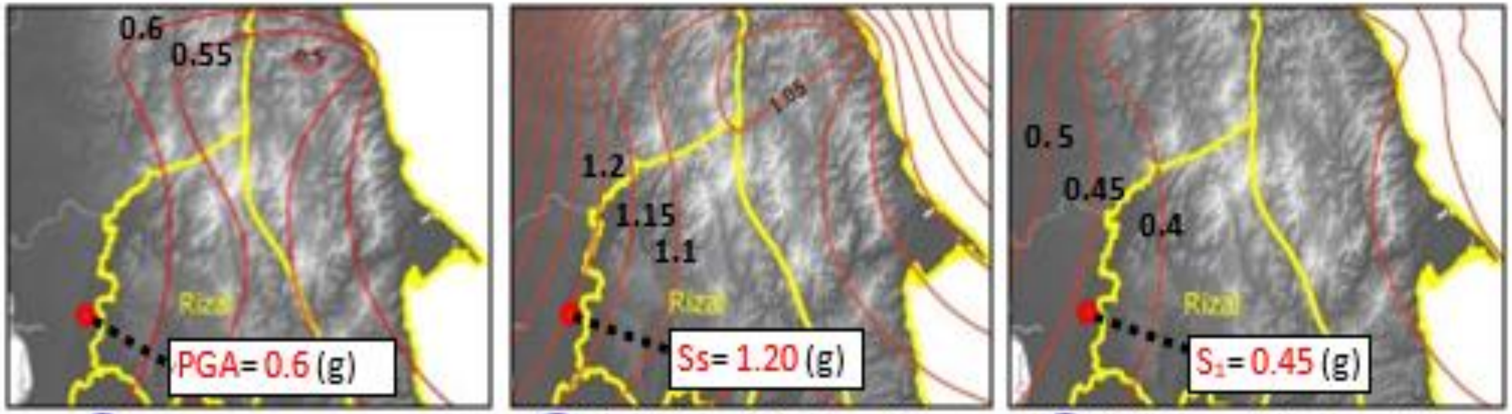
Table 7.2-1 Seismic Design Requirements and Ground Conditions

Bridge Operational Classification =	OTHERS
Earthquake Ground Motion =	Level 2
Ground Type =	3
Seismic Performance Level =	3
Seismic Performance Zone =	4
Peak Ground Acceleration =	0.6g

7.2.4 Site Factors

Site Factors:	
Fpga =	0.88
Fa =	0.92
Fv =	1.55
As =	0.53

1000 YEAR RETURN PERIOD (LEVEL 2 EQ)

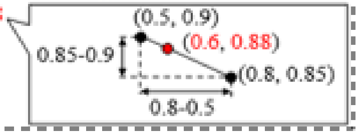


① PGA Contour Map ② Ss (0.2 sec.) Contour Map ③ S_1 (1.0 sec.) Contour Map

① Fpga (site factor for PGA)

PGA (T=0)		PGA< 0.10	PGA= 0.20	PGA= 0.30	PGA= 0.40	PGA= 0.50	PGA≥ 0.80
Soil type	I	1.2	1.2	1.1	1.0	1.0	1.0
	II	1.6	1.4	1.2	1.0	0.9	0.85
	III	2.5	1.7	1.2	0.9	0.8	0.75

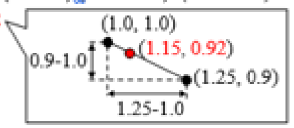
[Linear interpolation of Fpga]
F_{pga}= 0.9+(0.85-0.9)/(0.80-0.50)*(0.60-0.50)
= 0.88



② Fa (site factor for Ss)

Ss (T=0.2)		Ss< 0.25	Ss= 0.50	Ss= 0.75	Ss= 1.00	Ss= 1.25	Ss≥ 2.0
Soil type	I	1.2	1.2	1.1	1.0	1.0	1.0
	II	1.6	1.4	1.2	1.0	0.9	0.85
	III	2.5	1.7	1.2	0.9	0.8	0.75

[Linear interpolation of Fa]
Fa= 1.0+(0.9-1.0)/(1.25-1.00)*(1.20-1.00)
= 0.92



③ Fv (site factor for S_1)

S_1 (T=1.0)		S_1< 0.10	S_1= 0.20	S_1= 0.30	S_1= 0.40	S_1= 0.50	S_1≥ 0.80
Soil type	I	1.7	1.6	1.5	1.4	1.4	1.4
	II	2.4	2.0	1.8	1.6	1.5	1.5
	III	3.5	3.2	2.8	2.4	2.4	2.0

[Linear interpolation of Fv]
Fv= 1.6+(1.6-1.5)/(0.4-0.5)*(0.45-0.4)
= 1.55

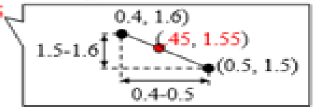


Figure 7.2-3 Site Factors

7.2.5 Borelogs (not to scale)

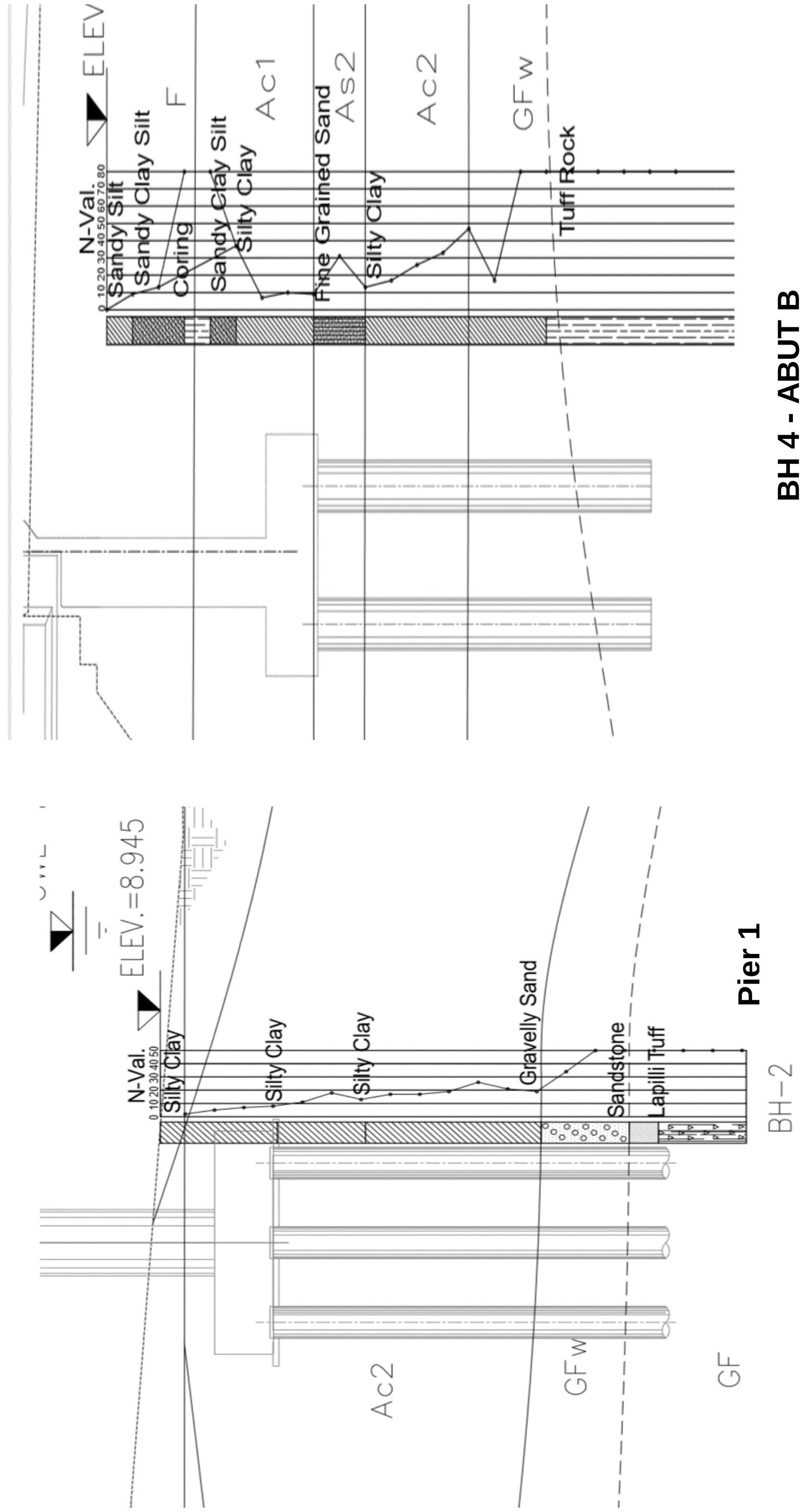


Figure 7.2-4 Borelogs of Pier and Abut B

Note: BHs of Pier 1 and Abut B will be the data to use for the design of Pier 1 and Abut B.

7.2.6 Hydrology and Hydraulics Data

100 years Return Period

Discharge : 3100 m^3/s

Water Level, DFL : 18.50m

Velocity: 4.12m/s

Freeboard : 0.0 m (no consideration)

Drainage Area |: 2,360 sqm

Computed scour depth : 6.67m

7.2.7 Design Loads

1. Permanent Loads	
DC =	Dead load pertaining to structural and non-structural components
DW =	Dead load pertaining to future wearing surface
EH =	Horizontal earth pressure
ES =	Earth surcharge
EV=	Vertical pressure from earth fill

2. Transient Loads	
EQ =	Earthquake Load
LL/IM =	Vehicular Load/Impact load
LS =	Liveload surcharge
WA =	Water Load
FR =	Friction Load
BF=	Braking Force Load

For Seismic load analysis: Refer to BSDS

Load Combinations and factors: Refer to DGCS 10.0

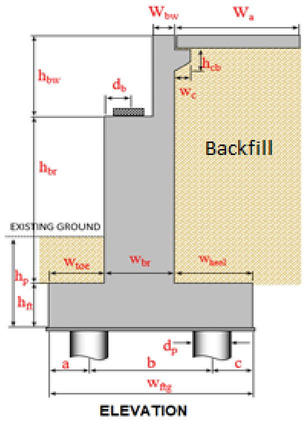
7.2.8 Material and Soil Property

Conc. compressive strength @ 28days, f'_c	28	MPa
Reinforcing steel (ASTM 615), f_y	415	MPa
Unit weight of concrete, δ_c	24	kN/m^3
Unit weight of soil , γ_{soil}	19	kN/m^3

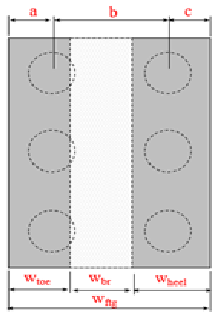
Angle of internal friction of soil for granular soil, Ø			
Estimated value Ø between,30^0-35^0		30	deg
Angle of friction between soil and wall (JRA, 2002), δ			
JRA Table C.2.2.5	seismic =	0	deg
	static =	10	deg.

7.3 Geometry and Load Calculations

7.3.1 Geometry of Abutment



ELEVATION



PILECAP PLAN

Railings

height (h_{rl}) =	1.00 m
thickness (t_{rl}) =	0.30 m
length (L_{rl}) =	4.00 m

Sidewalk

thickness (t_{sw}) =	0.30 m
width (w_{sw}) =	1.20 m
length (L_{sw}) =	4.00 m

Approach Slab

height (h_{ap}) =	0.40 m
width (w_{ap}) =	4.0 m
length (L_{ap}) =	10.5 m

Corbel

height (h_{cb}) =	0.50 m
width (w_{cb}) =	0.35 m
length (L_{cb}) =	10.50 m

Backwall

height (h_{bw}) =	2.50 m
width (w_{bw}) =	0.52 m
length (L_{bw}) =	10.50 m

Breast wall

height (h_{br}) =	7.50 m
width (w_{br}) =	2.00 m
length (L_{br}) =	10.50 m

Footing

height (h_{ftg}) =	2.00 m
width (w_{ftg}) =	7.00 m
width (w_{toe}) =	2.00 m
width (w_{heel}) =	3.00 m
length (L_{ftg}) =	10.50 m

Wingwall

height (h_{ww}) =	9.70 m
thickness (t_{ww}) =	0.70 m
width (w_{ww}) =	3.00 m

Bearing

d_b =	0.70 m

Piles size and location

d_p =	1.20 m
a =	1.50 m
b =	4.00 m
c =	1.50 m
No. of piles, (N) =	6

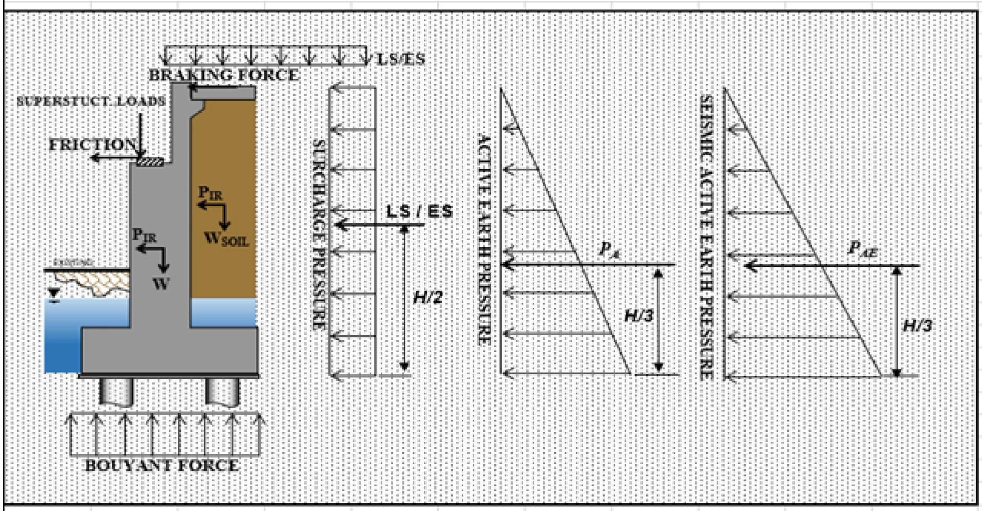
Height of soil or backfill

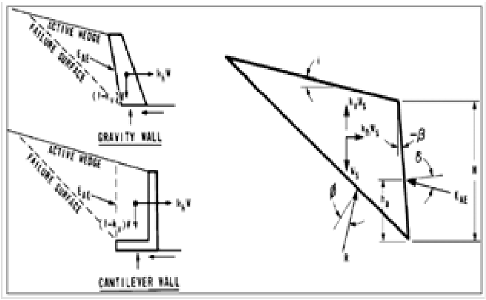
Active Soil (h_a) =	12.00 m
Passive Soil (h_p) =	3.00 m

Commentary

- All sketches and figures are not to scale.
- Dimensions of railings, sidewalks, approach slab, corbel are of typical sizes. Structural design of these items are excluded.
- Dimensions of structural components were set up according to initial preliminary design analysis.
- Height of backfill is assumed equal to total height of abutment to simplify calculations.

7.3.2 Diagram of forces acting to Abutment





Mononabe-Okabe Method to Consider Cohesion

Slope of wall to vertical, β	0	deg.
Slope angle of backfill, i	0	deg.
Wall slope to horizontal, α	90	deg.
Backslope angle, θ	14.84	deg.

- DGCS 16 & A16
- Applicable forces are discussed in the succeeding sections.
- Abutments and other walls shall be designed to meet overall, external, and internal stability during seismic loading. In this exercise, the abutment is founded on piles hence the external stability is skipped.
- As noted in the diagram the location of seismic force is 1/3 of the height.
- DGCS A16.3.1

$\beta = 0^0$ for non -inclined wall
$i = 0^0$ when backfill is level
$\alpha = 90^0$ for vertical wall

Commentary

a. Horizontal forces from earth pressure

a.1 Active lateral earth pressure $P_A = 1/2\ \gamma_{soil}\ H^2\ K_A$

Where:

$$K_A := \frac{(\sin(\alpha+\phi))^2}{(\sin(\alpha))^2 \cdot (\sin(\alpha-\delta_{static})) \cdot \left(1+\sqrt{\frac{\sin(\phi+\delta_{static}) \cdot \sin(\phi-i)}{\sin(\alpha-\delta_{static}) \cdot \sin(\alpha+i)}}\right)^2}$$

•DGCS 10.15.4.3
•DGCS 10.15.4.1
KA = coeff. of active pressure
KAE = coeff. of seismic pressure

Active lateral earth forces			
	P_A	P_A	P_A
K_A	Abutment (full length)	Backwall (per meter)	Breast wall(per meter)
0.308	4430.80 kN	18.32 kN	293.04 kN

a.2 Seismic lateral earth pressure $P_{AE} = 1/2\ \gamma_{soil}\ H^2\ K_{AE}$

Where:

$$K_{AE} := \frac{(\cos(\phi-\theta-\beta))^2}{\cos(\theta) \cdot (\cos(\beta))^2 \cdot \cos(\delta_{seismic}+\beta+\theta) \left[1+\sqrt{\frac{\sin(\phi+\delta_{seismic}) \cdot \sin(\phi-\theta-i)}{\cos(\delta_{seismic}+\beta+\theta) \cdot \cos((i-\beta))}}\right]^2}$$

•DGCS 16.2.6.2

•DGCS A16.3.1
Mononobe-Okabe analysis

Seismic active earth forces			
	P_{AE}	P_{AE}	P_{AE}
K_{AE}	Abutment	Backwall (per meter)	Breast wall(per meter)
0.533	7655.38 kN	31.64 kN	506.31 kN

b. Horizontal forces from inertial mass of abutment, P_{IR}

•DGCS 16.2.6.1
•Horizontal forces are forces due to seismic loading of wall mass of abutment.

Calculation of seismic acceleration coefficient of wall, k_h

Effective peak ground acc. coefficient, A_s =	0.53	From site specific factor analysis
Seismic horizontal coefficient, k_{ho} = As =	0.53	for other foundations
Seismic horizontal coefficient, k_{ho} = $1.2\ As$	0.636	for walls founded on hard or soft rock
Vertical acceleration coefficient, k_v	0	Assume 0 for vertical acceleration

•Refer to Chapter 5 for analysis of site specific factors for the value of acceleration coeff..,A_s

Note: in this exercise, the abutment is not founded on rock

Therefore, kho = As = 0.53

The final hor'l acceleration coefficient, kh = 1/2 kho= **0.265** for seismic coefficient of wall

•DGCS 16.2.6.1

However, the peak ground acceleration and backslope of angle shall be verified and satisfied:

verify peak ground acceleration: $(1 - k_v)\tan(\phi - i) = 0.577$ $\theta = \operatorname{atan}\left(\frac{k_h}{1 - k_v}\right) = 14.842\cdot°$

$0.577 > k_h$ OK

verify backslope angle: $i + \operatorname{atan}\left(\frac{k_h}{1 - k_v}\right) = 14.842\cdot°$

$\phi > 14.842°$ OK

therefore, $k_h = 0.265$

Horizontal forces due to inertial mass of abutment, P_{IR}

COMPONENT	Weight (kN/m)	kh*Weight (kN/m)	W* (kN)	kh*W*
Railings	96.00	25.44	57.60	15.26
Sidewalk	28.80	7.63	69.12	18.32
Wingwall	698.40	185.08	977.76	259.11
Approach slab	38.40	10.18	403.20	106.85
Backwall	31.20	8.27	327.60	86.81
Corbel	4.20	1.11	44.10	11.69
Breast wall	360.00	95.40	3780.00	1001.70
Footing	336.00	89.04	3528.00	934.92
Soil at heel	57.00	15.11	598.50	158.60

*w = full weight of component

• Summary of hor'l forces due to inertial mass of permanent loads to be applied in the design of structural components.

c. Horizontal forces from live load surcharge pressures, LS

•DGCS 10.15.5.4

A live load shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to the wall height behind the back face of the wall

DGCS Table 10.15.5.4-1

Abutment Height (m)	h_{eq}
1.5	1.2
3.0	0.9
greater than 6.0	0.61

← Equivalent height of soil for traffic perpendicular to wall

Labut = 10.50 m total length of Abutment wall

Habut = 12.0 m total height of Abutment wall

Horizontal pressure due to live load surcharge = LS =	$K_A \gamma_{soil} h_{eq}$	=	3.58	kPa
Live load surcharge acting on abutment = LS* Habut * Labut		=	450.46	kN
Live load surcharge acting on backwall =LS*hbw*1m		=	8.94	kN
Live load surcharge acting on breast wall wall = LS*(h_{bw}+h_{br})*1m		=	35.75	kN

•considering per m-strip
•considering per m-strip

d. Horizontal forces from uniform surcharge pressures, ES

•DGCS 10.15.5.1

Where a uniform surcharge is present, a constant horizontal earth pressure shall be added to the basic earth pressure.

This constant earth pressure may be taken as: $\Delta_p = k_s q_s$

where:

Δ_p = *Constant horizontal earth pressure due to uniform surcharge*
k_s = *Coefficient of earth pressure due to surcharge or* K_A
q_s = *Uniform surcharge applied to the upper surface of the active earth wedge*

•in this example no uniform surcharge is present. However, for conservative approach the effect of approach slab is taken as equivalent earth surcharge

Horizontal pressure due to live load surcharge = ES =	$K_A \delta c\ h_{ap}$	=	2.96	kPa
Live load surcharge acting on abutment = LS* Habut * Labut		=	373.12	kN
Live load surcharge acting on backwall =LS*hbw*1m		=	7.40	kN
Live load surcharge acting on breast wall wall = ES*(h_{bw}+h_{br})*1m		=	29.61	kN

•considering per m-strip

•considering per m-strip

e. Horizontal force due to movement of superstructure from via friction effect of bearing pad, FR

•DGCS 10.17

•Analysis of gravity loads (dead load and liveload) not included. Analysis was carried out separately for gravity loads to determine vertical reactions. The bearings at abutment are expansion/movable. It will result horizontal force, i.e. friction force(FR)from bearing pad friction effects during movement or sliding of superstructure.

Dead load reaction force of the superstructure, DC	2800.00	kN
Dead load of wearing surfaces and utilities, DW	150.00	kN
Live load reaction force, LL	750.00	kN
Coefficient of friction of bearing pad, µf	0.15	
Dynamic load allowance, IM	n/a	

Assumption

SERVICE 1

Load Factor		Type of load	Unfactored Load	Factored	
Maximum	Minimum			max	min
1	n/a	DC	2800	2800.00	n/a
1	n/a	DW	150	150.00	n/a
1	n/a	LL	750	750.00	n/a
			V =	**3700.00**	
			FR = Vx μ_f	**555.00**	

STRENGTH 1

Load Factor, Υ_p		Type of load	Unfactored Load	Factored	
Maximum	Minimum			max	min
1.25	0.9	DC	2800	3500.00	2520.00
1.5	0.65	DW	150	225.00	97.50
1.75		LL	750	1312.50	
			V =	**5037.50**	**2617.50**
			FR = Vx μ_f	**755.63**	**392.63**

For the load factors, refer to:

•DGCS Table 10.3-1
•DGCS Table 10.3-2

EXTREME EVENT 1

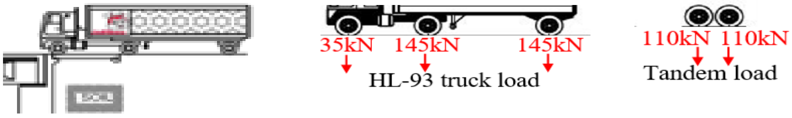
Load Factor, Υ_p		Type of load	Unfactored Load	Factored	
Maximum	Minimum			max	min
1.25	0.9	DC	2800	3500.00	2520.00
1.5	0.65	DW	150	225.00	97.50
0.5		LL	750	375.00	
			V =	**4100.00**	**2617.50**
			FR = Vx μ_f	**615.00**	**392.63**

f. Horizontal force due to braking force at approach slab, BF

The breaking force shall be taken as the greater of:

a. 25% of the axle weights of the design truck or design tandem

b. 5% of the design truck plus lane load or 5% of the design tandem plus lane load



Design lane load	9.34	kN/m
Number of lanes	2	
Length of approach slab	4.0	m
Length of abutment	10.5	m

Tandem:	25% x 2 x (110kN) x 2lanes / Labut =	10.48 kN/m	**Governs!**
Lane:	5% x 2 x (110kN) + lane load x length of slab =	4.61 kN/m	
Truck:	25% x 35kN x 2lanes / L_{abut} =	1.67 kN/m	
Lane:	5% x 35kN + lane load x length of slab =	3.72 kN/m	

BF = 10.48 kN (considering 1m strip)

BF = 110 kN (considering full length)

•DGCS 10.10

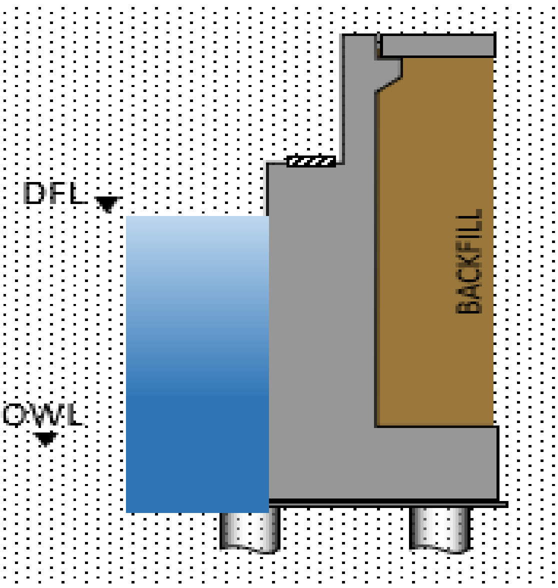
•No braking force effect from superstructure as the bearing is movable or sliding. However, vehicular truck will cause BF to backwall because the approach slab is pinned or doweled connected to corbel.

•By inspection the length of approach slab will accommodate the 2 axles of Tandem, while 1 axle only for the Truck. Hence, Tandem load will produce maximum BF force effect.

g. Uplift force due to buoyant force, WA

Buoyancy shall be considered to be an uplift force, taken as the sum of the vertical components of static pressures acting on all components below the water.

Unit weight of water	9.81	kN/m^3	
Height of ordinary water level	2	m	(from bottom of ftg)
Height of design flood level	6.5	m	(from bottom of ftg)
Height of passive soil, hp	3.00	m	



•DGCS 10.12.2

•The buoyant force is calculated in two cases. Ordinary water level during load combination Extreme event I while DFL during load combination Strength I.

Weight of water during ordinary water level, OWL

WA = weight of water x (volume of submerged including backfill/passive soil)

WA	DIST.
1442.07	3.5

Buoyant force acting on footing, WA 1442.07 kN during OWL

horizontal distance from toe 3.50 m

Weight of water during ordinary water level, DFL

WA = weight of water x (volume of submerged including backfill/passive soil)

WA	DIST.
3965.69	3.50

Buoyant force acting on footing, WA 3965.69 kN during OWL

horizontal distance from toe 3.50 m

•WA is the uplift force from footing for submerged structure.

7.3.3 Load modifiers, factors, and combinations

General load equation:

The total factored force effect shall be taken as:

$$Q = \sum \eta_i \gamma_i Q_i$$

where:

For loads for which a maximum value of is appropriate:

$\eta_i = \eta_D \eta_R \eta_I \geq 0.95$

For loads for which a minimum value of γi is appropriate:

$$\eta_i = \frac{1}{\eta_D \eta_R \eta_I} \leq 1.0$$

where:

γi	=	load factor
η_i	=	load modifier
η_D	=	a factor relating to ductility
η_R	=	a factor relating to redundancy
η_I	=	a factor relating to operational importance
Q_i	=	force effect

Load modifiers:

• Ductility

For strength limit state

for non-ductile components and connections	$\eta D \geq$	1.05
for conventional designs complying with AASHTO	$\eta D =$	1.00
for components and connections	$\eta D \geq$	0.95
For all other limit states	$\eta D \geq$	1.00

Commentary

•DGCS 10.3

•Factor Υi , refer to DGCS Tables 10.3-1 and 10.3-2

•In this exercise, the bridge is classified as typical concrete bridge, non-ductile and conventional level of redundancy

Commentary

- **Redundancy**

For strength limit state

for non-redundant members	$\eta R \geq$	1.05
for conventional levels of redundancy	$\eta R =$	1.00
for exceptional level of redundancy	$\eta R \geq$	0.95
For all other limit states	$\eta R \geq$	1.00

- **Operational Importance**

For strength limit state

for critical and essential bridges	$\eta I \geq$	1.05
for typical bridges	$\eta I =$	1.00
for relatively less important bridges	$\eta I \geq$	0.95
For all other limit states	$\eta I \geq$	1.00

Load factors:

•DGCS Table 10.3-1

Table 10.3-1 Load Combination and Load Factors

Load Combination / Limit State	DC DD DW EH EV ES EL PS CR SH	LL IM CE BR PL LS	WA	WS	FR	TU	TG	SE	EQ	Use one of these at a time: BL	CT	CV
STRENGTH-I (Unless noted)	γ_p	1.75	1.00	-	1.00	0.50/1.20	0.0	γ_{SE}	-	-	-	-
STRENGTH-II	γ_p	1.35	1.00	-	1.00	0.50/1.20	0.0	γ_{SE}	-	-	-	-
STRENGTH-III	γ_p	1.35	1.00	-	1.00	0.50/1.20	0.0	γ_{SE}	-	-	-	-
STRENGTH-III	γ_p	-	1.00	1.4	1.00	0.50/1.20	0.0	γ_{SE}	-	-	-	-
STRENGTH-IV	γ_p	-	1.00	-	1.00	0.50/1.20	-	-	-	-	-	-
EH, EV,ES,DW,	1.5											
DC ONLY												
STRENGTH-V	γ_p	1.35	1.00	0.40	1.00	0.50/1.20	0.0	γ_{SE}	-	-	-	-
EXTREME EVENT - I	γ_p	γ_{EQ}	1.00	-	1.00	-	-	-	1.00	-	-	-
EVENT - I												
EXTREME EVENT - II	γ_p	0.5	1.00	-	1.00	-	-	-	-	1.00	1.00	1.00
EVENT - II												
SERVICE - I	1.00	1.00	1.00	0.30	1.00	1.00/1.20	0.0	γ_{SE}	-	-	-	-
SERVICE - II	1.00	1.3	1.00	-	1.00	1.00/1.20	-	-	-		-	-
SERVICE - III	1.00	0.8	1.00	-	1.00	1.00/1.20	0.0	γ_{SE}	-	-	-	-
SERVICE - IV	1.00	-	1.00	0.70	1.00	1.00/1.20	-	1.0	-	-	-	-
FATIGUE – 1 LL, IM, & CE ONLY	-	1.50	-	-	-	-	-	-	-	-	-	-
FATIGUE – II LL, IM, & CE ONLY	-	0.75	-	-	-	-	-	-	-	-	-	-

Table 10.3-2 Load Factors for Permanent Loads, γ_p

Type of Load	Load Factor: Max	Min
DC: Component and Attachments	1.25	0.90
DD: Downdrag	1.80	0.45
DW: Wearing Surfaces and Utilities	1.50	0.65
EH: Horizontal Earth Pressure		
Active	1.50	0.90
At-Rest	1.35	0.90
EL: Locked-in Erection Stresses	1.00	1.00
EV: Vertical Earth Pressure		
Retaining Walls and Abutments	1.35	1.00
Rigid Buried Structure	1.30	0.90
Rigid Frames	1.35	0.90
Flexible Buried Structures other than Metal Box Culverts	1.95	0.90
Flexible Metal Box Culverts	1.50	0.90
ES: Earth Surcharge	1.50	0.75

•The applicable basic load combinations for this exercise are Strength I, Extreme Event I and Service I.

7.4 DESIGN OF BACKWALL

7.4.1 Determine the applicable loads calculated from section 7.3 Geometry and Load Calculations

1. Permanent loads

			Commentary
Dead load from the superstructure, DC 1	N/A		•loads per meter strip
Dead load from self weight, DC 2			
a) Weight of backwall, DC 2.1	31.20	kN	
b) Weight of corbel, DC 2.2	4.20	kN	
c) Weight of approach slab, DC 2.3	38.40	kN	
Deadload of future wearing surface and utilities, DW	N/A		• Future wearing surface is considered N/A over top of approach slab. However in actual practice, wearing surface maybe applied immediately in the approch slab .
Horizontal earth pressure load, EH (=P_A)	18.32	kN	
Earth surcharge load, ES	7.40	kN	

2. Braking force , BF — 10.48 kN

3. Earthquake force, EQ

Seismic active earth force, P_{AE}	31.64	kN
Seismic inertial force, P_{IR}		
a) kh * Backwall	8.27	kN
b) kh *Corbe	1.11	kN
c) kh * Approach slab	10.18	kN
d) kh * Soil	37.76	kN

4.	Vehicular live load	N/A	
5.	Live load surcharge, LS	8.94	kN
6.	Friction load, FR	N/A	
7.	Water load and stream pressure, WA	N/A	

• Vehicular live load not applicable, however the effect of LL surcharge and BF are applied.

7.4.2 Determine the load combinations with applied load modifiers and load factors

a. Load Combination: STRENGTH 1

Load modifier for maximum values, ηi 1.05
Load modifier for minimum values, ηi 0.95

Load Factor max	Load Factor min	Load Type	Unfactored load, kN Hor'l	Unfactored load, kN Vert'l	Lever arm (m)	Factored Axial force, kN max	Axial force, kN min	Shear force, kN max	Shear force, kN min	Moment, kN-m max	Moment, kN-m min
1.25	0.90	DC 2.1	0	31.20	0.0	39.00	28.08	0	0	0	0
1.25	0.90	DC 2.2	0	4.20	-0.40	5.25	3.78	0	0	-2.10	-1.51
1.25	0.90	DC 2.3	0	38.40	-0.40	48.00	34.56	0	0	-19.20	-13.82
1.50	0.90	EH	18.32	0	0.83	0	0	27.47	16.48	22.80	13.68
1.50	0.75	ES	7.40	0	1.25	0	0	11.10	5.55	13.88	6.94
1.75		BF	10.48	0	2.50	0	0	18.33	0	45.83	0
1.75		LS	8.94	0	1.25	0	0	15.64	0	19.55	0
					(Strength I) Design load:	96.86	63.10	76.18	20.93	84.81	5.02

•Modifier, ηi
$\eta_D \geq$ 1.05
$\eta_R \geq$ 1.00
$\eta_i \geq$ 1.00
for max. values
η_i = 1/(1.05x1.0x1.0)
η_i = 1.05
for min. values
η_i = 1/(1.05x1.0x1.0)
η_i = 0.95

b. Load Combination: EXTREME EVENT I

Commentary

•DGCS 16.2.6

Note: The lateral force to be applied to the wall due to seismic and earth pressure loading should be determined considering the combined effects of PAE and PIR considering and them not to be concurrent. Two cases shoud be investigated:

- CASE 1: Combined 100% of $P_{\Delta E}$ plus 50% of P_{IR}.
- CASE 2: Combined 50% of $P_{\Delta E}$ but no less than static active earth pressure with 100% of P_{IR}.

The most conservative result of the two cases shall be used in the design of the backwall.

•Modifier, η_i

η_i = 1.00

b1. Load Combination: EXTREME EVENT I (CASE 1) (Case1: 100% $P_{\Delta E}$ + 50% P_{IR})

Load Factor		Load Type	Unfactored load, kN		Lever arm (m)	Factored					
						Axial force, kN		Shear force, kN		Moment, kN-m	
max	min		Hor'l	Vert'l		max	min	max	min	max	min
1.25	0.90	DC 2.1	0	31.20	0.0	39.00	28.08	0	0	0	0
1.25	0.90	DC 2.2	0	4.20	-0.40	5.25	3.78	0	0	-2.10	-1.51
1.25	0.90	DC 2.3	0	38.40	-0.40	48.00	34.56	0	0	-19.20	-13.82
1.50	0.75	ES	7.40	0	1.25	0	0	11.10	5.55	13.88	6.94
0.50		BF	10.48	0	2.50	0	0	5.24	0	13.10	0
0.50		LS	8.94	0	1.25	0	0	4.47	0	5.59	0
1.00		50%P_{IR}_a	4.13	0	1.25	0	0	4.13	0	5.17	0
1.00		50%P_{IR}_b	0.56	0	1.85	0	0	0.56	0	1.03	0
1.00		50%P_{IR}_c	5.09	0	2.30	0	0	5.09	0	11.70	0
1.00		50%P_{IR}_d	18.88	0	1.25	0	0	18.88	0	23.60	0
1.00		P_{AE}	31.64	0	0.83	0	0	31.64	0	26.26	0
(Extreme Event 1-Case1) Design load:						**92.25**	**66.42**	**81.12**	**5.55**	**79.03**	**-8.40**

b2. Load Combination: EXTREME EVENT I (CASE 2) (Case2: 50% $P_{\Delta E}$ + 100% P_{IR})

•Modifier, η_i

η_i = 1.00

However, if 50% P_{AE} < P_A, use P_A, else use 50%P_{AE}

Verification:

50% P_{AE} = 15.82

P_A =EH = **18.32** > 50% P_{AE}

therefore use EH

Load Factor		Load Type	Unfactored load, kN		Lever arm (m)	Factored					
						Axial force, kN		Shear force, kN		Moment, kN-m	
max	min		Hor'l	Vert'l		max	min	max	min	max	min
1.25	0.90	DC 2.1	0	31.20	0	39.00	28.08	0	0	0	0
1.25	0.90	DC 2.2	0	4.20	-0.40	5.25	3.78	0	0	-2.10	-1.51
1.25	0.90	DC 2.3	0	38.40	-0.40	48.00	34.56	0	0	-19.20	-13.82
1.50	0.75	ES	7.40	0	1.25	0	0	11.10	5.55	13.88	6.94
0.50		BF	10.48	0	2.50	0	0	5.24	0	13.10	0
0.50		LS	8.94	0	1.25	0	0	4.47	0	5.59	0
1.00		P_{IR}_a	8.27	0	1.25	0	0	8.27	0	10.34	0
1.00		P_{IR}_b	1.11	0	1.85	0	0	1.11	0	2.06	0
1.00		P_{IR}_c	10.18	0	2.30	0	0	10.18	0	23.40	0
1.00		P_{IR}_d	37.76	0	1.25	0	0	37.76	0	47.20	0
1.00		EH	18.32	0	0.83	0	0	18.32	0	15.20	0
(Extreme Event 1-Case 2) Design load:						**92.25**	**66.42**	**96.45**	**5.55**	**109.47**	**-8.40**

c. Load Combination : SERVICE 1

Load Factor		Load Type	Unfactored load, kN		Lever arm (m)	Factored					
						Axial force, kN		Shear force, kN		Moment, kN-m	
max	min		Hor'l	Vert'l		max	min	max	min	max	min
1.00		DC 2.1	0	31.20	0.0	31.20	0	0	0	0	0
1.00		DC 2.2	0	4.20	-0.40	4.20	0	0	0	-1.68	0
1.00		DC 2.3	0	38.40	-0.40	38.40	0	0	0	-15.36	0
1.00		EH	18.32	0	0.83	0	0	18.32	0	15.20	0
1.00		ES	7.40	0	1.25	0	0	7.40	0	9.25	0
1.00		BF	10.48	0	2.50	0	0	10.48	0	26.19	0
1.00		LS	8.94	0	1.25	0	0	8.94	0	11.17	0
					(Service 1) Design load:	**73.80**	**0.00**	**45.13**	**0.00**	**44.78**	**0**

Commentary

•Modifier, η_i

η_i = 1.00

b3. Summary of Load Combinations:

STRENGTH I						EXTREME EVENT 1 (CASE 1)					
Axial force (kN)		Shear Force (kN)		Moment (kN-m)		Axial force (kN)		Shear Force (kN)		Moment (kN-m)	
max	min	max	min	max	min	max	min	max	min	max	min
96.86	**63.10**	**76.18**	**20.93**	**84.81**	**5.02**	**92.25**	**66.42**	**81.12**	**5.55**	**79.03**	**-8.40**

EXTREME EVENT 1 (CASE 2)						SERVICE 1					
Axial force (kN)		Shear Force (kN)		Moment (kN-m)		Axial force (kN)		Shear Force (kN)		Moment (kN-m)	
max	min	max	min	max	min	max	min	max	min	max	min
92.25	**66.42**	**96.45**	**5.55**	**109.47**	**-8.40**	**73.80**	**0.00**	**45.13**	**0.00**	**44.78**	**0.00**

7.4.3 Determine the governing design forces:

Load Combination: EXTREME EVENT I (CASE 2)

Axial force = **92.25** kN

Shear Force = **96.45** kN

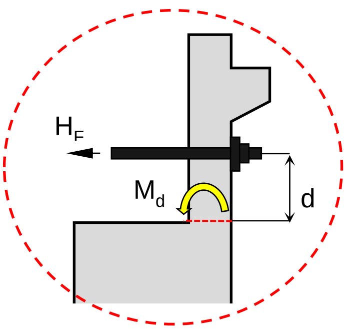
Moment = **109.47** kN-m

Note: The governing design forces shall be verified from : Verification of demand forces for unseating prevention device from Design of Breast Wall.

The design forces required for the unseating prevention device as determined from Breast wall design are as follows:

H_F = Vd = **400.00** kN

Md = **240.00** kN-m



•The summary of load combinations show that the Extreme Event 1 (Case 2) is the critical load case. However, BSDS section 7.3 shall be satisfied.

•BSDS 7.3

• It shows the demand forces from unseating prevention device is significantly larger than the design force from lateral loads of backwall. Therefore, the backwall shall be designed from the demand forces from unseating device.

7.4.4 Verification of flexural resistance

				Commentary
Demand moment, Md	**240.00**	kN-m		
Concrete cover	75	mm		
Diameter of reinforcing bar	25	mm	Ab= 490.87	
Diameter of shrinkage bar	16	mm		
Diameter of cross ties	12	mm		
Effective depth of concrete , de	404.5	mm		• per 1m-width design
Width to be considered, b	1000	mm		
Overall thickness of component, h	520	mm		• DGCS 12.4.3.3

Minimum reinforcement

Flexural cracking variability factor, γ_1	1.6	**for all other concrete*	
Ratio of specified min. to ult. tensile strength of steel, γ_3	0.67	**for A615, 414MPa steel*	
Modulus of rupture, f_r	3.334	MPa	
Section modulus, S_c	4.5E+07	mm^3	
Cracking moment, M_{cr}			
$\cdot M_{cr}=\gamma_3(\gamma_1 \cdot f_r) S_c$	161.05	kN-m	
*Mu_min 1.33*Md =*	319.20	kN-m	
Condition: if Md > (min (Mcr, Mu_min), Md, (min (Mcr,Mu_min))			
Therefore, Design moment for backwall, Md	**240.00**	kN-m	**Governs!!!**

Steel ratio

• DGCS 12.4.2.1

β_1 = Coefficient Criterion: = 0.85

the factor β_1 shall be taken as 0.85 for concrete strengths not exceeding 28MPa. For concrete strength exceeding 28MPa, β_1 shall be reduced at a rate of 0.05 for each 7MPa strength excess of 28MPa but not less than 0.65

For required steel ratio, ρ

m_1 =	0.0573	
m_2 =	0.084	mm^2/N
R_n =	1.630	MPa
ρ =	0.0041	

Computation for main reinforcement

Required steel area, *As*	1647.02	mm^2		
Required spacing, s	298.04	say:	250	mm
Provided steel for backwall, *As*	1963.49	mm^2		
Compression fiber to neutral axis, *c*	40.28	mm		
Depth of compression block, *a*	34.24	mm		
Nominal moment capacity of section, M_n	315.66	kN-m		
Resistance factor, Ø	0.9	tension is controlled		
$\cdot 0.75 \le \phi = 0.65 + 0.15 (d_{eff}/c - 1) \le 0.9$ 2.01				
Ultimate moment capacity of section, $ØM_n$	**284.09**	kN-m	**OK!**	
	c/d=	1.18		

Using of 25mm Ø main bars spaced at 250mm O.C. for backwall is adequate

Commentary

Control of cracking by distribution of reinforcement

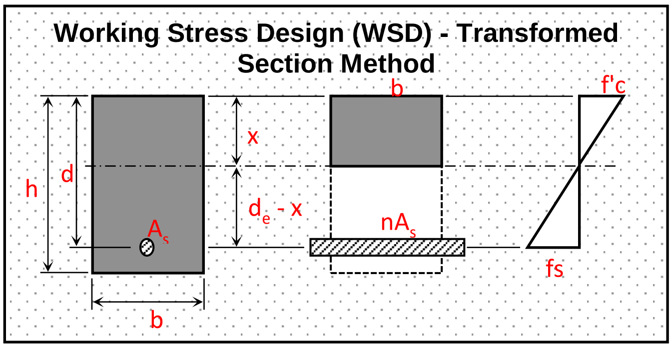
• DGCS 12.4.3.4

Applies to all reinforcements of concrete that exceeds 80% of the modulus of rupture, except deck slabs.

• DGCS 12.1.1.6

Moment demand at Service 1			**44.78**	kN-m
80% of Modulus of rupture, f_r	80% x 0.52√f'c		2.201	MPa
Tensile stress in steel at the service limit, f_{ss}	M_s/S_c	=	0.994	MPa

Tensile stress in steel does not exceed 80% of the modulus of rupture, this provision does not need to be satisfied



Extreme tension fiber to center of flexural reinforcement, d_c	-	mm
Overall thickness of component, h	-	mm
Compression fiber to the centroid of extreme tension steel, d_e	-	mm
Neutral axis to extreme compression fiber, x	-	mm
Modulus elasticity of steel, E_s	-	GPa
Modulus elasticity of concrete, E_c	-	GPa
Modular ratio, n	-	
Cracked section moment of inertia of section, I_{NA}	-	mm^4
Exposure factor, Υ_e	-	
Exposure condition: Class 1		
Tensile stress in steel reinforcement at the service limit, f_s	-	mPa
$\beta s = 1 + d_c/(0.7\,(h - dc))$	-	

• This section is N/A because 80% fr > fss limit.

The spacing shall satisfy: $s \leq 123000\gamma_{e}/\beta_s f_{ss} - 2\mathrm{dc}$

Initial spacing: - mm

7.4.5 Verification of shear resistance

• DGCS 12.5.3.2

Effective shear depth, d_v		387.38 mm
Taken as the distance measured perpendicular to the neutral axis, between the resultants of the tensile and compressive forces due to flexure; it need not to be taken to be less than the greater of 0.9de or 0.72h	=0.9 * de	364.05 mm
	=0.72 * h	374.4 mm

			Commentary
Factor indicating ability of diagonally cracked concrete to transmit tension,	2.63	= β	
Solution for β: **GENERAL PROCEDURE**			• General procedure is basically applicable to design of walls, slab and footings with thickness > 400mm
Area of prestressing steel on tension side, A_{ps}	0	mm^2	
Area of non-prestressing steel, A_s	1963.49	mm^2	
Maximum aggregate size, a_g	20	mm	
Modulus of elasticity of prestressing tendons, f_{po}	0	MPa	• DGCS 12.5.3.3.2
Factored axial force, N_u	-92250	N	
**Positive for tension; Negative for compression*			
Factored shear force, V_u	400000.00	N	
Absolute value of the factored moment, $\|M_u\|$	2.4E+08	N-mm	
**But not less than $\|V_u - V_p\| d_v$*			
Modulus of elasticity of prestressing steel, E_p	0	GPa	
Modulus of elasticity of steel, E_s	200	GPa	
Net longitudinal tensile strain, e_s	0.001		
Crack spacing parameter, S_{xe}	360	mm	
Shear resistance from steel, V_s	0	kN	
Effective prestressing force, V_p	0	kN	
Shear resistance provided by concrete, V_c	447.59	kN	
The nominal shear resistance, V_n	447.59	kN	
**shall be determined as the lesser of:*			
$V_n = Vc + Vs + Vp$	447.59	kN	
$V_n = 0.25 fc' \, b_v d_v + Vp$	2711.67	kN	
Vu = Vd =	**400.00**	kN	
Resistance factor for normal weight concrete, Ø	0.9		
Ultimate shear capacity of section, $ØV_n$	**402.83**	kN **OK!**	
	c/d =	1.01	
Section without shear reinforcement is adequate			

7.4.6 Verification of interface shear resistance

Interface shear transfer shall be considered across a given plane at:			•DGCS 12.5.5
a) An existing or potential crack			
b) An interface between dessimilar materials			
c) An interface between two concrete cast at different times			
d) The interface between different elements of the cross-section			
Number of bars provided (for both faces per meter strip), N say	8	pcs	•N= (b/s) x 2sides
Area of shear reinforcement crossing the shear plane, A_{vf}	3926.99	mm^2	•DGCS 12.5.5.3
**Minimum area of shear interface shall satisfy:*			
$A_{vf} \geq (0.35\, A_{cv})/f_y$	242.89	mm^2 **SATISFIED!**	
Interface length considered to be engaged inshear transfer, L_{vi}	1000	mm	
Interface width considered to be engaged inshear transfer, b_{vi}	288	mm	
Area of concrete considered to be engaged in interface shear transfer, A_{cv}	288000	mm^2	
Permanent net compressive force normal to the shear plan, P_c	92250	N	
Factored interface shear force due to total load, V_{ui}	400.00	kN	

Commentary

Cohesion and friction factors

**for concrete placed against a clean concrete surface, free of laitance, with surface intentionally roughened to an amplitude of 6mm:*

•DGCS 12.5.5.2

Cohesion factor, c	1.7	MPa
Friction factor, m	1.00	
Fraction of concrete to resist interface shear, K_1	0.25	
Limiting interface shear resistance, K_2	10.3	MPa

The nominal shear resistance of the interface plane shall be taken as:

$V_{ni}=cAcv+\mu(Avffy+Pc)$ 2211.55 kN

The nominal shear resistance, V_{ni} shall not be greater than the lesser of:

a) $K_1 f^{\wedge\prime} cA_c$ 2016 kN

b) $K_2 A_{cv}$ 2966.40 kN

Nominal shear resistance of the interface plane, V_{ni}	2016	kN	
Resistance factor for normal weight concrete, Ø	0.9		
Factored interface shear resistance of the section, V_{ri}	1814.40	kN	**OK!**

Section is adequate at interface shear transfer

7.4.7 Verification of shrinkage and temperature reinforcement

•DGCS 12.7.8

Reinforcement for shrinkage and temperature stresses shall be provided near surfaces of concrete exposed to daily temperaturechanges and in structural mass concrete.

Diameter of shrinkage and temperature bar	16	mm
Assumed spacing, S	200	mm
Assumed shrinkage and temperature reinforcement, A_s	1.00531	mm^2/mm

Shrinkage and temperature reinforcement shall satisfy:

a) $As \geq (0.75\, bh)/(2\,(b+h) f_y)$ 0.389 1.01 mm^2/mm

b) $0.233 \leq As \leq 1.27$ 1.00531 mm^2/mm

Spacing shall not exceed:

a) 3.0 times the component thickness, or 450 mm

b) 300 mm for walls and footings greater than 450 mm thick

c) 300 mm for other components greather than 900 mm thick

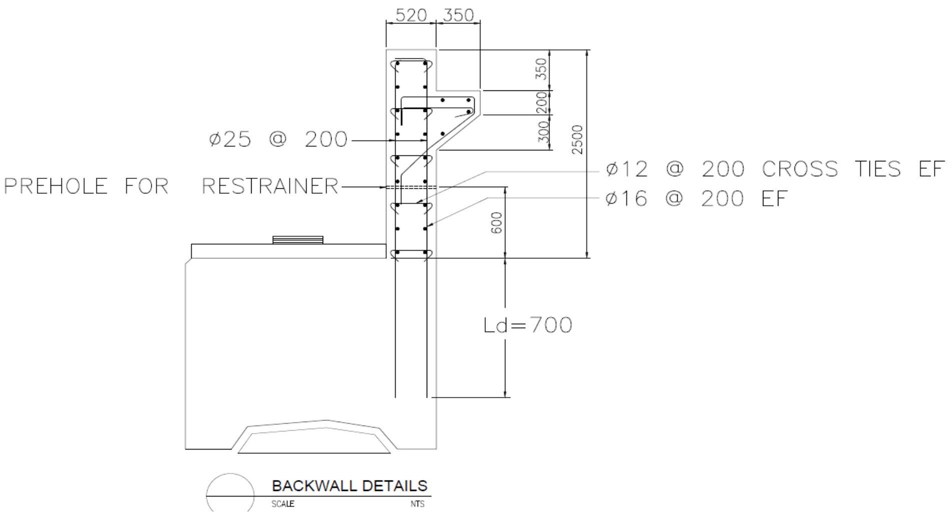
Final shrinkage and temperature reinforcement, A_s	1005.31	mm^2 per meter
Final spacing to be used	200	mm
say:	200	mm

Use 16mm for temperature and shrinkage bar spaced at 200mm O.C. eachface

7.4.8 Development of reinforcement

					Commentary
Diameter of main bars, d_b			25	mm	•DGCS 12.8.2.1
Area of main bars, A_b			490.87	mm^2	
Basic tension development length, l_{db}			769.96	mm	
Minimum development length *(only for d_b 36 mm and lesser)*			622.50	mm	
Modification Factor.					
Modification factor 1	0.8				
Modification factor 2	0.839				
Final development length, l_d	622.50	say:	700	mm	

7.4.9 Backwall details



• Note: the restrainer details and reinforcement not shown.

7.5 DESIGN OF BREAST WALL

7.5.1 Determine the applicable loads calculated from section 7.3 Geometry and Load Calculations

			Commentary
1. Permanent loads			•loads per meter strip
Dead load reaction force of the superstructure, DC 1	266.67	kN	
Dead load from self-weight, DC 2			
a) Weight of backwall, DC 2.1	31.20	kN	
b) Weight of corbel, DC 2.2	4.20	kN	
c) Weight of breast wall, DC 2.3	360.00	kN	
d) Weight of approach slab, DC 2.4	38.40	kN	
Deadload of wearing surfaces and utilities, DW	14.29	kN	
Horizontal earth pressure load, EH (=P_A)	293.04	kN	
Earth surcharge load, ES	29.61	kN	
2. Braking force, BF	10.48	kN	•BF is the breaking from approach slab. BF from superstructure is N/A.
3. Earthquake force, EQ			
Seismic active earth force, P_{AE}	506.31	kN	

Seismic inertial force, P_{IR}

a) kh * Backwall	8.27	kN
b) kh *Corbel	1.11	kN
c) kh * Breast wall	95.40	kN
d) kh * Approach slab	10.18	kN
e) kh * Soil	151.05	kN
4. Vehicular live load	71.43	kN
5. Live load surcharge, LS	35.75	kN
6. Friction load, FR		
a) Strength 1		
- at maximum condition	71.96	kN
- at minimum condition	37.39	kN
b) Extreme event 1		
- at maximum condition	58.57	kN
- at maximum condition	37.39	kN
c) Service 1		
- at maximum condition	52.86	kN
- at maximum condition	0	kN
7. Water load and stream pressure, WA	N/A	kN

Commentary

• Note: FR loads are already factored.

7.5.2 Determine the load combinations with applied load modifiers and load factors.

Load modifier for maximum values, η_i 1.05

Load modifier for minimum values, η_i 0.95

a. Load Combination: STRENGTH 1

Load Factor		Load Type	Unfactored load, kN		Lever arm (m)	Factored					
			Hor'l	Vert'l		Axial force, kN		Shear force, kN		Moment, kN-m	
max	min					max	min	max	min	max	min
1.25	0.90	DC 1	0.00	266.67	0.30	333.33	240.00	0.00	0.00	100.00	72.00
1.25	0.90	DC 2.1	0.00	31.20	-0.74	39.00	28.08	0.00	0.00	-28.86	-20.78
1.25	0.90	DC 2.2	0.00	4.20	-1.18	5.25	3.78	0.00	0.00	-6.20	-4.46
1.25	0.90	DC 2.3	0.00	360.00	0.00	450.00	324.00	0.00	0.00	0.00	0.00
1.25	0.90	DC 2.4	0.00	38.40	-1.18	48.00	34.56	0.00	0.00	-56.64	-40.78
1.50	0.65	DW	0.00	14.29	0.30	21.43	9.29	0.00	0.00	6.43	2.79
1.50	0.90	EH	293.04	0.00	3.33	0.00	0.00	439.56	263.74	1463.75	878.25
1.50	0.75	ES	29.61	0.00	5.00	0.00	0.00	44.42	22.21	222.10	111.05
		FR	0.00	0.00	7.50	0.00	0.00	71.96	37.39	0.00	280.45
1.75		LL	0.00	71.43	0.30	125.00	0.00	0.00	0.00	37.50	0.00
1.75		BF	10.48	0.00	10.00	0.00	0.00	18.33	0.00	183.33	0.00
1.75		LS	35.75	0.00	5.00	0.00	0.00	62.56	0.00	312.82	0.00
(Strength I) Design load:						**1073.11**	**607.72**	**668.69**	**307.17**	**2345.94**	**1214.58**

•Application of modifiers are similar to backwall.
•Modifier, ηi
$\eta_D \geq$ 1.05
$\eta_R \geq$ 1.00
$\eta_i \geq$ 1.00
for max. values
η_i = 1/(1.05x1.0x1.0)
η_i = 1.05
for min values
η_i = 1/(1.05x1.0x1.0)
η_i = 0.95

•Note: Friction forces are already factored on section Geometry and Load

b. Load Combination: EXTREME EVENT I

b.1 Load Combination: EXTREME EVENT I (CASE 1) (Case1: 100% P_{AE} + 50% P_{IR})

Commentary

•Modifier, η_i

η_i = 1.00

Load Factor max	Load Factor min	Load Type	Unfactored load, kN Hor'l	Unfactored load, kN Vert'l	Lever arm (m)	Factored Axial force, kN max	Axial force, kN min	Shear force, kN max	Shear force, kN min	Moment, kN-m max	Moment, kN-m min
1.25	0.90	DC 1	0.00	266.67	0.30	333.33	240.00	0.00	0.00	100.00	72.00
1.25	0.90	DC 2.1	0.00	31.20	-0.74	39.00	28.08	0.00	0.00	-28.86	-20.78
1.25	0.90	DC 2.2	0.00	4.20	-1.18	5.25	3.78	0.00	0.00	-6.20	-4.46
1.25	0.90	DC 2.3	0.00	360.00	0.00	450.00	324.00	0.00	0.00	0.00	0.00
1.25	0.90	DC 2.4	0.00	38.40	-1.18	48.00	34.56	0.00	0.00	-56.64	-40.78
1.50	0.65	DW	0.00	14.29	0.30	21.43	9.29	0.00	0.00	6.43	2.79
1.50	0.75	ES	29.61	0.00	5.00	0.00	0.00	44.42	22.21	222.10	111.05
0.50		LL	0.00	71.43	0.30	35.71	0.00	0.00	0.00	10.71	0.00
0.50		BF	10.48	0.00	10.00	0.00	0.00	5.24	0.00	52.38	0.00
0.50		LS	35.75	0.00	5.00	0.00	0.00	17.88	0.00	89.38	0.00
		FR	0.00	0.00	7.50	0.00	0.00	58.57	37.39	439.29	280.45
1.00		P_{AE}	506.31	0.00	3.33	0.00	0.00	506.31	0.00	1686.01	0.00
1.00		50%P_{IR_a}	4.13	0.00	8.75	0.00	0.00	4.13	0.00	36.17	0.00
1.00		50%P_{IR_b}	0.56	0.00	9.40	0.00	0.00	0.56	0.00	5.23	0.00
1.00		50%P_{IR_c}	47.70	0.00	3.75	0.00	0.00	47.70	0.00	178.88	0.00
1.00		50%P_{IR_d}	5.09	0.00	9.83	0.00	0.00	5.09	0.00	50.02	0.00
1.00		50%P_{IR_e}	75.53	0.00	5.00	0.00	0.00	75.53	0.00	377.63	0.00
					(Extreme Event 1-Case1) Design load:	**932.73**	**639.71**	**765.42**	**59.60**	**3162.51**	**400.26**

•Note: Friction forces are already factored on section Geometry and Load

b.2 Load Combination: EXTREME EVENT I (CASE 2) (Case2: 50% P_{AE} + 100% P_{IR})

However, if 50% P_{AE} < P_A, use P_A, else use 50%P_{AE}

Verification:

50% P_{AE} = 253.15

P_A =EH = 293.04 > 50% P_{AE}

therefore use EH

•Modifier, η_i

η_i = 1.00

Load Factor max	Load Factor min	Load Type	Unfactored load, kN Hor'l	Unfactored load, kN Vert'l	Lever arm (m)	Factored Axial force, kN max	Axial force, kN min	Shear force, kN max	Shear force, kN min	Moment, kN-m max	Moment, kN-m min
1.25	0.90	DC 1	0.00	266.67	0.30	333.33	240.00	0.00	0.00	100.00	72.00
1.25	0.90	DC 2.1	0.00	31.20	-0.74	39.00	28.08	0.00	0.00	-28.86	-20.78
1.25	0.90	DC 2.2	0.00	4.20	-1.18	5.25	3.78	0.00	0.00	-6.20	-4.46
1.25	0.90	DC 2.3	0.00	360.00	0.00	450.00	324.00	0.00	0.00	0.00	0.00
1.25	0.90	DC 2.4	0.00	38.40	-1.18	48.00	34.56	0.00	0.00	-56.64	-40.78
1.50	0.65	DW	0.00	14.29	0.30	21.43	9.29	0.00	0.00	6.43	2.79
1.50	0.75	ES	29.61	0.00	5.00	0.00	0.00	44.42	22.21	222.10	111.05
0.50		LL	0.00	71.43	0.30	35.71	0.00	0.00	0.00	10.71	0.00
0.50		BF	10.48	0.00	10.00	0.00	0.00	5.24	0.00	52.38	0.00
0.50		LS	35.75	0.00	5.00	0.00	0.00	17.88	0.00	89.38	0.00
		FR	0.00	0.00	7.50	0.00	0.00	58.57	37.39	439.29	280.45
1.00		EH	293.04	0.00	3.33	0.00	0.00	293.04	0.00	975.83	0.00
1.00		P_{IR_a}	8.27	0.00	8.75	0.00	0.00	8.27	0.00	72.35	0.00
1.00		P_{IR_b}	1.11	0.00	9.40	0.00	0.00	1.11	0.00	10.46	0.00
1.00		P_{IR_c}	95.40	0.00	3.75	0.00	0.00	95.40	0.00	357.75	0.00
1.00		P_{IR_d}	10.18	0.00	9.83	0.00	0.00	10.18	0.00	100.03	0.00
1.00		P_{IR_e}	151.05	0.00	5.00	0.00	0.00	151.05	0.00	755.25	0.00
					(Extreme Event 1-Case 2) Design load:	**932.73**	**639.71**	**685.15**	**59.60**	**3100.26**	**400.26**

•Note: Friction forces are already factored on section Geometry and Load

Commentary

c. **Load Combination: SERVICE 1**

Load Factor		Load Type	Unfactored load, kN		Lever arm (m)	Factored					
						Axial force, kN		Shear force, kN		Moment, kN-m	
max	min		Hor'l	Vert'l		max	min	max	min	max	min
1.00		DC 1	0.00	266.67	0.30	266.67	0.00	0.00	0.00	80.00	0.00
1.00		DC 2.1	0.00	31.20	-0.74	31.20	0.00	0.00	0.00	-23.09	0.00
1.00		DC 2.2	0.00	4.20	-1.18	4.20	0.00	0.00	0.00	-4.96	0.00
1.00		DC 2.3	0.00	360.00	0.00	360.00	0.00	0.00	0.00	0.00	0.00
1.00		DC 2.4	0.00	38.40	-1.18	38.40	0.00	0.00	0.00	-45.31	0.00
1.00		DW	0.00	14.29	0.30	14.29	0.00	0.00	0.00	4.29	0.00
1.00		EH	293.04	0.00	3.33	0.00	0.00	293.04	0.00	975.83	0.00
1.00		ES	29.61	0.00	5.00	0.00	0.00	29.61	0.00	148.06	0.00
1.00		FR	0.00	0.00	7.50	0.00	0.00	52.86	0.00	396.43	0.00
1.00		LL	0.00	71.43	0.30	71.43	0.00	0.00	0.00	21.43	0.00
1.00		LS	35.75	0.00	5.00	0.00	0.00	35.75	0.00	178.76	0.00
1.00		BF	10.48	0.00	10.00	0.00	0.00	10.48	0.00	104.76	0.00
					(Service 1) Design load:	**786.18**	**0.00**	**421.74**	**0.00**	**1836.20**	**0.00**

•Modifier, η_i

$\eta_i =$ 1.00

Summary of Load Combinations:

STRENGTH I						EXTREME EVENT 1 (CASE 1)					
Axial force (kN)		Shear Force (kN)		Moment (kN-m)		Axial force (kN)		Shear Force (kN)		Moment (kN-m)	
max	min	max	min	max	min	max	min	max	min	max	min
1073.11	**607.72**	**668.69**	**307.17**	**2345.94**	**1214.58**	**932.73**	**639.71**	**765.42**	**59.60**	**3162.51**	**400.26**

EXTREME EVENT 1 (CASE 2)						SERVICE 1					
Axial force (kN)		Shear Force (kN)		Moment (kN-m)		Axial force (kN)		Shear Force (kN)		Moment (kN-m)	
max	min	max	min	max	min	max	min	max	min	max	min
932.73	**639.71**	**685.15**	**59.60**	**3100.26**	**400.26**	**786.18**	**0.00**	**421.74**	**0.00**	**1836.20**	**0.00**

7.2.3 Determine the governing design forces:

Load Combination: EXTREME EVENT I (CASE 1)

Axial force = **932.73** kN

Shear Force = **765.42** kN

Moment = **3162.51** kN-m

•The summary of load combinations show that the Extreme Event 1 (Case 1) is the critical load case.

7.5.4 Verification of flexural resistance

				Commentary
Demand moment	3162.51	kN-m		
Concrete cover	75	mm		
Diameter of reinforcing bar	36	mm	Ab = 1017.88	
Diameter of shrinkage bar	16	mm		
Diameter of cross ties	12	mm		
Effective depth of concrete, de	1879	mm		
Width to be considered, b	1000	mm		• per 1m-width design
Overall thickness of component, h	2000	mm		

Minimum reinforcement • DGCS 12.4.3.3

Flexural cracking variability factor, γ_1	1.6	**for all other concrete*	
Ratio of specified min. to ult. tensile strength of steel, γ_3	0.67	**for A615, 414MPa steel*	
Modulus of rupture, f_r	3.334	mPa	
Section modulus, S_c	6.7E+08	mm^3	
Cracking moment, M_{cr}			
$\cdot M_{cr}=\gamma_3(\gamma_1 \cdot fr)\ Sc$	2382.45	kN-m	
*Mu_min 1.33*Md =*	4206.14	kN-m	
Condition: if Md > (min (Mcr, Mu_min), Md, (min (Mcr,Mu_min))			
Design moment for breast wall	**3162.51**	kN-m	**Governs!!!**

Steel ratio • DGCS 12.4.2.1

β_1 Coefficient Criterion: = 0.85

the factor β_1 shall be taken as 0.85 for concrete strengths not exceeding 28MPa. For concrete strength exceeding 28MPa, β_1 shall be reduced at a rate of 0.05 for each 7MPa strength excess of 28MPa but not less than 0.65

For required steel ratio, ρ

m_1 =	0.0573	
m_2 =	0.084	mm^2/N
R_n =	0.995	MPa
ρ =	0.0025	

Computation for main reinforcement

Required steel area, *As*	4604.625	mm^2		
Required spacing	221.06	say:	200	mm
Provided steel for breast wall, *As*	5089.38	mm^2		
Compression fiber to neutral axis, *c*	104.40	mm		
Depth of compression block, *a*	88.74	mm		
Nominal moment capacity of section, M_n	3874.91	kN-m		
Resistance factor, Ø	0.9	tension is controlled		

$$\cdot 0.75 \le \phi = 0.65 + 0.15\,(d_{eff}/c - 1) \le 0.9 \qquad 3.20$$

Ultimate moment capacity of section, $\text{Ø}M_n$	**3487.41**	kN-m	**OK!**
c/d =	1.10		

Commentary

<u>**Control of cracking by distribution of reinforcement**</u>

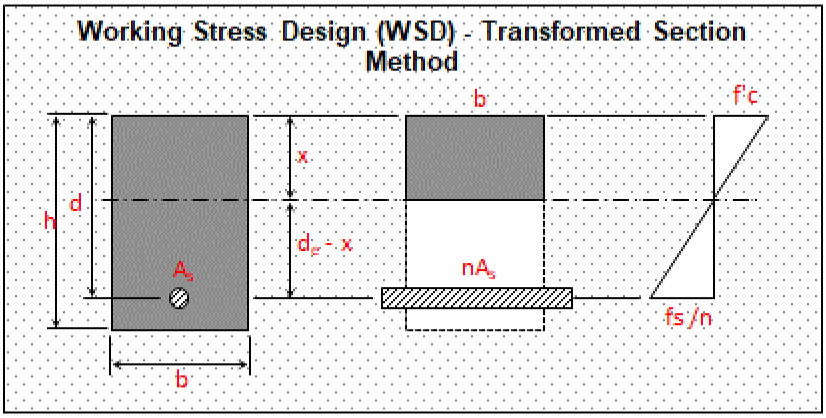
• DGCS 12.4.3.4

Applies to all reinforcements of concrete that exceeds 80% of the modulus of rupture, except deck slabs.

• DGCS 12.1.1.6

Moment demand at Service 1	1836.20	kN-m
80% of Modulus of rupture, f_r	2.201	MPa
Tension in the cross section	2.754	MPa

Tension in the cross-section exceeds 80% of the modulus of rupture, this provision has to be satisfied



Extreme tension fiber to center of flexural reinforcement, d_c	121	mm
Overall thickness of component, h	2000	mm
Compression fiber to the centroid of extreme tension steel, d_e	1879	mm
Neutral axis to extreme compression fiber, x	353.38	mm
Modulus elasticity of steel, E_s	200	GPa
Modulus elasticity of concrete, E_c	24.87	GPa
Modular ratio, n	8.042	
Cracked section moment of inertia of section, I_{NA}	1.1E+11	mm^4
Exposure factor, γ_e	1.00	
Exposure condition: <u>Class 1</u>		
Tensile stress in steel reinforcement at the service limit, f_s	204.85	MPa

•by quadratic equation to determine x:

a = 1000
b = 81855.53
c = -1.5E+08

x_1 = 353.38
x_1 = -435.24

$\beta s = 1 + d_c/(0.7\,(h - dc))$ 1.09

The spacing shall satisfy: $s \leq 123000\gamma_e / \beta_s f_{ss} - 2dc$ 307.84 mm

Initial spacing: **200** mm **SATISFIED!**

Using of 36mm Ø main bars spaced at 200mm O.C. for breast wall is adequate and safe

7.5.5 Verification of shear resistance

• DGCS 12.5.3.2

Effective shear depth, d_v 1834.63 mm

Taken as the distance measured perpendicular to the neutral axis, between the resultants of the tensile and compressive forces due to flexure; it need not to be taken to be less than the greater of 0.9de or 0.72h

=0.9 * de 1691.1 mm

=0.72 * h 1440 mm

Factor indicating ability of diagonally cracked concrete to transmit tension,	1.30		
Solution for β: **GENERAL PROCEDURE**			
Area of prestressing steel on tension side, A_{ps}	0	mm^2	
Area of non-prestressing steel, A_s	5089.38	mm^2	
Maximum aggregate size, a_g	20	mm	
Modulus of elasticity of prestressing tendons, f_{po}	0	MPa	
Factored axial force, N_u	-932726.2	N	
**Positive for tension; Negative for compression*			
Factored shear force, V_u	765415.9	N	
Absolute value of the factored moment, $\|M_u\|$	3.2E+09	N-mm	
**But not less than* $\|V_u - V_p\| d_v$			
Modulus of elasticity of prestressing steel, E_p	0	GPa	
Modulus of elasticity of steel, E_s	200	GPa	
Net longitudinal tensile strain, e_s	0.001		
Crack spacing parameter, S_{xe}	1741	mm	
Shear resistance from steel, V_s	0	kN	
Effective prestressing force, V_p	0	kN	
Shear resistance provided by concrete, V_c	1048.26	kN	
The nominal shear resistance, V_n	1048.26	kN	
**shall be determined as the lesser of:*			
$V_n = Vc + Vs + Vp$	1048.26	kN	
$V_n = 0.25 fc^{\wedge\prime} b_v d_v + Vp$	12842.40	kN	
Resistance factor for normal weight concrete, Ø	0.9		
Ultimate shear capacity of section, $ØV_n$	**943.44**	kN	**OK!**
	c/d =	1.23	
Section without shear reinforcement is adequate and safe.			

Commentary

- General procedure is basically applicable to design of walls, slab and footings with thickness > 400mm
- DGCS 12.5.3.3.2

7.5.6 Verification of interface shear resistance

Interface shear transfer shall be considered across a given plane at:

a) An existing or potential crack
b) An interface between dessimilar materials
c) An interface between two concrete cast at different times
d) The interface between different elements of the cross-section

Number of bars provided (for both faces per meter strip), N	10	pcs	
Area of shear reinforcement crossing the shear plane, A_{vf}	10178.8	mm^2	
**Minimum area of shear interface shall satisfy:*			
$A_{vf} \geq (0.35\, A_{cv})/f_y$	1472.530	mm^2	**SATISFIED!**

- •DGCS 12.5.5
- •N= (b/s) x 2sides
- •DGCS 12.5.5.3

				Commentary
Interface length considered to be engaged inshear transfer, L_{vi}	1000	mm		
Interface width considered to be engaged inshear transfer, b_{vi}	1746	mm		
Area of concrete considered to be engaged in interface shear transfer, A_{cv}	1746000	mm^2		
Permanent net compressive force normal to the shear plan, P_c	932726	N		
Factored interface shear force due to total load, V_{ui}	765.416	kN		

Cohesion and friction factors — •DGCS 12.5.5.2

**for concrete placed against a clean concrete surface, free of laitance, with surface intentionally roughened to an amplitude of 6mm:*

Cohesion factor, c	1.7	MPa
Friction factor, m	1.00	
Fraction of concrete to resist interface shear, K_1	0.25	
Limiting interface shear resistance, K_2	10.3	MPa

The nominal shear resistance of the interface plane shall be taken as:

$$V_{ni}=cA_{cv}+\mu(Avff_y+Pc)$$ 8125.11 kN

The nominal shear resistance, V_{ni} shall not be greater than the lesser of:

a) $K_1 f^{\wedge\prime} cA_{cv}$	12222	kN
b) $K_2 A_{cv}$	17983.80	kN

Nominal shear resistance of the interface plane, V_{ni}	8125.11	kN	
Resistance factor for normal weight concrete, f	0.9		
Factored interace shear resistance of the section, V_{ri}	7312.60	kN	**OK!**

Section is adequate for interface shear transfer.

7.5.7 Verification of shrinkage and temperature reinforcement

•DGCS 12.7.8

Reinforcement for shrinkage and temperature stresses shall be provided near surfaces of concrete exposed to daily temperaturechanges and in structural mass concrete.

Diameter of shrinkage and temperature bar	16	mm
Assumed spacing, S	150	mm
Assumed shrinkage and temperature reinforcement, A_s	1.34	mm^2/mm

Shrinkage and temperature reinforcement shall satisfy:

a) $As \geq (0.75\ bh)/(2\ (b+h) f_y)$	1.43	1.43	mm^2/mm
b) $0.233 \leq As \leq 1.27$		1.2700	mm^2/mm

Spacing shall not exceed:

a) 3.0 times the component thickness, or 450 mm
b) 300 mm for walls and footings greater than 450 mm thick
c) 300 mm for other components greather than 900 mm thick

			Commentary
Final shrinkage and temperature reinforcement, A_s	1270	mm^2 per meter	
Final spacing to be used	158	mm	
say:	150	mm	

Therefore use 16mm for temperature and shrinkage bar spaced at 150mm O.C. eachface

7.5.8 Development of reinforcement

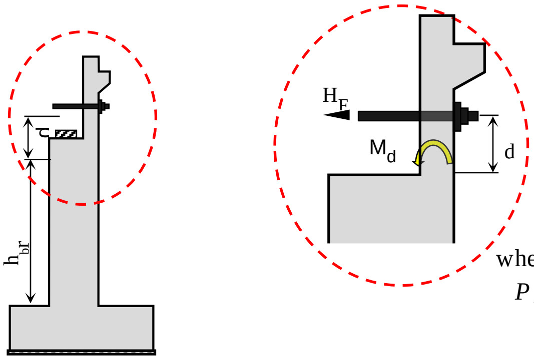
					•DGCS 12.8.2.1
Diameter of main bars, d_b	36	mm			
Area of main bars, A_b	1017.88	mm^2			
Basic tension development length, l_{db}	1596.59	mm			
Minimum development length *(only for d_b 36 mm and lesser)*	896.4	mm			
Modification Factor That Decrease					
Modification factor 1	0.8				
Modification factor 2	0.905				
Final development length, l_d	1155.62	say:	1500	mm	

Note: For effective anchorage the rebars should rest on pilecap bottom bars

7.5.9 Verification of demand forces for unseating prevention device for backwall.

•BSDS 7.3

The ultimate strenght of an unseating prevention device shall not be less than the design seismic force.



When the unseating prevention device directly connects the superstructure, the design seismic force shall be:

$$H_F = P_{LG}$$

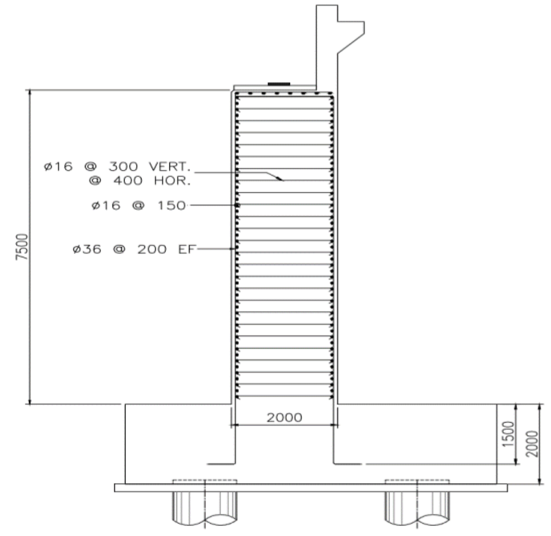
However, H_F shall not exceed 1.5 R_D

where:

P_{LG} = The lesser value corresponding to lateral (hor'l) capacity of the breast wall calculated from its nominal flexural resistance, or the nominal shear resistance of the breast wall.

Distance of unseating prevention device from the base of backwall, d	0.6	m	
Nominal flexural resistance of the breast wall, M_n	3874.91	kN-m	
Deadload reaction from superstructure (per meter strip), R_D	266.67	kN	
a. Lateral capacity of breast wall from its nominal flexural resistance	516.65	kN	$= M_n/h_{br}$
b. Nominal shear resistance of the breast wall, V_n	1048.26	kN	
c. 1.5 times the deadload reaction of superstructure	400	kN	
Therefore, the design seismic force of the unseating prevention device	**400**	kN	$= H_F$
Design moment to be considered in designing the backwall, M_d	**240**	kN-m	$= H_F x\ d$

7.5.10 Breast wall details



BREASTWALL

Commentary

•Typical details of breast wall. Other miscellaneous details not shown. In practice the vertical reinforcement are applied to both faces of breast wall.

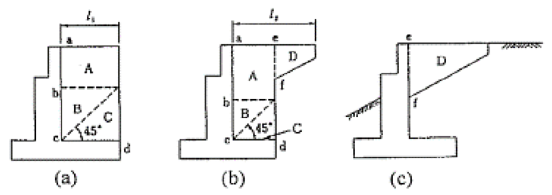
7.6 DESIGN OF WING WALLS

Reference: JRA, 2002-Part IV substructure Art. 8.4.4

Design of wing walls

(1) The wing walls shall be designed as slabs to receive superimposed loads due to live loads and the earth pressure.

(2) The slabs in this case shall be cantilevers fixed to a wall or slabs fixed on two sides to a wall and footing.



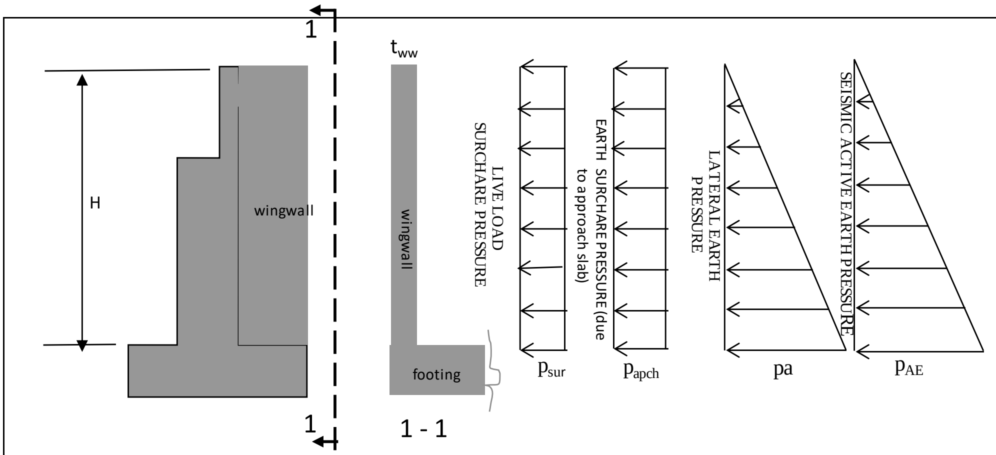
Shapes of wing walls

Note:

When l_1 or l_2 are greater than 8m, design the wingwall fixed on 2 sides (vertical wall and footing).Otherwise design part A, B, and C as cantilever wall fixed on sides a-b, b-c, and c-d, respectively. While part D fixed on side e-f.

• The wing wall type in this exercise is a vertical wall fixed on 2 sides (breast wall and footing). The type shown in (a) is generally applied for urban bridges when bridge sides are utilized for secondary roads. It is recommended to adopt the JRA method for the calculation of loads and design of wing walls.

7.6.1 Diagram of forces acting to wing wall



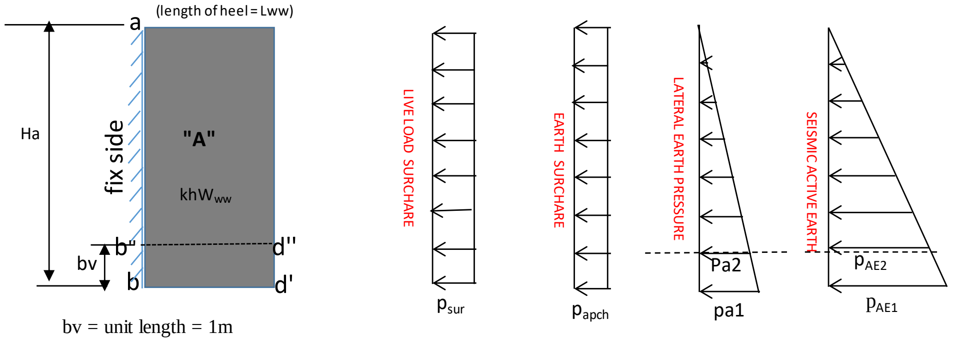
Schematic diagram of lateral forces acting on wing wall.

Commentary

- In this exiercise, the approach slab is considered equivalent earth surcharge.
- As noted the wingwall height is extended to level of top of backwall to simplify the analysis.

7.6.2 Determine the applicable loads acting to each part (part "A", "B" and "C") as shown in the shape of wing wall (a).

PART "A"



bv = unit length = 1m

Diagram of lateral forces acting to part "A"

From given parameters and calculated coefficients :

	kh =	0.265	
	ka =	0.308	
	k_{AE} =	0.533	
δ_c =	Υc =	27	kN/m^3
Υ_{soil} =	Υ_s =	19	kN/m^3
	H =	10	m
wingwall dim.	t_{ww} =	0.7	m
	L_{ww} =	3.00	m
Surcharge =	h_{sur} =	0.61	m
Approach slab =	h_{apch} =	0.4	m

Hb = $(\tan 45^0) \times L_{ww}$ = 3.0 m

Ha = (H - Hb) = 7.0 m

- It is recommended the the aproach of analysis is part by part as per shown in JRA shape of wing wall..
- The unit weight of concrete was increased by 15% to compensate the weights of concrete railing and sidewalk to simplify the load calculation.

Commentary

Calculation of horizontal pressure:

Loads	pressure at bd' (for Ha)		pressure at b''d'' (for Ha -1.0m)	
due to lateral pressure	kPa		kPa	
$pa1 = ka*\Upsilon_s*Ha$	pa1 =	40.96	pa2 = ka*ϒs*(Ha - 1.0m) =	35.11
$p_{AE1} = k_{AE}*\Upsilon_s*Ha$	p_{AE1} =	70.89	$p_{AE}2$ = kAE*ϒs*(Ha - 1.0m) =	60.76
$p_{sur} = ka * \Upsilon_s*h_{sur}$	p_{sur} =	3.57		
$p_{apch} = ka * \Upsilon c*h_{apch}$	p_{apch} =	3.33		
due to Inertial mass, p_{IR}				
$khW_{ww} = kh*(L_{ww}*Ha*t_{ww}*\Upsilon_c)$	khW_{ww} =	105.18	kN	

Calculation of forces per unit length: bv = 1.00 m

Loads	Force, kN		Lever arm , m		Moment,kN*m
Earth pressure (EH)	(pa1 +pa2)/2 *L_{ww}*bv =	114.11	$L_{ww}/2$ =	1.5	171.17
Seismic earth pressure (p_{AE})	(p_{AE1} + p_{AE2})/2 *L_{ww}*bv =	197.48	$L_{ww}/2$ =	1.5	296.21
Liveload surcharge(LS)	p_{sur} *L_{ww}*bv =	10.71	$L_{ww}/2$ =	1.5	16.06
Earth surcharge(ES)	p_{apch} *L_{ww}*bv =	9.98	$L_{ww}/2$ =	1.5	14.97
Inertial force mass (p_{IR})	khW_{ww}/Ha =	15.03	$L_{ww}/2$ =	1.5	22.54

where : Moment = Force x lever arm

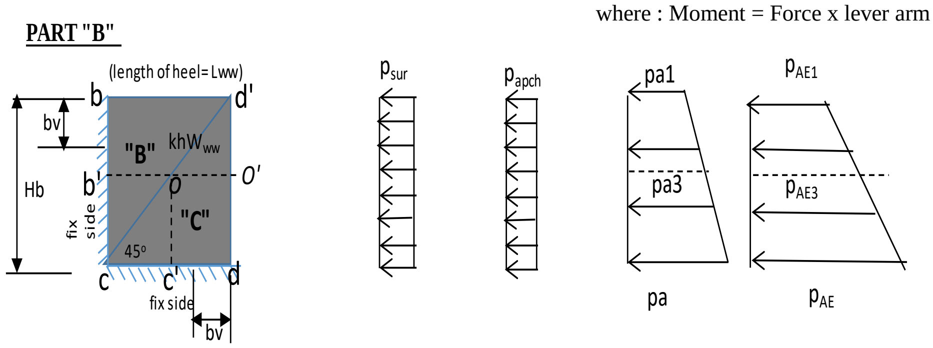


Diagram of lateral forces of part "B" and "C"

Calculation of horizontal pressure :

Loads	pressure at bd' (for Ha)		pressure at b' O' (for Ha+Hb/2)	
due to lateral pressure	kPa		kPa	
$pa1 = ka*\Upsilon_s*Ha$	pa1 =	see above	pa3 = ka*ϒs*(Ha+Hb/2) =	49.74
$p_{AE1} = k_{AE}*\Upsilon_s*Ha$	p_{AE1} =	see above	p_{AE3} = kAE*ϒs*(Ha +Hb/2)=	86.08
$p_{sur} = ka * \Upsilon_s*h_{sur}$	p_{sur} =	see above		
$p_{apch} = ka * \Upsilon c*h_{apch}$	p_{apch} =	see above		
due to Inertial mass, p_{IR}				
$khW_{ww} = kh*(1/2*L_{ww}*Hb*t_{ww}*\Upsilon_c)$	khW_{ww} =	22.54	kN	

Note: ¾ L_{ww} is average length between b-b'

where : Moment = Force x lever arm

Commentary

PART "C" (refer to figure of PART "B")

Calculation of horizontal pressure:

Loads	pressure at cd (for H)	
due to lateral pressure	kPa	
$pa = ka*\Upsilon_s*H$	pa=	58.52
$p_{AE} = k_{AE}*\Upsilon_s*H$	p_{AE} =	101.27
$p_{sur} = ka * \Upsilon_s*h_{sur}$	p_{sur} =	see above
$p_{apch} = ka * \Upsilon c*h_{apch}$	p_{apch} =	see above
due to Inertial mass, p_{IR}		
$khW_{ww} = kh*(1/2*L_{ww}*Hb*t_{ww}*\Upsilon_c)$	khW_{ww} =	22.54 kN

Calculation of forces per unit length: bv = 1.00 m

Loads	Force, kN		Lever Arm , m		Moment,kN*m
Earth pressure(EH)	pa*3/4 Hb*bv =	131.67	3/4Hb/2 =	1.125	148.13
Seismic earth pressure (p_{AE})	p_{AE} *3/4Hb*bv =	227.86	3/4Hb/2 =	1.125	256.34
Liveload surcharge(LS)	p_{sur} *3/4 *Hb*bv =	8.03	3/4Hb/2 =	1.125	9.04
Earth surcharge(ES)	p_{apch} * 3/4 *Hb*bv =	7.48	3/4Hb/2 =	1.125	8.42
Inertial force mass(p_{IR})	khW_{ww}/Lww =	7.51	3/4Hb/2 =	1.125	8.45

Note: 3/4 Hb is average height between c'-d where : Moment = Force x lever arm

7.6.3 Design Part "A"

Summary of unfactored loads		
Loads	Force, kN	Moment,kN*m
Earth pressure (EH)	114.11	171.17
Seismic earth pressure (p_{AE})	197.48	296.21
Liveload surcharge(LS)	10.71	16.06
Earth surcharge(ES)	9.98	14.97
Inertial force mass (p_{IR})	15.03	22.54

η = 1.0 for other bridges hence modifier for wingwalls is assumed 1.0

- The load combinations and reinforced design is basically same approach as to breast wall and backwall design.

a. Load Combinations:

STRENGTH - 1

Load factor		Loads	Factored Loads			
			Force, kN		Moment,kN*m	
max	min		max	min	max	min
1.5	0.9	EH	171.17	102.70	256.76	154.05
1.75	0	LS	18.74	0.00	28.11	0.00
1.5	0.75	ES	14.97	7.48	22.45	11.23
		Total	**204.88**	**110.19**	**307.32**	**165.28**

Commentary

EXTREME EVENT - 1 : CASE 1 (100% P_{AE} + 50% P_{IR})

Load factor		Loads	Factored Loads			
			Force, kN		Moment,kN*m	
max	min		max	min	max	min
1	0	p_{AE}	197.48	0.00	296.21	0.00
0.5	0	LS	5.35	0.00	8.03	0.00
1.5	0.75	ES	14.97	7.48	22.45	11.23
1	0	50%p_{IR}	7.51	0.00	11.27	0.00
		Total	**225.31**	**7.48**	**337.97**	**11.23**

EXTREME EVENT - 1 : CASE 2 (50% P_{AE} +100% P_{IR})

Verification: if 50%P_{AE} < Pa, use Pa, else use 50% P_{AE}

Force, kN	
50% P_{AE} =	98.74
Pa =EH =	114.11

Moment, kN*m	
50% P_{AE}	148.107
Pa =EH =	171.17

- It shows 50% PAE is lesser than Pa (=EH), therefore use Pa.

Load factor		Loads	Factored Loads			
			Force, kN		Moment,kN*m	
max	min		max	min	max	min
1.0	0	EH	114.11	0.00	171.17	0.00
0.5	0	LS	5.35	0.00	8.03	0.00
1.5	0.75	ES	14.97	7.48	22.45	11.23
1.0	0	p_{IR}	15.03	0.00	22.54	0.00
		Total	**149.46**	**7.48**	**224.19**	**11.23**

SERVICE - 1

Load factor		Loads	Factored Loads			
			Force, kN		Moment,kN*m	
max	min		max	min	max	min
1	0	EH	114.11	0.00	171.17	0.00
1	0	LS	10.71	0.00	16.06	0.00
1	0	ES	9.98	0.00	14.97	0.00
		Total	**134.80**	**0.00**	**202.20**	**0.00**

b. Determine the governing design forces:

Load Combination: EXTREME EVENT I (CASE 1)

Shear Force = **225.31** kN

Moment = **337.97** kN-m

- The load combinations show that the Extreme Event 1 (Case 1) is the critical load case.

Commentary

c. Verification of flexural resistance

Moment demand, Md =			**337.97**	kNm
Main reinf. (inner face/horizontal), db1	Ab1=	491 mm^2	25	mm
Secondary reinf. (outer face/hor'l and vertical), db2	Ab2=	201 mm^2	16	mm
conc. cover, cc			75	mm
concrete compressive strength, f'c			28	MPa
yield strenght of steel, fy			415	MPa
thickness of wall, tww			700	mm
unit width of wall, bv			1000.0	mm
Height of wall, Ha			7000.0	mm
Length of wall, Lww			3000.0	mm

• per 1m-width design

• DGCS 12.4.3.3

Minimum reinforcement

$Mcr = \Upsilon_3 (\Upsilon_1 f_r)S_c$

where:

f_r = 3.334 MPa (fr = 0.63*√f'c)

Υ_1 = 1.6 *for all other concrete*

Υ_3 = 0.67 *for A615, 414MPa steel*

S_c = 1/6(bt^2) Section modulus

S_c = (bv*tww^2)/6

S_c = 81,666,666.7 mm^3

$Mcr = \Upsilon_3 (\Upsilon_1 f_r)S_c$ = 291.88 kN-m

Mu_min 1.33*Md = 449.50 kN-m

Condition: if Md > (min (Mcr, Mu_min), Md, (min (Mcr,Mu_min))

Therefore, Md = **337.97** kN-m

Computation for main reinforcement

effective de = tww-cc-1/2 main reinf	de=	613	mm
m1=0.85*f'c/fy=	m1	0.057	
m2=2/(0.85*f'c)=	m2	0.084	mm^2/N
Rn= Md/(Ø bv*de^2)	Rn=	1.001	Mpa
ρ = m1*(1-sqrt(1-m2*Rn)) =	ρ =	0.002465	
As = ρ * bv *de =	As =	1509.785	mm^2
S =Ab1*bv/As	S =	325	mm
Try : say S_prov =	S_prov =	250	mm
As_prov =Ab1*bv/S_prov	As_prov	1963	mm^2
β =	β =	0.85	
c= As_prov *fy/0.85*f'c*β*bv	c=	40.26	mm
a=c*β1 =	a=	34.22	mm
Mn =(As_prov*fy)(de-a/2) =	Mn =	484.9	kN-m
Check net tensile strain, εt			
εt = 0.003*((de/c)-1) =	εt =	0.043	> 0.005

Tension Controlled!!!, Reduction factor =0.9

Ultimate moment capacity of section, $ØM_n$	ØMn =	**436.4**	kN-m
	c/d =	1.29	

Section is safe in flexure!!!

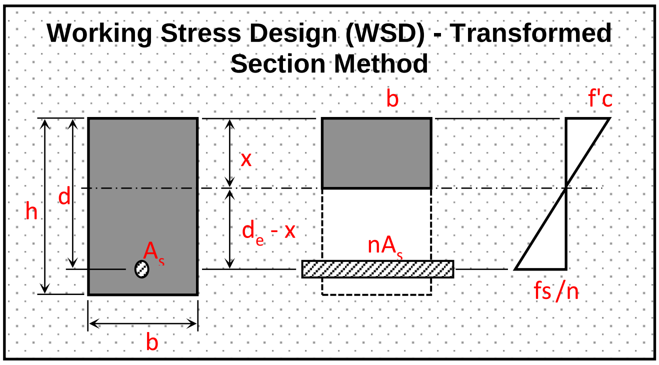
Control of cracking by distribution of reinforcement

Note: This provision applies to all members when tension in the cross section exceeds the 80% of the modulus rupture @ applicable service limit load combination.

Moment demand at Service 1		**Ms =**	**202.2**	kN-m
fr = 0.52*√f'c		fr =	2.75	MPa
		80%fr =	2.20	MPa
fss = Ms/Sc		fss=	2.48	MPa

Section 12.4.3.4 need to satisfy!!!

Es =	Modulus of elasticity of reinf.	Es =	200,000.00	MPa
Ec =	Modulus of elasticity of concrete	Ec=	27,000.00	MPa
n=	modular ratio		7.41	
n*As=	transformed area of reinforcement	n*As =	14,537	mm^2
de=	effective de	de =	613	mm
bv =	unit width	bv=	1,000	mm



Determine N.A. Gen. Eq. : 1/2 *bv *x^2 = nAs*(de-x)

x= 120 mm

j*de = de - (x/3) = 572.6 mm

Actual f_{ss} = Mservice 1 /(As j.de) = 179.94 MPa

where

Υ_e = 1.0 for class 1 exposure	Υ_e=	1	
d_c = conc cover to centroid of main bars	d_c =	87.5	mm
h= thickness of wall, tww			
f_{ss} = tensile stress of reinf at service limit	= Mservice 1/Asjd		

$$\beta s = 1 + d_c/(0.7\,(h - dc))$$

β_s= 1.204

The spacing shall satisfy: Eq 12.4.3.4-1

$$s \leq 123000\gamma_e / \beta_s f_{ss} - 2dc$$

s= 392.7 mm

S_prov = 250.0 mm

Section 12.4.3.4 satisfied!!!

The following shall be satisfied:

.

Commentary

- DGCS 12.4.3.4
- DGCS 12.1.1.6
- DGCS 12.1.1.6

Commentary

Allowable f_{ss} = 123000*Υ_e/(β_s(S_prov + 2*d_c)) 240.4 | >Actual f_{ss} = 179.94

Section 12.4.3.4 satisfied!!!

< 0.6fy = 249

Section 12.4.3.4 satisfied!!!

Therefore using 25mm diam @ 250 is adequate

<u>Check for minimum spacing of reinforcement.</u>

• DGCS 12.7.3.1

For cast in place concrete, clear distance between paraller bars in a layer shall not be less than:

▪ 1.5 x nominal diam of bars	=	37.5	mm	***satisfied the required min. spacing!!!***
▪ 1.5 x maximum size of aggregates	=	37.5	mm	***satisfied the required min. spacing!!!***
▪ 38mm	=	38	mm	***satisfied the required min. spacing!!!***

<u>Check for maximum spacing of reinforcement (for walls and slabs)</u>

• DGCS 12.7.3.2

▪ s < 1.5 x t	=	1050	mm	***satisfied the required max. spacing!!!***
▪ 450mm	=	450	mm	***satisfied the required max. spacing!!!***

d. Verification of shear resistance

• DGCS 12.5.3.2

3 procedures of determining shear resistance:

- Simplied procedure for non-prestressed sections
- **<u>General procudure</u>** <<<< this procedure is applicable for design of wingwall
- Simplied procedure for prestressed and prestressed sections

• DGCS 12.5.3.3.2

Shear demand, Vd = **225.31** kN

Nominal shear resistance, Vn= min (Vn1, Vn2)

where :

Vn1= Vc + Vs

Vc = 0.083*β(√f'c)bv dv

$$Vs = \frac{Av*fy*dv*(\cot\theta + \cot\alpha)*\sin\alpha}{s}$$

Vn2 = 0.25*f'c*bv*dv

de =	613	mm
dv1 = (de - a/2)	595	mm
dv2= 0.9 x de	551	mm
dv3= 0.72tww	504	mm
dv=	595	mm

Calculate for β:

$$\beta = \frac{4.8}{(1+750\varepsilon_s)}\frac{51}{(39+s_{xe})}$$

DGCS Eq 12.5.3-2
β in english units

where :

$$\varepsilon_s = \frac{\left(\frac{|M_u|}{d_v}+0.5N_u+|V_u-V_p|-A_{ps}f_{po}\right)}{\left(E_sA_s+E_pA_{ps}\right)}$$

Mu =	337.97 kN-m				
dv=	0.595 m				
Nu =	0.00 kN	(note: consider no vertical axial action on wall)			
Vu=	225.31 kN				
Es=	200,000,000.00 kN/m^2				
As=	0.001963 m^2				
Mu > Vu*dv	OK!				
EpAps =	0				
Apsfpo=	0				
Vp =	0				
Sxe = Sx *(35/ag + 16)					
where :					
ag =size of aggregates	20 mm				
Sx =dv	595 mm				
Sxe =	578.851 mm	300mm≤Sxe≤2025mm	limit to min.:	OK!!!	
ε_s =	0.0020		limit to max.:	OK!!!	
β =	1.575				

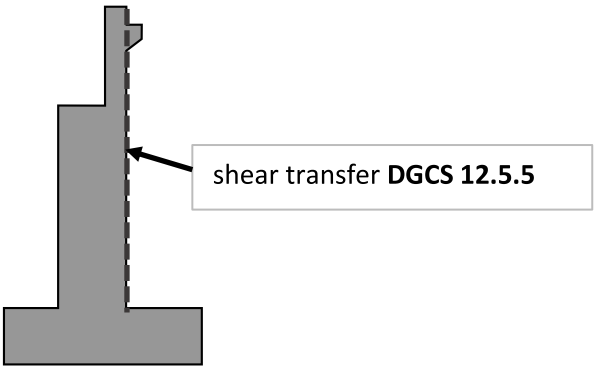
Therefore:

Vc= 0.083*β(√f'c)bv dv =	411.89 kN	
Vs=	0.00 kN	Assume no transverse reinf.
Vn1 = Vc + Vs	411.89 N	
Vn2 = 0.25*f'c*bv*dv =	4,167.7 kN	
Vn = min (Vn1, Vn2) =	411.9 kN	
Ultimate shear resistance, ØVn =	**370.7** kN	
Shear demand, Vd =	**225.31** kN	
	c/d = 1.65	

Section is safe in shear!!!

e. Verification of interface shear resistance

Investigate the interface shear resistance between breast wall and wing wall



Shear demand, Vd = Vui Vui = **225.31** kN

Nominal interface shear resistance, Vni

Factored interface resistance, Vri = ØVni

Vri ≥ Vui

Commentary

• DGCS 12.5.5

Commentary

where :

$V_{ni} = c\,A_{cv} + \mu\,(A_{vf}\,fy + P_c)$

$V_{ni} \leq \min\,(K_1 f'_c\,A_{cv},\ K_2\,A_{cv})$

A_{cv} =	area of concrete cosidered to be engaged in interface shear transfer, mm^2	529,500	mm^2
	= $b_{vi}\,L_{vi}$		
b_{vi} =	interface width considered to be engaged in shear transfer, mm	529.5	mm
	= tww - 2x cc-1/2(db1)-1/2(db2)*bv		
L_{vi} =	interface length considered to be engaged in shear transfer, = bv, mm	1000.0	mm
A_{vf} =	area of shear reinf. crossing the shear plane witihin the area of A_{cv}, mm^2	2766	mm^2
	inner face = A_prov = 1963 mm^2		
	outer face = Ab2*bv/S_outerface 803.84 mm^2		
	Note: area of outer face to be confirmed, see Shrinkage /Temp bars computations.		
	try S_outerface = 250 mm		
c =	cohesion factor, MPa	1.7	MPa
μ =	friciton factor, MPa	1	
fy =	yield strength of reinf. not to exceed 60MPa	415	MPa
P_c =	permanent net compressive normal to the shear plane; if force is tensile, P_c=0.0kN	0.0	kN
f'_c =	compressive strength of the weaker concrete either side of the interface, MPa	28	MPa
K_1 =	fraction of concrete available to resist interface transfer shear,	0.25	
K_2 =	limiting interface shear resistance	10.3	MPa

•DGCS 12.5.5.2

$V_{ni} = c\,A_{cv} + \mu\,(A_{vf}\,fy + P_c)$		2048	kN
$V_{n1} = K_1 f'_c\,A_{cv}$		3,707	kN
$V_{n2} = K_2\,A_{cv}$		5,454	kN
Therefore,	Vni =	2048	kN
Factored interface resistance, Vri = ØVni =		**1843.363**	kN
Shear demand, Vd = Vui	Vui =	**225.31**	kN
	c/d =	8.18	

Section is safe in interface shear!!!

Minimum area of interface shear

•DGCS 12.5.5.3

$A_{vf} \geq 0.35A_{cv}/fy$ = 446.57 mm^2

satisfied the required min. area!!!

f. Verification of shrinkage and temperature reinforcement

•DGCS 12.7.8

Ast = 0.75(bxh)/2(b+h)fy

Limitations: 0.223<Ast<1.27

where:

b = least width of component section, mm 7000.0 mm

h=least thickness of component section, mm 700.0 mm

Commentary

Ast= 0.575 mm^2/mm

Limitations: 0.223<Ast<1.27

therefore, Ast= 0.575 mm^2/mm each face/each direction

db1 = 16 mm

Ab1= 201 mm^2

S = Ab1/ Ast

S = 349.479 say S_prov = 250 mm

checking: Ast= Ab1/S_prov = 0.80384 mm^2/mm

Use : 16 mm

where Spacing S is: @ outer face/bothway

- S should not be greater than 3 X thickness, or 450mm *satisfied the required min. spacing!!!*
- S should not be greater than 300 for walls and footings > 450mm. *satisfied the required min. spacing!!!*
- S should be 300mm for other components > 900mm thick.

g. Development length of the reinforcement.

•DGCS 12.8.2.1

Development length of bars under tension.

sizes	Ab	l_{db}		Modification factors(MF) that increase ld	Modification factors(MF) that decrease l_d	l_d =ldb*MF
	mm^2	mm				mm
36	1017.36	0.02*Ab*fy/√f'c OR .06$d_b$fy-minimum	1596	Note: factors are not applicable in this exercise	see below factors	
25	490.625		770			616
28	615.44		965			
20	314		498			
16	200.96		398			319

fy = 415 MPa

f'c = 28 MPa

ld =300mm minimum

MF = As_req'd/As_prov

Note: (a) Reinforcement being developed in the length under consideration space laterally not less than 150mm c to c and not less than 75mm clear cover measured in the direction of the spacing. , MF =0.8

(b) Anchorage or development for the full yield strength of reinf. is not required or where reinf. in flexural is in excess of that required by analysis.

(c) Reinf. is enclosed within a spiral composed of bars of not less than 6mm in diam and spaced at not more than a 100 mm pitch. MF = 0.75

7.6.4 Design Part "B"

Summary of unfactored loads		
Loads	Force, kN	Moment,kN*m
Earth pressure (EH)	102.04	114.80
Seismic earth pressure (p_{AE})	176.59	198.66
Liveload surcharge(LS)	8.03	9.04
Earth surcharge(ES)	7.48	8.42
Inertial force mass (p_{IR})	7.51	8.45

•The following sections only summarize the load combinations and governing forces for Parts "B" and "C" of the wing wall. The design analysis is skipped since the design approach is similar to Part "A" . By inspection the governing design forces of Part "A"is relatively larger than design forces for Part "B" and "C".

a. Load Combinations:

STRENGTH - 1

Load factor		Loads	Factored Loads			
			Force, kN		Moment,kN*m	
max	min		max	min	max	min
1.5	0.9	EH	153.07	91.84	172.20	103.32
1.75	0	LS	14.06	0.00	15.81	0.00
1.5	0.75	ES	11.23	5.61	12.63	6.31
		Total	**178.35**	**97.45**	**200.64**	**109.63**

EXTREME EVENT - 1 : CASE 1 (100% P_{AE} + 50% P_{IR})

Load factor		Loads	Factored Loads			
			Force, kN		Moment,kN*m	
max	min		max	min	max	min
1	0	p_{AE}	176.59	0.00	198.66	0.00
0.5	0	LS	4.02	0.00	4.52	4.52
1.5	0.75	ES	11.23	5.61	12.63	12.63
1	0	50%p_{IR}	3.76	0.00	4.23	4.23
		Total	**195.59**	**5.61**	**220.04**	**21.37**

EXTREME EVENT - 1 : CASE 2 (50% P_{AE} +100% P_{IR})

Verification: if 50%P_{AE} < Pa, use Pa, else use 50% P_{AE}

Force, kN	
50% P_{AE} =	88.29
Pa =EH =	102.04

Moment, kN*m	
50% P_{AE} =	99.33
Pa =EH =	114.80

Load factor		Loads	Factored Loads			
			Force, kN		Moment,kN*m	
max	min		max	min	max	min
1.0	0	EH	102.04	0.00	114.80	0.00
0.5	0	LS	4.02	0.00	4.52	0.00
1.5	0.75	ES	11.23	5.61	12.63	6.31
1.0	0	p_{IR}	7.51	0.00	8.45	0.00
		Total	**124.80**	**5.61**	**140.40**	**6.31**

SERVICE - 1

Load factor		Loads	Factored Loads			
			Force, kN		Moment,kN*m	
max	min		max	min	max	min
1	0	EH	102.04	0.00	114.80	0.00
1	0	LS	8.03	0.00	9.04	0.00
1	0	ES	7.48	0.00	8.42	0.00
		Total	**117.56**	**0.00**	**132.26**	**0.00**

Commentary

For detailing purposes, adopt the design results of Part "A".

• It shows 50% PAE is lesser than Pa (=EH), therefore use Pa.

7.6.5 Design Part "C"

Summary of unfactored loads		
Loads	Force, kN	Moment,kN*m
Earth pressure (EH)	131.67	148.13
Seismic earth pressure (p_{AE})	227.86	256.34
Liveload surcharge(LS)	8.03	9.04
Earth surcharge(ES)	7.48	8.42
Inertial force mass (p_{IR})	7.51	8.45

a. Load Combinations:

STRENGTH - 1

Load factor		Loads	Factored Loads			
			Force, kN		Moment,kN*m	
max	min		max	min	max	min
1.5	0.9	EH	197.51	118.50	222.19	133.32
1.75	0	LS	14.06	0.00	15.81	0.00
1.5	0.75	ES	11.23	5.61	12.63	6.31
		Total	**222.79**	**124.12**	**250.64**	**139.63**

EXTREME EVENT - 1 : CASE 1 (100% P_{AE} + 50% P_{IR})

Load factor		Loads	Factored Loads			
			Force, kN		Moment,kN*m	
max	min		max	min	max	min
1	0	p_{AE}	227.86	0.00	256.34	0.00
0.5	0	LS	4.02	0.00	4.52	4.52
1.5	0.75	ES	11.23	5.61	12.63	12.63
1	0	50%p_{IR}	3.76	0.00	4.23	4.23
		Total	**246.86**	**5.61**	**277.71**	**21.37**

EXTREME EVENT - 1 : CASE 2 (50% P_{AE} +100% P_{IR})

Verification: if 50%P_{AE} < Pa, use Pa, else use 50% P_{AE}

Force, kN	
50% P_{AE} =	113.93
Pa =EH =	131.67

Moment, kN*m	
50% P_{AE} =	128.17
Pa =EH =	148.13

Load factor		Loads	Factored Loads			
			Force, kN		Moment,kN*m	
max	min		max	min	max	min
1.0	0	EH	131.67	0.00	148.13	0.00
0.5	0	LS	4.02	0.00	4.52	0.00
1.5	0.75	ES	11.23	5.61	12.63	6.31
1.0	0	p_{IR}	7.51	0.00	8.45	0.00
		Total	**154.43**	**5.61**	**173.73**	**6.31**

Commentary

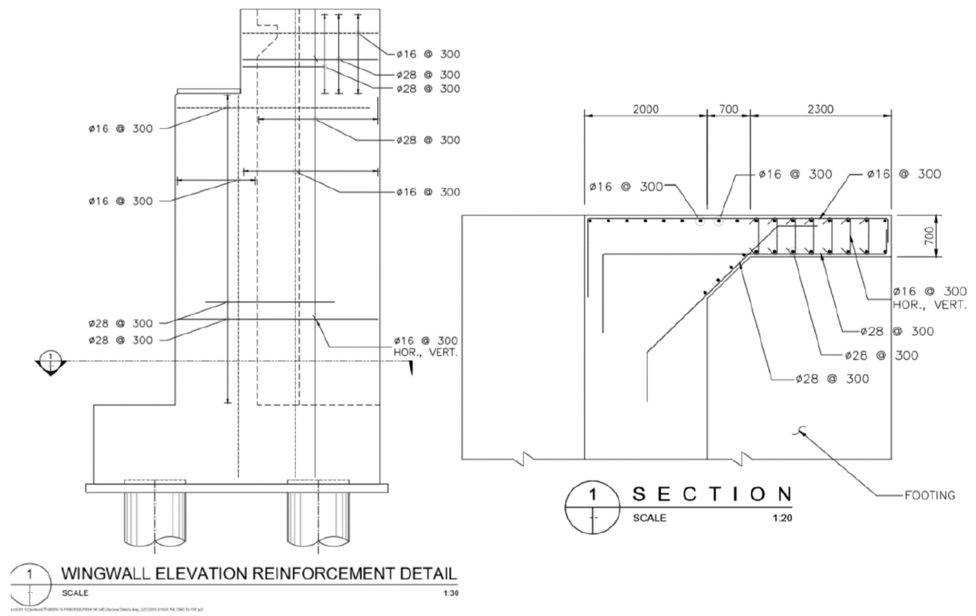
• It shows 50% PAE is lesser than Pa (=EH), therefore use Pa.

Commentary

SERVICE - 1

Load factor		Loads	Factored Loads			
			Force, kN		Moment,kN*m	
max	min		max	min	max	min
1	0	EH	131.67	0.00	148.13	0.00
1	0	LS	8.03	0.00	9.04	0.00
1	0	ES	7.48	0.00	8.42	0.00
		Total	**147.19**	**0.00**	**165.58**	**0.00**

7.6.6 Wing wall details



• Typical details of wing wall.

7.7 DESIGN OF PILE CAP

7.7.1 Determine the applicable loads calculated from section 7.3 Geometry and Load Calculations

1. Permanent loads

Dead load rection force of the superstructure, *DC 1*	2800	kN
Dead load from self weight, *DC 2*		
a) Weight of backwall, *DC 2.1*	328	kN
b) Weight of corbel, *DC 2.2*	44.10	kN
c) Weight of breast wall, *DC 2.3*	3780	kN

•Note: Loads considering full length of abutment

Commentary

d) Weight of footing, *DC 2.4*	3528	kN
e) Weight of wingwalls, *DC 2.5*	977.76	kN
f) Weight of approach slab, *DC 2.6*	403.20	kN
Vertical pressure from dead load of earth fill, *EV*	5985.0	kN
Deadload of wearing surfaces and utilities, *DW*	150	kN
Horizontal earth pressure load, *EH* ($=P_A$)	4430.80	kN
Earth surcharge load, *ES*	373.12	kN
2. Braking force, BF	110	kN
3. Earthquake force, EQ		
Seismic active earth force, P_{AE}	7655.38	kN
Seismic inertial force, P_{IR}		
a) kh * Backwall	86.81	kN
b) kh * Corbel	11.69	kN
c) kh * Breast wall	1001.70	kN
d) kh * Footing	934.92	kN
e) kh *Wing walls	259.11	kN
f)kh *Approach slab	106.85	kN
g)kh * Soil	1586.03	kN
4. Vehicular live load	750	kN
5. Live load surcharge, LS	450.46	kN
6. Friction load, FR		
a) Strength 1		
- at maximum condition	755.63	kN
- at minimum condition	392.63	kN
b) Extreme event 1		
- at maximum condition	615	kN
- at minimum condition	392.63	kN
c) Service 1		
- at maximum condition	555	kN
- at minimum condition	0	kN
7. Stream Flow, WA		
Water load and stream pressure, *WA*		
a) Water load due to OWL, *WA 1*	1442.07	kN
b) Water load due to DFL, *WA 2*	3965.69	kN

7.7.2 Determine the load combinations with applied load modifiers and load factors.

Load modifier for maximum values, η_i 1.05

Load modifier for minimum values, η_i 0.95

Load Combination: STRENGTH 1

Load Factor max	Load Factor min	Load Type	Unfactored load, kN Hor'l	Unfactored load, kN Vert'l	Lever arm (m)	Factored Axial force, kN max	Axial force, kN min	Shear force, kN max	Shear force, kN min	Moment, kN-m max	Moment, kN-m min
1.25	0.90	DC 1	0.00	2800.00	0.80	3500.00	2520.00	0.00	0.00	2800.00	2016.00
1.25	0.90	DC 2.1	0.00	327.60	-0.28	409.50	294.84	0.00	0.00	-114.66	-82.56
1.25	0.90	DC 2.2	0.00	44.10	-0.68	55.13	39.69	0.00	0.00	-37.49	-26.99
1.25	0.90	DC 2.3	0.00	3780.00	0.50	4725.00	3402.00	0.00	0.00	2362.50	1701.00
1.25	0.90	DC 2.4	0.00	3528.00	0.00	4410.00	3175.20	0.00	0.00	0.00	0.00
1.25	0.90	DC 2.5	0.00	977.76	-2.00	1222.20	879.98	0.00	0.00	-2444.40	-1759.97
1.25	0.90	DC 2.6	0.00	403.20	-0.68	504.00	362.88	0.00	0.00	-342.72	-246.76
1.35	1.00	EV	0.00	5985.00	-2.00	8079.75	5985.00	0.00	0.00	-16159.50	-11970.00
1.50	0.65	DW	0.00	150.00	0.80	225.00	97.50	0.00	0.00	180.00	78.00
1.50	0.90	EH	4430.80	0.00	4.00	0.00	0.00	6646.20	3987.72	26584.82	15950.89
1.50	0.75	ES	373.12	0.00	6.00	0.00	0.00	559.68	279.84	3358.08	1679.04
		FR	0.00	0.00	9.50	0.00	0.00	755.63	392.63	7178.44	3729.94
1.75	0	LL	0.00	750.00	0.80	1312.50	0.00	0.00	0.00	1050.00	0.00
1.75	0	BR	110.00	0.00	12.00	0.00	0.00	192.50	0.00	2310.00	0.00
1.75	0	LS	450.46	0.00	6.00	0.00	0.00	788.31	0.00	4729.88	0.00
1.00	0	WA 2	0.00	-3965.69	0.00	-3965.69	0.00	0.00	0.00	0.00	0.00
					(Strength I) Design load:	**21501.25**	**15919.24**	**9389.44**	**4427.18**	**33027.70**	**10515.17**

Load Combination: EXTREME EVENT I
Load Combination: EXTREME EVENT I (CASE 1) (Case1: 100% P_{AE} + 50% P_{IR})

Load Factor max	Load Factor min	Load Type	Unfactored load, kN Hor'l	Unfactored load, kN Vert'l	Lever arm (m)	Factored Axial force, kN max	Axial force, kN min	Shear force, kN max	Shear force, kN min	Moment, kN-m max	Moment, kN-m min
1.25	0.90	DC 1	0.00	2800.00	0.80	3500.00	2520.00	0.00	0.00	2800.00	2016.00
1.25	0.90	DC 2.1	0.00	327.60	-0.24	409.50	294.84	0.00	0.00	-98.28	-70.76
1.25	0.90	DC 2.2	0.00	44.10	-0.68	55.13	39.69	0.00	0.00	-37.49	-26.99
1.25	0.90	DC 2.3	0.00	3780.00	0.50	4725.00	3402.00	0.00	0.00	2362.50	1701.00
1.25	0.90	DC 2.4	0.00	3528.00	0.00	4410.00	3175.20	0.00	0.00	0.00	0.00
1.25	0.90	DC 2.5	0.00	977.76	-2.00	1222.20	879.98	0.00	0.00	-2444.40	-1759.97
1.25	0.90	DC 2.6	0.00	403.20	-0.68	504.00	362.88	0.00	0.00	-342.72	-246.76
1.35	1.00	EV	0.00	5985.00	-2.00	8079.75	5985.00	0.00	0.00	-16159.50	-11970.00
1.50	0.65	DW	0.00	150.00	0.80	225.00	97.50	0.00	0.00	180.00	78.00
1.50	0.75	ES	373.12	0.00	6.00	0.00	0.00	559.68	279.84	3358.08	1679.04
0.50	0.0	LL	0.00	750.00	0.80	375.00	0.00	0.00	0.00	300.00	0.00
0.50	0.0	BR	110.00	0.00	12.00	0.00	0.00	55.00	0.00	660.00	0.00
0.50	0.0	LS	450.46	0.00	6.00	0.00	0.00	225.23	0.00	1351.39	0.00
		FR	0.00	0.00	9.50	0.00	0.00	615.00	392.63	5842.50	3729.94
1.00	0.0	WA 1	0.00	-1442.07	0.00	-1442.07	0.00	0.00	0.00	0.00	0.00
1.00	0.0	P_{AE}	7655.38	0.00	4.00	0.00	0.00	7655.38	0.00	30621.52	0.00
1.00	0.0	50%P_{IR_a}	86.81	0.00	10.75	0.00	0.00	43.41	0.00	466.63	0.00
1.00	0.0	50%P_{IR_b}	11.69	0.00	11.40	0.00	0.00	5.84	0.00	66.61	0.00
1.00	0.0	50%P_{IR_c}	1001.70	0.00	5.75	0.00	0.00	500.85	0.00	2879.89	0.00
1.00	0.0	50%P_{IR_d}	934.92	0.00	1.00	0.00	0.00	467.46	0.00	467.46	0.00
1.00	0.0	50%P_{IR_e}	259.11	0.00	6.85	0.00	0.00	129.55	0.00	887.44	0.00
1.00	0.0	50%P_{IR_f}	106.85	0.00	11.83	0.00	0.00	53.42	0.00	632.01	0.00
1.00	0.0	50%P_{IR_g}	1586.03	0.00	10.00	0.00	0.00	793.01	0.00	7930.13	0.00
					(Extreme Event 1-Case1) Design load:	**22063.51**	**16757.09**	**11103.84**	**672.47**	**41723.77**	**-4870.50**

Commentary

- Application of modifiers are similar to backwall.
- Modifier, η_i

$\eta_D \geq$ 1.05
$\eta_R \geq$ 1.00
$\eta_i \geq$ 1.00

for max. values

η_i = 1/(1.05x1.0x1.0)

η_i = 1.05

for min values

η_i = 1/(1.05x1.0x1.0)

η_i = 0.95

- Note: Friction forces are already factored on section Geometry and Load Calculation.

- Note: Friction forces are already factored on section Geometry and Load Calculation.

- Modifier, η_i

η_i = 1.00

Commentary

Load Combination: EXTREME EVENT I (CASE 2) **(Case2: 50% P_{AE} + 100% P_{IR})**

However, if 50% $P_{AE} < P_A$, use P_A, else use 50%P_{AE} Verification: 50% P_{AE} = 3827.69

P_A =EH = 4430.80 > 50% P_{AE} therefore use EH

Load Factor max	Load Factor min	Load Type	Unfactored load, kN Hor'l	Unfactored load, kN Vert'l	Lever arm (m)	Factored Axial force, kN max	Factored Axial force, kN min	Factored Shear force, kN max	Factored Shear force, kN min	Factored Moment, kN-m max	Factored Moment, kN-m min
1.25	0.90	DC 1	0.00	2800.00	0.80	3500.00	2520.00	0.00	0.00	2800.00	2016.00
1.25	0.90	DC 2.1	0.00	327.60	-0.24	409.50	294.84	0.00	0.00	-98.28	-70.76
1.25	0.90	DC 2.2	0.00	11.69	-0.68	14.61	10.52	0.00	0.00	-9.93	-7.15
1.25	0.90	DC 2.3	0.00	3780.00	0.50	4725.00	3402.00	0.00	0.00	2362.50	1701.00
1.25	0.90	DC 2.4	0.00	3528.00	0.00	4410.00	3175.20	0.00	0.00	0.00	0.00
1.25	0.90	DC 2.5	0.00	977.76	-2.00	1222.20	879.98	0.00	0.00	-2444.40	-1759.97
1.25	0.90	DC 2.6	0.00	403.20	-0.68	504.00	362.88	0.00	0.00	-342.72	-246.76
1.35	1.00	EV	0.00	5985.00	-2.00	8079.75	5985.00	0.00	0.00	-16159.50	-11970.00
1.50	0.65	DW	0.00	150.00	0.80	225.00	97.50	0.00	0.00	180.00	78.00
1.50	0.75	ES	373.12	0.00	6.00	0.00	0.00	559.68	279.84	3358.08	1679.04
0.50	0.0	LL	0.00	750.00	0.80	375.00	0.00	0.00	0.00	300.00	0.00
0.50	0.0	BF	110.00	0.00	12.00	0.00	0.00	55.00	0.00	660.00	0.00
0.50	0.0	LS	450.46	0.00	6.00	0.00	0.00	225.23	0.00	1351.39	0.00
		FR	0.00	0.00	9.50	0.00	0.00	615.00	392.63	5842.50	3729.94
1.00	0.00	WA 1	0.00	-1442.07	0.00	-1442.07	0.00	0.00	0.00	0.00	0.00
1.00	0.00	EH	4430.80	0.00	4.00	0.00	0.00	4430.80	0.00	17723.21	0.00
1.00	0.00	P_{IR}_a	86.81	0.00	10.75	0.00	0.00	86.81	0.00	933.25	0.00
1.00	0.00	P_{IR}_b	11.69	0.00	11.40	0.00	0.00	11.69	0.00	133.23	0.00
1.00	0.00	P_{IR}_c	1001.70	0.00	5.75	0.00	0.00	1001.70	0.00	5759.78	0.00
1.00	0.00	P_{IR}_d	934.92	0.00	1.00	0.00	0.00	934.92	0.00	934.92	0.00
1.00	0.00	P_{IR}_e	259.11	0.00	6.85	0.00	0.00	259.11	0.00	1774.88	0.00
1.00	0.00	P_{IR}_f	106.85	0.00	11.83	0.00	0.00	106.85	0.00	1264.01	0.00
1.00	0.00	P_{IR}_g	1586.03	0.00	10.00	0.00	0.00	1586.03	0.00	15860.25	0.00
(Extreme Event 1-Case 2) Design load:						**22022.99**	**16727.92**	**9872.82**	**672.47**	**42183.17**	**-4850.66**

•Note: Friction forces are already factored on section Geometry and Load Calculation.

•Modifier, ηi

ηi = 1.00

Load Combination: SERVICE 1

Load Factor max	Load Factor min	Load Type	Unfactored load, kN Hor'l	Unfactored load, kN Vert'l	Lever arm (m)	Factored Axial force, kN max	Factored Axial force, kN min	Factored Shear force, kN max	Factored Shear force, kN min	Factored Moment, kN-m max	Factored Moment, kN-m min
1.00	0.00	DC 1	0.00	2800.00	0.80	2800.00	0.00	0.00	0.00	2240.00	0.00
1.00	0.00	DC 2.1	0.00	327.60	-0.24	327.60	0.00	0.00	0.00	-78.62	0.00
1.00	0.00	DC 2.2	0.00	44.10	-0.68	44.10	0.00	0.00	0.00	-29.99	0.00
1.00	0.00	DC 2.3	0.00	3780.00	0.50	3780.00	0.00	0.00	0.00	1890.00	0.00
1.00	0.00	DC 2.4	0.00	3528.00	0.00	3528.00	0.00	0.00	0.00	0.00	0.00
1.00	0.00	DC 2.5	0.00	977.76	-2.00	977.76	0.00	0.00	0.00	-1955.52	0.00
1.00	0.00	DC 2.6	0.00	403.20	-0.68	403.20	0.00	0.00	0.00	-274.18	0.00

Commentary

Load Factor		Load Type	Unfactored load, kN		Lever arm (m)	Factored					
						Axial force, kN		Shear force, kN		Moment, kN-m	
max	min		Hor'l	Vert'l		max	min	max	min	max	min
1.00	0.00	EV	0.00	5985.00	-2.00	5985.00	0.00	0.00	0.00	-11970.00	0.00
1.00	0.00	DW	0.00	150.00	0.80	150.00	0.00	0.00	0.00	120.00	0.00
1.00	0.00	EH	4430.80	0.00	4.00	0.00	0.00	4430.80	0.00	17723.21	0.00
1.00	0.00	ES	373.12	0.00	6.00	0.00	0.00	373.12	0.00	2238.72	0.00
		FR	0.00	0.00	9.50	0.00	0.00	555.00	0.00	5272.50	0.00
1.00	0.00	LL	0.00	750.00	0.80	750.00	0.00	0.00	0.00	600.00	0.00
1.00	0.00	BR	110.00	0.00	12.00	0.00	0.00	110.00	0.00	1320.00	0.00
1.00	0.00	LS	450.46	0.00	6.00	0.00	0.00	450.46	0.00	2702.79	0.00
1.00	0.00	WA 1	0.00	-1442.07	0.00	-1442.07	0.00	0.00	0.00	0.00	0.00
(Service 1) Design load:						**17303.59**	**0.00**	**5919.39**	**0.00**	**19798.91**	**0.00**

•Note: Friction forces are already factored on section Geometry and Load Calculation.

•Modifier, ηi

η_i = 1.00

Summary of Load Combinations:

STRENGTH I						EXTREME EVENT 1 (CASE 1)					
Axial force (kN)		Shear Force (kN)		Moment (kN-m)		Axial force (kN)		Shear Force (kN)		Moment (kN-m)	
max	min	max	min	max	min	max	min	max	min	max	min
21501.25	**15919.24**	**9389.44**	**4427.18**	**33027.70**	**10515.17**	**22063.51**	**16757.09**	**11103.84**	**672.47**	**41723.77**	**-4870.50**

EXTREME EVENT 1 (CASE 2)						SERVICE 1					
Axial force (kN)		Shear Force (kN)		Moment (kN-m)		Axial force (kN)		Shear Force (kN)		Moment (kN-m)	
max	min	max	min	max	min	max	min	max	min	max	min
22022.99	**16727.92**	**9872.82**	**672.47**	**42183.17**	**-4850.66**	**17303.59**	-	**5919.39**	-	**19798.91**	-

•The summary of load combinations show that the Extreme Event 1 (Case 2) is the critical load case.

7.7.3 Determine the governing design forces

Load Combination: EXTREME EVENT I (CASE 2)

Axial force(min) = **16727.92** kN

Axial force(max)= **22022.99** kN

Shear Force = **9872.82** kN

Shear Force = **11103.84** kN ← *Obtain from Case 1 which is slightly larger than Case 2.*

Max. Moment = **42183.17** kN-m

•Note: Analysis shall be evaluated both Pmax and Pmin. Pmin can be critical in the pull-out or tension action.

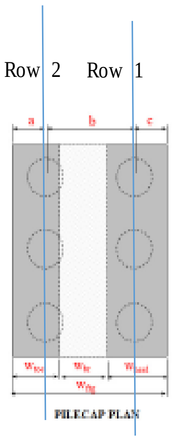
Determination of pile reactions

	No. of piles/row	Mom. of Inertia	Dist. from N.A.
Row 1	3	12	2
Row 2	3	12	2
	I =	24	m^4

**Modified stress formula for piles*

$$F=P/N\pm(Mc)/I \quad \begin{array}{l}(+)\ \textit{For Compression}\\(-)\ \textit{For Tension}\end{array}$$

Design moment, *Mu*	**42183.17**	kN-m		
Distance of neutral axis to piles, *c*				
For row 1, C_1	2	m	a=c=	1.5 m
For row 2, C_2	2	m	wftg =	7 m
Moment of Inertia, *I*	24	m^4	L=	10.5 m
Number of piles, *N*	6			
Reaction per pile:				
(@Axial min.)for piles at row 1, (F_1)	**-727.28**	kN	***Governs!!!***	
(@Axial max.)for piles at row 1, (F_1)	155.23	kN		
(@Axial min.)for piles at row 2, (F_2)	6303.25	kN		
(@Axial max.)for piles at row 2, (F_2)	**7185.76**	kN	***Governs!!!***	



NOTE: For verification, the reactions from pile group analysis may be compared to the reactions obtained from section 7.8 Design of Piles analysis and use the conservative values for design of pile cap.

Commentary

7.7.4 Verification of flexural resistance

Determine design moment for compression side (to design bottom bars)

Passive soil above footing	1.0	m	(estimated depth)
Weight of soil above toe	399	kN	
Weight of toe	1008	kN	
Number of piles at row 2	3		
Total reaction force from piles in row 2	21557.29	kN	(@ compression)
Distance of piles from face of column	0.5	m	
Design moment for bottom bars	9371.64	kN-m	

Determine design moment for tension side (to design top bars bars)

Height of soil above heel	10	m	
Weight of soil above heel	5985	kN	
Weight of heel	1512	kN	
Number of piles at row 1	3		
Total reaction force from piles in row 1	-2181.83	kN	(@ tension)
Distance of piles in row 1 at face of column	1.5	m	
Design moment for top bars	-14518	kN-m	

			Commentary

Design for bottom bars

Demand moment for bottom bars	9371.64	kN-m			
Concrete cover	30	mm			
Pile embedment	100	mm			
Diameter of reinforcing bar	28	mm	Ab =	615.75	
Diameter of shrinkage bar	20	mm			
Diameter of cross ties	16	mm			
Effective depth of concrete	1840	mm			
Length to be considered	10500	mm			
Overall thickness of component	2000	mm			

Minimum reinforcement

•DGCS 12.4.3.3

Flexural cracking variability factor, γ_1	1.6	*for all other concrete	
Ratio of specified min. to ult. tensile strength of steel, γ_3	0.67	*for A615, 414MPa steel	
Modulus of rupture, f_r	3.334	mPa	
Section modulus, S_c	7E+09	mm^3	
Cracking moment, M_{cr}			
$\cdot M_{cr}=\gamma_3(\gamma_1 \cdot fr)\ Sc$	25015.68	kN-m	
*Mu_min 1.33*Md =*	12464.28	kN-m	
Design moment for bottom bars	**12464.28**	kN-m	

Steel ratio

β_1 Coefficient Criterion: 0.85

• DGCS 12.4.2.1

the factor β_1 shall be taken as 0.85 for concrete strengths not exceeding 28MPa. For concrete strength exceeding 28MPa, β_1 shall be reduced at a rate of 0.05 for each 7MPa strength excess of 28MPa but not less than 0.65

For required steel ratio, ρ

m_1 =	0.0573	
m_2 =	0.084	mm^2/N
R_n =	0.390	MPa
ρ =	0.0009	

Computation for main reinforcement

Required steel area	18287.6	mm^2			
Required spacing	353.54	say:	150	mm	
Provided steel for bottom bars, A_s	43102.65	mm^2			
Compression fiber to neutral axis, c	84.21	mm			
Depth of compression block, a	71.58	mm			
Nominal moment capacity of section, M_n	32273.00	kN-m			

• Conservative spacing is assumed to satisfy other design requirements.

Commentary

Resistance factor, Ø 0.9 tension is controlled

$0.75 \leq \phi = 0.65 + 0.15\,(d_{eff}/c - 1) \leq 0.9$ 3.78

Ultimate moment capacity of section, $ØM_n$ **29045.70** kN-m **OK!**

c/d = 2.33

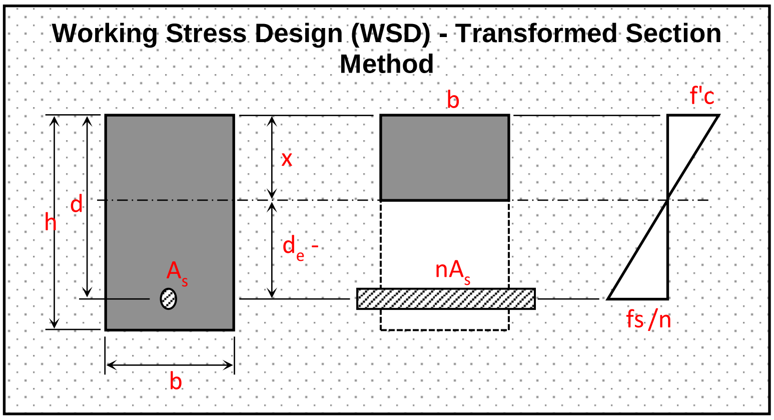
Control of cracking by distribution of reinforcement

• DGCS 12.4.3.4

Applies to all reinforcements of concrete that exceeds 80% of the modulus of rupture, except deck slabs.

Ultimate moment from service loads, M_s	19798.91	kN-m
80% of Modulus of rupture, f_r	2.201	mPa
Tension in the cross section, fss	2.828	mPa

Tension in the cross-section exceeds 80% of the modulus of rupture, this provision has to be satisfied



Extreme tension fiber to center of flexural reinforcement, d_c	144	mm
Overall thickness of component, h	2000	mm
Compression fiber to the centroid of extreme tension steel, d_e	1856	mm
Neutral axis to extreme compression fiber, x	318.60	mm
Modulus elasticity of steel, E_s	200	GPa
Modulus elasticity of concrete, E_c	24.87	GPa
Modular ratio, n	8.042	
Cracked section moment of inertia of section, I_{NA}	9.3247E+11	mm^4
Exposure factor, γ_e	1.00	
Exposure condition: Class 1		
Tensile stress in steel reinforcement at the service limit, f_s	249	MPa

•by quardratic equation to determine x:

a = 10500
b = 693245.568
c = -1.287E+09

x_1 = 318.60
x_1 = -384.62

$\beta s = 1 + d_c / (0.7\,(h - dc))$ 1.11

The spacing shall satisfy: $s \leq 123000\gamma_e / \beta_s f_{ss} - 2dc$ 156.69 mm

Initial spacing: **150** mm **SATISFIED!**

Using of 28mm Ø main bars spaced at 150mm O.C. is adequate and safe

Design for top bars			Commentary
Design for top bars			
Demand moment for top bars	14518.25	kN-m	
Concrete cover	75	mm	
Diameter of reinforcing bar	28	mm Ab = 615.752	•concrete cover is from top of footing.
Diameter of shrinkage bar	20	mm	
Diameter of cross ties	16	mm	
Effective depth of concrete	1895	mm	
Length to be considered	10500	mm	
Overall thickness of component	2000	mm	
Minimum reinforcement			•DGCS 12.4.3.3
Flexural cracking variability factor, γ_1	1.6	**for all other concrete*	
Ratio of specified min. to ult. tensile strength of steel, γ_3	0.67	**for A615, 414MPa steel*	
Modulus of rupture, F_r	3.334	mPa	
Section modulus, S_c	7000000000	mm^3	
Cracking moment, M_{cr}			
$\cdot M_{cr}=\gamma_3(\gamma_1 \cdot fr)\ Sc$	25015.68	kN-m	
*Mu_min 1.33*Md =*	19309.27	kN-m	
Design moment for bottom bars	19309.27	kN-m	
Steel ratio			
β_1 Coefficient Criterion:	0.85		
the factor β_1 shall be taken as 0.85 for concrete strengths not exceeding 28MPa. For concrete strength exceeding 28MPa, β_1 shall be reduced at a rate of 0.05 for each 7MPa strength excess of 28MPa but not less than 0.65			
For required steel ratio, ρ	m_1 = 0.0573		
	m_2 = 0.084	mm^2/N	
	R_n = 0.569	mPa	
	ρ = 0.0014		
Computation for main reinforcement			
Required steel area	27615.51	mm^2	
Required spacing	234.12	say: 150 mm	
Provided steel for breast wall, A_s	43102.65	mm^2	
Compression fiber to neutral axis, c	84.21	mm	
Depth of compression block, a	71.58	mm	
Nominal moment capacity of section, M_n	33256.81	kN-m	
Resistance factor, Ø	0.9	tension is controlled	
$\cdot 0.75 \leq \phi = 0.65+0.15\,(d_{eff}/c-1)\ \leq 0.9$ 3.88			
Ultimate moment capacity of section, $ØM_n$	**29931.13**	kN-m **OK!**	
	c/d =	1.55	

Commentary

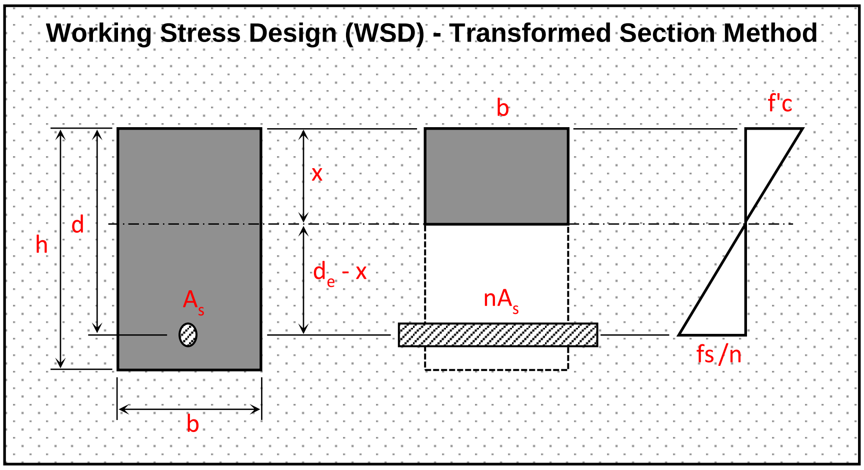
• DGCS 12.4.3.4

Control of cracking by distribution of reinforcement

Applies to all reinforcements of concrete that exceeds 80% of the modulus of rupture, except deck slabs.

Ultimate moment from service loads, M_s	19798.91	kN-m
80% of Modulus of rupture, f_r	2.201	mPa
Tension in the cross section	2.828	mPa

Tension in the cross-section exceeds 80% of the modulus of rupture, this provision has to be satisfied



Extreme tension fiber to center of flexural reinforcement, d_c	89	mm
Overall thickness of component, h	2000	mm
Compression fiber to the centroid of extreme tension steel, d_e	1911	mm
Neutral axis to extreme compression fiber, x	323.72	mm
Modulus elasticity of steel, E_s	200	GPa
Modulus elasticity of concrete, E_c	24.87	GPa
Modular ratio, n	8.042	
Cracked section moment of inertia of section, I_{NA}	9.9204E+11	mm^4
Exposure factor, Υ_e	1.00	
Exposure condition: Class 1		
Tensile stress in steel reinforcement at the service limit, f_s	249	mPa

•by quardratic equation to determine x:

a = 10500
b = 693245.568
c = -1.325E+09

x_1 = 323.724154
x_1 = -389.74754

$$\beta s = 1 + d_c/(0.7\,(h - dc)) \qquad 1.07$$

The spacing shall satisfy: $s \leq 123000\gamma_e/\beta_s f_{ss} - 2dc$ 285.161 mm

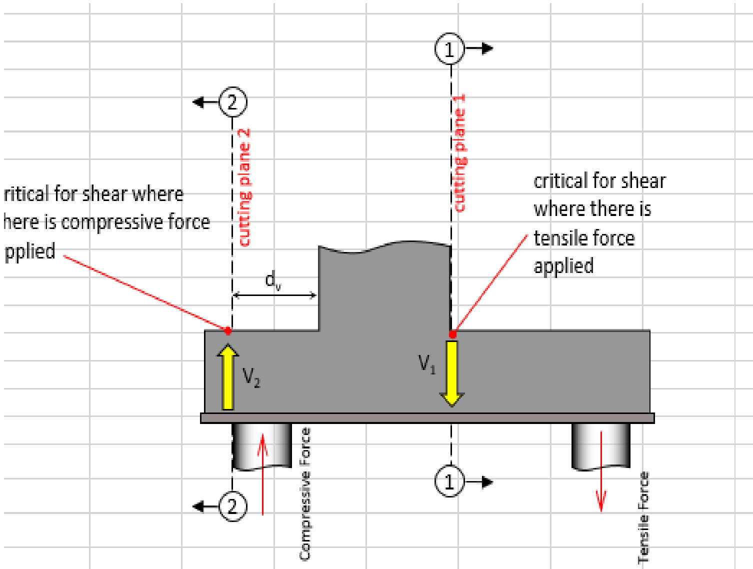
Initial spacing: 150 mm **SATISFIED!**

•for detailing purposes, it is recommended to adopt the conservative spacings of bars for top and bottom bars.

Using of 28mm Ø main bars spaced at 150mm O.C. is adequate and safe

7.7.5 Verification of shear resistance

	Commentary
a. Nominal resistance for one-way action	• DGCS 12.5.3.2



For section 1 - 1

Reaction force of single pile in row 1, F_1	-727.28	kN
Total force acting on tension side	-9678.83	kN

**Tension governs; critical for shear is at face of column*

For section 2 - 2

Reaction force of single pile in row 2, F_2	7185.76	kN
Total force acting on compression side	20150.29	kN

**Compression governs; Critical for shear is at distance dv*

Total force acting on section 2-2 — 3631.94 kN

**Assuming 1/2 demand shear force is effective*

• by inspection, compression side is not critical because shear action falls outside the critical shear.

Ultimate shear based on demand, V_u — 9678.83 kN — ***Governs!!!***

Effective shear depth, d_v — 1804.21 mm

Taken as the distance measured perpendicular to the neutral axis, between the resultants of the tensile and compressive forces due to flexure; it need not to be taken to be less than the greater of 0.9de or 0.72h

• DGCS 12.5.3.3.2

Factor indicating ability of diagonally cracked concrete to transmit tension, — 2.75

Solution for β: **GENERAL PROCEDURE**

Commentary

Area of prestressing steel on tension side, A_{ps}	0	mm^2	
Area of non-prestressing steel, A_s	43102.65	mm^2	
Maximum aggregate size, a_g	20	mm	
Modulus of elasticity of prestressing tendons, f_{po}	0	mPa	
Factored axial force, N_u	-22022988	N	
**Positive for tension; Negative for compression*			
Factored shear force, V_u	9872815	N	
Absolut value of the factored moment, $\|M_u\|$	4.2183E+10	N-mm	
**But not less than $\|V_u - V_p\| d_v$*			
Modulus of elasticity of prestressing steel, E_p	0	GPa	
Modulus of elasticity of steel, E_s	200	GPa	0.00258
Net longitudinal tensile strain, e_s	0.001		
Crack spacing parameter, S_{xe}	300	mm	
Angle of inclination of diagonal compressive stresses, q	32.5	deg.	
Shear resistance from steel, V_s	0	kN	
Effective prestressing force, V_p	0	kN	
Shear resistance provided by concrete, V_c	22905.99	kN	
The nominal shear resistance, V_n	22905.98805	kN	
**shall be determined as the lesser of:*			
$V_n=Vc+Vs+Vp$	22905.99	kN	
$V_n=0.25fc^{\wedge\prime}\, b_v d_v+Vp$	132609.4706	kN	
Resistance factor for normal weight concrete,∅	0.9		
Ultimate shear capacity of section, $\emptyset V_n$	**20615.39**	kN	**OK!**
	c/d =	2.13	

Section without shear reinforcement is adequate and safe

b. Nominal shear resistance for two-way action

•DGCS 12.10.3.5

For two-way action for sections without transverse reinforcement, the nominal shear resistance, V_n of the concrete shall be taken as:

$$V_n=(0.17+0.33/\beta_{c)}\sqrt{(fc')}\ bodv \leq 0.33\sqrt{(fc')}\ bodv$$

Effective shear depth of footing, d_v	1804.21	mm
Ratio of long to short direction of pile, b_c	1	
Perimeter of the critical section, b_o	9438.01	mm
Maximum reaction of single pile, V_u	7185.76	kN
Nominal shear capacity of concrete, V_n	33080.30	kN
$a)\ Vn=(0.17+0.33/\beta_{c)}\sqrt{(fc')}\ bodv$	50121.67	kN
$b)\ Vn=0.33\sqrt{(fc')}\ bodv$	33080.30	kN

				Commentary
Resistance factor for normal weight concrete, Ø	0.90			
Ultimate shear capacity of section, $ØV_n$	29772.27	kN	OK!	
Pile cap is adequate for shear due to maximum reaction of pile				

7.7.6 Verification of shrinkage and temperature reinforcement

Reinforcement for shrinkage and temperature stresses shall be provided near surfaces of concrete exposed to daily temperaturechanges and in structural mass concrete.

•DGCS 12.7.8

Diameter of shrinkage and temperature bar	20	mm
Assumed spacing, *S*	200	mm
Assumed shrinkage and temperature reinforcement, A_s	1.57	mm^2/mm

Shrinkage and temperature reinforcement shall satisfy:

a) $As \geq (0.75\, bh)/(2\,(b+h) f_y)$	1.57	mm^2/mm
b) $0.233 \leq As \leq 1.27$	1.27	mm^2/mm

Spacing shall not exceed:

a) 3.0 times the component thickness, or 450 mm

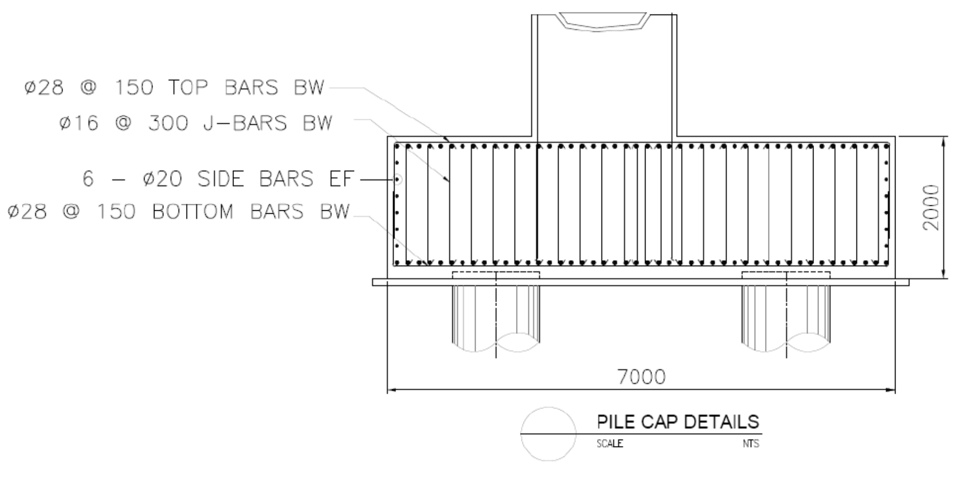
b) 300 mm for walls and footings greater than 450 mm thick

c) 300 mm for other components greather than 900 mm thick

Final shrinkage and temperature reinforcement, A_s	1270	mm^2 per meter
Final spacing to be used	247	mm
say:	200	mm

Therefore use 20mm Ø for temperature and shrinkage bar spaced at 200mm O.C.

7.7.7 Pile cap details



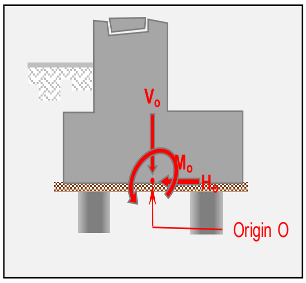
7.8 DESIGN OF PILES

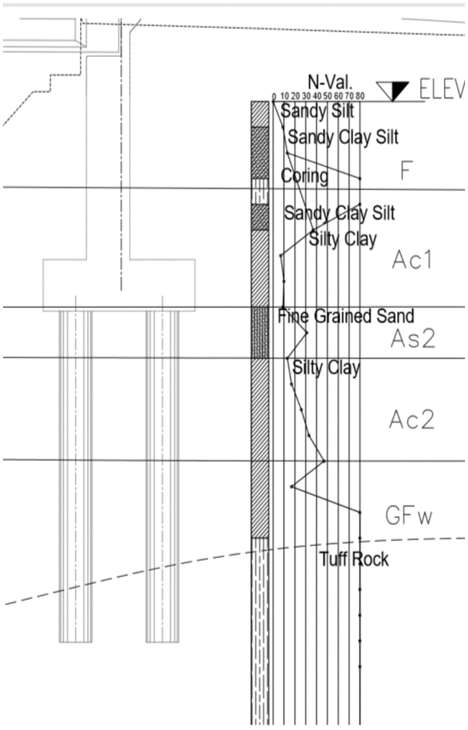
7.8.1 Determine the pile springs and geometric properties

Design forces in Longitudinal Direction			
Governing Load Case			
Extreme Event I Limit State -Case2			
Forces acting at the bottom of pile cap/footing			
Location	Vertical Load	Lateral Load	Moment
	V_o	H_o	M_o
Origin O	kN	kN	kN-m
	16727.92	11103.84	42183.17

Pmax = 22022.99

Note: Piles shall be evaluated both Pmax and Pmin. Pmin is critical in the pull out action.





BH 4 - ABUT B

Commentary

•The investigation of stability of piles is basically based on JRA method. Refer to BSDS sections 4 and 5.

•Vo, Ho, Mo = elastic forces for Abutment design

•Borelog of Abut B is the basis of subsurface data.

7.8.2 Determine the pile springs and geometric properties

a. Horizontal pile spring constant (K_H)

Note: The coefficient of subgrade reaction shall be determined, in principle, by using the modulus of deformation obtained from a variety of surveys and tests by considering the influence of loading width of foundations and other relevant factors:

BSDS Table C.4.4.2-1 Modulus of Deformation E_0 and *a*			
Modulus of deformation E_0 to be obtained by means of the following testing methods		*a*	
Method	Definition	Ordinary	Earthquake
Method A	A value equal to 0.5 of the modulus of deformation to be obtained from a repetitive curve of a plate bearing test using a rigid disc of 30cm. in diameter.	1	2
Method B	Modulus of deformation to be measured in the bore hole.	4	8
Method C	Modulus of deformation to be obtained by means of an unconfined or triaxial compression test of samples.	4	8
Method D	Modulus of deformation to be estimated from E_0 = 2,800*N using the N-value of the standard penetration test.	1	2

<<<< to be used

•JRA method determines the reaction and soil spring constant to model the foundation.

Commentary

The coefficient of horizontal subgrade reaction should be obtained by using BSDS Equation C.4.4.2-4 :

$$k_H = k_{HO} \left(\frac{B_H}{0.3} \right)^{-3/4}$$

BSDS Eq. C.4.4.2-4

where :

k_H	coefficient of horizontal subgrade reaction (kN/m^3)
k_{HO}	coefficient of horizontal subgrade reaction coresponding to the value obtained by the plate bearing test using a rigid disc of diameter 0.3m, k_{HO} = (a*E_0/0.3) (kN/m^3).
B_H	equivalent loading width of foundation to be obtained from BSDS Table C.4.4.2-2 (m)
E_0	modulus of deformation at the design location, measured or estimated by the procedures in Table C.4.4.2-1
A_H	loading area of foundation perpendicular to the load direction (m^2)
D	loading width of foundation perpendicular to the load direction (m)
B_e	effective loading width of foundation perpendicular to the load direction (m)
L_e	effective embedment depth of a foundation (m)
$1/b$	ground depth relating to the horizontal resistance and equal to or less than the effective embedment depth (m)
b	characteristic value of foundation
EI	flexural stiffness of foundation (kN-m^2)

$$K_{HP} = k_H A_{HP}$$

BSDS Eq. C4.4.3-9

where :

K_{HP}	horizontal spring constant of pile section corresponding to area A_{HP} (kN/m)
A_{HP}	effective projected vertical area of the ground corresponding to pile spring K_{HP} (m2)

When analyzing the ground resistance of a pile foundation as a linear spring, the equivalent loading width B_H should take a value of $(D/b)^{1/2}$.

b. Geometric properties of piles

Select Pile Section :

- ☑ Circular Section
- ☐ Square Section

Select Pile Installation Method :

- ☐ Driven Piles (Blow Method)
- ☐ Driven Piles (Vibro-Hammer Method)
- ☑ Cast-in-place RC Piles
- ☐ Bored Piles
- ☐ Pre-Boring Piles
- ☐ Steel Pile Soil Cement Piles

•Note: Cast-in-place RC pile is the common use in the Philippines. The bored piles refers to Japan's method.

Input Pile Dimension : Diameter	1.20	m
Input Number of Piles	6	piles
Input Pile Length :	16.00	m
Calculate Section Properties :		
Cross-section Area	1.131	m^2
Perimeter of Pile :	3.770	m
Pile Moment of Inertia :	0.102	m^4
Pile Flexural Stiffness :	2.75E+06	$kN\text{-}m^2$
Concrete Material Properties :		
Design Compressive Strength at 28th day	28	N/mm^2
Unit Density for Concrete	2400	kg/m^3
Unit weight for Reinforced Concrete	24	kN/m^3
Young's Modulus of Elasticity	2.70E+07	kN/m^2
Reinforcement Material Properties :		
Minimum Yield Strength	415	N/mm^2
Ultimate Tensile Strength	620	N/mm^2
Young's Modulus of Elasticity	2.00E+08	kN/m^2

Method used to determine Modulus of Deformation :	**Method D**	
Specify Limit State used in determining subgrade coeff.	**Ordinary Condition**	
Coefficient to be used for estimating subgrade raction :	1	
Unit weight of water	10	kN/m^3

Soil Layer Type	Layer Thickness	N-Value	Unit weight		aE_0	$(1/b_1)\text{-}d_i$	t_i	$aE_0{*}t_i$	Layer Depth
			γ_t	γ'					
	m	Average			kN/m^2	m	m	kN/m	m
Clay	2.00	12	18.0	8.0	33600	1.618	2.000	67200.00	2.00
Sand	2.40	40	19.0	9.0	112000	-0.782	1.618	181161.45	4.40
Clay	6.50	15	19.0	9.0	42000	-7.282	0.000	0.00	10.90
Rock	4.40	25	20.0	10.0	70000	-11.682	0.000	0.00	15.30
Rock	1.00	50	20.0	10.0	140000	-12.682	0.000	0	16.30

Commentary

•The initial pile length is to founded into hard strata minimum of 1.m depth.

BSDS Table C4.4.2-1

BSDS Table C4.4.2-1

Assumption on Effective range of Horizontal Subgrade Reaction, $1/b_1$ 3.618 m

Note:

Typically it is 4 to 6 times as large as pile diameter

Iterate value of $1/b_1$ until becomes equa Iterate

Try Between	
4D	6D
m	m
4.8	7.2

Initial Equivalent Loading Width, B_{H1} 2.084 m

Average value of Modulus of Deformation aE_0 within effective ran 68655.3 kN/m²

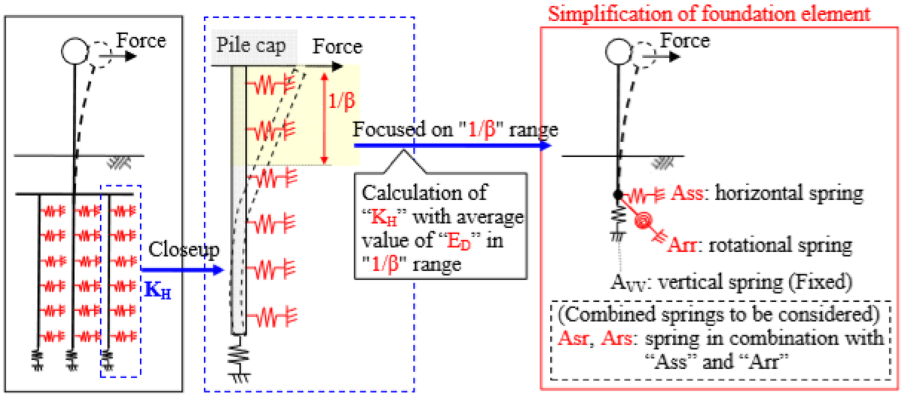
Coefficient of Horizontal Subgrade Reaction, K_H 53493.01 kN/m³

53493.01 kN/m³ ← Coefficient of Horizontal Subgrade Reaction, K_H (Based from derived Equation)

Characteristic Value of Foundation, b 0.276 m⁻¹

iteration until 1/b1 = *1/b* 3.618 m

Horizontal Pile Spring Constant, k_H 232213.98 kN/m



c. Axial pile spring constant (K_V)

The axial spring constant K_V of a single pile use for design shall be estimated from the empirical formula derived from the vertical pile loading test and results of soil test, or from load-settlement curves from vertical loading test .

$$K_V = \frac{a * A_p * E_P}{L}$$

where:

- K_V axial spring constant of pile (kN/m)
- a proportional coefficient (BSDS Equation C5.4.3.6-3)
- A_p net cross-sectional area of pile (m²)
- E_p Young's modulus of pile (kN/m²)
- L pile length (m)
- D pile diameter (m)

Commentary

•By JRA method the horizontal springs approximately between 4D to 6D are lumped to simplify the subgrades. Iterate the value l/b1 until equal to l/b.

Commentary

Embedment Ratio, L/D 13.33

Note:

For Piles L/D<10, L/D=10

Proportional Coefficient, $\alpha = 0.031$ (L/D) - 0.15 0.263

Axial Spring Constant of Pile, K_V 502576.28 kN/m

d. Radial pile spring constant (K_1, K_2, K_3, K_4)

The radial spring constants K_1 to K_4 of a pile are:

K_1, K_3 radial force and bending moment (kN-m/m) to be applied on a pile head when displacing a unit displacement in the radial direction while keeping it from rotating (kN/m)

K_2, K_4 radial force and bending moment (kN-m/rad) to be applied on a pile head when rotating the head by a unit rotation in the radial direction while keeping it from moving in a radial direction (kN/rad)

NOTE: If the coefficient of horizontal subgrade reaction is constant irrespective of the depths and if the embedded depth of a pile is sufficiently long, the constants can be computed from BSDS Table C4.4.3-2.

Specify Limit State used in design : **During Earthquake**

Coefficient to be used, a : 2

Characteristic value of foundation, b' : 0.329 m^{-1}

Pile length above design ground surface, h : 0 m

$b'*L_e$: 5.26 Piles with semi-infinite length

Select restrictive condition of pile hea

☑ Rigid Frame of Pile Head

☐ Hinged Frame of Pile Head

Radial Spring Constants of Piles, K_1 : 390535.81 kN/m

Radial Spring Constants of Piles, K_2 : 593995.91 kN-m/m

Radial Spring Constants of Piles, K_3 : 593995.91 kN/rad

Radial Spring Constants of Piles, K_4 : 1806908.04 kN-m/rad

BSDS Table C4.4.3-2 - Hayashi Chang Theory

Rigid		Hinged	
1171609.42	390535.81	195267.90	195267.90
1781990.78	593995.91	0.00	0.00
1781990.78	593995.91	0.00	0.00
1806908.04	1806908.04	0.00	0.00

7.8.3 Determine the pile displacement and reaction force

Pile reactions and displacements shall be evaluated considering the properties of the pile structure and the ground. In the displacement method, the coordinate is formed with the origin set at an arbitrary point O of the foundation. The origin O may be selected from arbitrary points, but it is recommended to coincide it with the centroid of the pile group.

Commentary

$$\left.\begin{aligned} A_{xx}*d_x + A_{xy}*d_y + A_{xa}*a &= H_o \\ A_{yx}*d_x + A_{yy}*d_y + A_{ya}*a &= V_o \\ A_{ax}*d_x + A_{ay}*d_y + A_{aa}*a &= M_o \end{aligned}\right\}$$

where:

H_o	lateral loads acting at the bottom of pil	dx	lateral displacement from origin O, m
V_o	vertical loads acting at the bottom of p	dy	vertical displacement form origin O, m
M_o	moment (external force) at the origin (	a	rotational angle of the footing at the origin O, rad

The displacements (dx, dy, and a) below are derived by solving BSDS

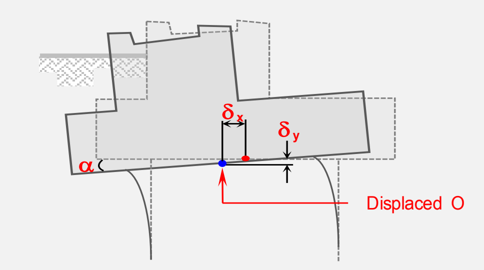
$$\left.\begin{aligned} d_x &= \frac{H_o*A_{aa} - M_o*A_{xa}}{A_{xx}*A_{aa} - A_{xa}*A_{ax}} \\ d_y &= \frac{V_o}{A_{yy}} \\ a &= \frac{-H_o*A_{ax} + M_o*A_{xx}}{A_{xx}*A_{aa} - A_{xa}*A_{ax}} \end{aligned}\right\}$$

BSDS C5.4.3.7-2 COEFFICIENTS FOR DISPLACEMENT CALCULATION

Row	No. of Piles	x_i	q_i	Ayy	Axx	Axa	Aax	Aaa	cos(q_i)	sin(q_i)
1	3	2	0	1507729	1171607	-1781988	-2E+06	11451640	1	0
2	3	2	0	1507729	1171607	-1781988	-2E+06	11451640	1	0
									1	0
									1	0
									1	0
									1	0
									1	0
			Sum =	3015458	2343215	-3563975	-3563975	22903279		

a. Calculation for Displacement:

Longitudinal Displacement			
For Governing Load Case			
Extreme Event I - case 2			
Location	Displacement		
	Lateral	Vertical	Rotational
	d_x	d_y	a
Origin O	m	m	rad
	0.0099	0.0055	0.0034



Deformed Shape

Commentary

b. Calculation of Reaction:

By using the displacements at the footing origin O obtained from the results of the above calculations, the pile axial force, radial force, and moment acting on each pile head can be obtained using the following equations:

$$\left.\begin{aligned} P_{Ni} &= K_V * d_{yi}' \\ P_{Hi} &= K1 * d_{xi}' - K_2 * a \\ M_{ti} &= -K_3 * d_{xi}' + K_4 * a \end{aligned}\right\}$$

BSDS Eq. C5.4.3.7-4

$$\left.\begin{aligned} d_{xi}' &= d_x * cosqi - (d_y + ax_i) * sinq_i \\ d_{yi}' &= d_x * sinq_i + (d_y + ax_i) * cosq_i \end{aligned}\right\}$$

BSDS Eq. C5.4.3.7-5

where:

d_{xi}'	radial displacement at the i-th pile head, m
d_{yi}'	axial displacement at the i-th pile head, m
x_i	x-coordinate of the i-th pile head, m
q_i	vertical axis angle from the i-th pile axis for battered pile, degree
P_{Ni}	axial force of the i-th pile, kN
P_{Hi}	radial force of the i-th pile, kN
M_{ti}	moment as external force acting on the i-th pile head, kN-m

Pile Reaction in Longitudinal Direction

Column	Number of Piles	x_i	q_i	Axial P_{Ni}	Radial P_{Hi}	Moment M_{ti}	$\cos(q_i)$	$\sin(q_i)$	
		m	deg.	kN	kN	kN-m			
									-608.3258
1	3	-2.00	0	-608.33	1850.64	237.90	1	0	6184.3
2	3	2.00	0	6184.30	1850.64	237.90	1	0	2787.987
							1	0	
							1	0	
							1	0	
							1	0	
							1	0	
	6								

Maximum Axial Force for Capacity verification, $P_{Ni\text{-}max}$ **6184.30** kN

Minimum Axial Force for Capacity verification, $P_{Ni\text{-}min}$ **-608.33** kN

•By comparison of the axial forces to the results from pile group analysis in the pile cap design, it appears the design of pile cap is practically conservative, hence no need to redesign of the pile cap.

Commentary

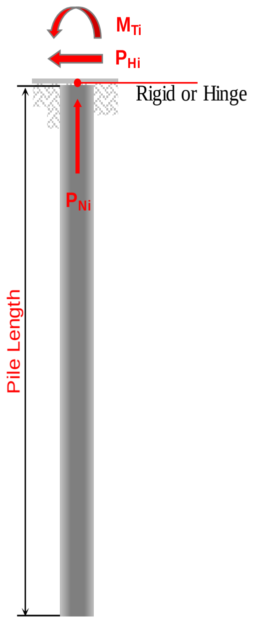
Graphing of Reaction Force and Displacement of each pile

PILE EMBEDDED IN THE GROUND (h = 0)						
Depth	Rigid Pile Head Connection			Hinged Pile Head Connection		
	Deflection	Moment	Shear	Deflection	Moment	Shear
m	mm	kN-m	kN	mm	kN-m	kN
0.00	9.88	-237.90	-1850.64	9.48	0.00	-1850.64
1.00	6.64	-1525.65	-794.39	6.46	-1308.29	-830.74
2.00	3.92	-1955.55	-123.48	3.89	-1782.64	-173.01
3.00	1.91	-1874.04	243.37	1.95	-1751.07	194.71
4.00	0.57	-1540.23	395.59	0.64	-1462.28	354.96
5.00	-0.22	-1127.63	412.91	-0.13	-1085.12	382.76
6.00	-0.59	-738.39	357.65	-0.52	-720.87	337.62
7.00	-0.69	-421.82	273.37	-0.63	-419.97	261.70
8.00	-0.63	-192.16	187.16	-0.60	-198.71	181.64
9.00	-0.51	-43.26	113.38	-0.48	-53.15	111.90
10.00	-0.36	40.63	57.50	-0.35	30.54	58.35
11.00	-0.23	77.77	19.61	-0.23	69.16	21.53
12.00	-0.13	84.92	-3.08	-0.13	78.41	-0.90
13.00	-0.05	75.41	-14.36	-0.06	71.00	-12.39
14.00	-0.01	58.75	-17.97	-0.01	56.12	-16.41
15.00	0.02	40.98	-17.04	0.01	39.67	-15.94
16.00	0.03	25.39	-13.91	0.03	24.98	-13.22
17.00	0.03	13.37	-10.12	0.03	13.48	-9.75
18.00	0.03	5.05	-6.59	0.02	5.42	-6.44
19.00	0.02	-0.04	-3.73	0.02	0.41	-3.72
20.00	0.01	-2.67	-1.67	0.01	-2.26	-1.73
21.00	0.01	-3.62	-0.34	0.01	-3.29	-0.43
22.00	0.00	-3.55	0.40	0.00	-3.31	0.31
23.00	0.00	-2.96	0.72	0.00	-2.80	0.64
24.00	0.00	-2.19	0.78	0.00	-2.11	0.72
25.00	0.00	-1.46	0.68	0.00	-1.42	0.64
26.00	0.00	-0.85	0.53	0.00	-0.84	0.51
27.00	0.00	-0.40	0.37	0.00	-0.41	0.36
28.00	0.00	-0.11	0.23	0.00	-0.12	0.22
29.00	0.00	0.06	0.12	0.00	0.04	0.12
30.00	0.00	0.14	0.04	0.00	0.12	0.05
31.00	0.00	0.16	0.00	0.00	0.15	0.00
32.00	0.00	0.14	-0.03	0.00	0.14	-0.02
33.00	0.00	0.11	-0.03	0.00	0.11	-0.03
34.00	0.00	0.08	-0.03	0.00	0.08	-0.03
35.00	0.00	0.05	-0.03	0.00	0.05	-0.03
36.00	0.00	0.03	-0.02	0.00	0.03	-0.02
37.00	0.00	0.01	-0.01	0.00	0.01	-0.01
38.00	0.00	0.00	-0.01	0.00	0.00	-0.01
39.00	0.00	0.00	0.00	0.00	0.00	0.00
40.00	0.00	-0.01	0.00	0.00	-0.01	0.00

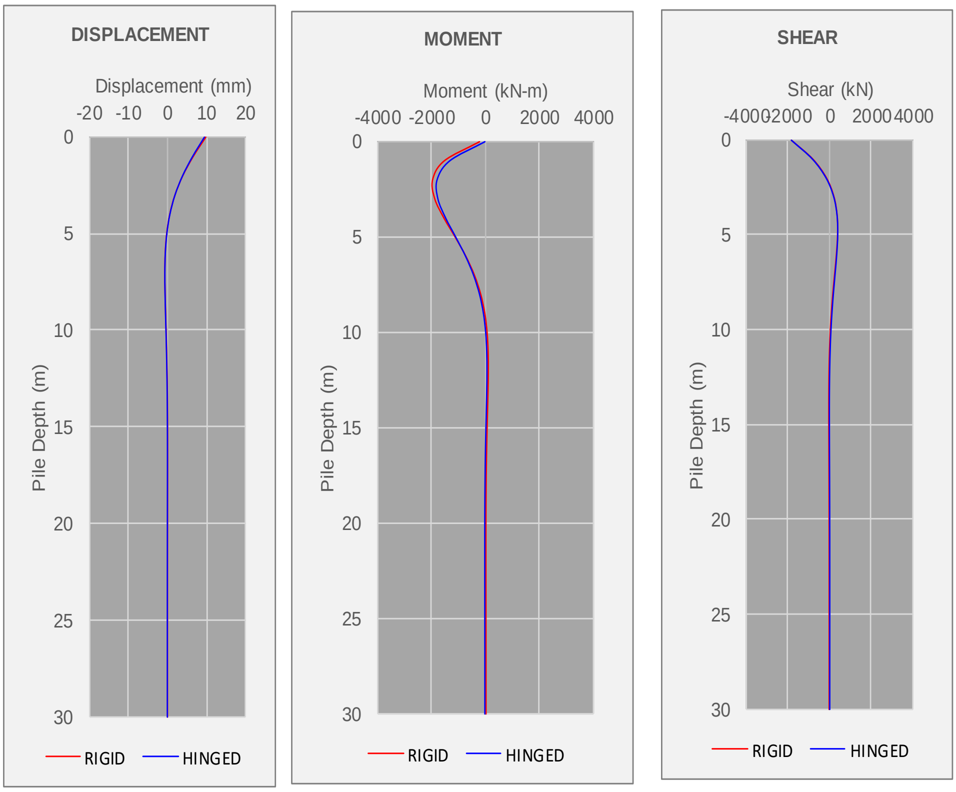
Parameters		
Mt	237.902	kN-m
Ph	1850.640	kN
E	2.700E+07	kN/m^2
I	0.102	m^4
b'	0.329	m^{-1}
ho	0.129	m
l_m	2.266	m

-237.902

-1971.502



Commentary



Reaction Force and Displacement of Single Pile

7.8.4 Verification of Pile stability

a. The factored resistance of piles shall be taken as :

$$R_R = \Upsilon(\emptyset R_n - Ws) + W_s - W$$

BSDS Eq. 5.4.3.3-1

where:

R_R	factored resitance of pile, kN
R_n	nominal resistance of pile, kN
W_s	effective weight of soil replaced by pile, kN
W	effective weight of pile and soil inside pile, kN
$\emptyset$	resistance factor for pile under extreme event limit state 0.65-BSDS Article 5.4.1(5)
Υ	modification coefficient depending on nominal bearing resistance 1.00-BSDS Table 5.4.3.3-1

b. The nominal bearing capacity can be obtained from the empirical bearing capacity estimation formula:

$$R_n = q_d A_p + U\Sigma L_i f_i$$

BSDS Eq. C5.4.3.3-1

where:

R_n	nominal bearing capacity of pile, kN
A_p	area of pile tip
q_d	nominal end bearing resistance intensity per unit area, kN/m^2
U	perimeter of pile
L_i	thickness of soil layer considering shaft resistance, m
f_i	maximum shaft resistance of soil layer considering pile shaft resistance, kN/m^2

c. The factored axial pull-out resistance of a single pile shall be obtained considering soil conditions and construction methods:

$$P_R = \varnothing P_n + W$$

Commentary: BSDS Eq. 5.4.3.4-1

where:

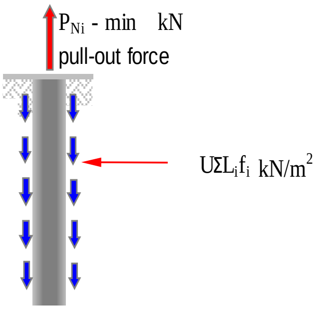
P_R	factored axial pull-out resistance of pile, kN
P_n	nominal axial pull-out resistance, kN
W	effective weight of pile, kN
$\varnothing$	resistance factore for pile under extreme event limit state 0.5 -BSDS Article 5.4.1(5)

d. Estimation of Nominal End Bearing Resistance Intensity (qd)

For Cast-in-place RC Piles : nominal end bearing resistance intensity 5000 kN/m^2

On the basis of the recent results of loading tests on cast-in-place RC piles, the nominal end bearing resistance intensity may take the value of 5,000 kN/m2, when a fully hardened sturdy gravelly ground with an N value of 50 or larger and with a thickness of 5m or greater is selected as supporting layer.

e. Estimation of Shaft Resistance Intensity fi acting on Pile Skin



Cast-in-place RC Piles

For Sandy Soil:	5N ($\leq$ 200)
For Cohesive Soil:	c or 10N ($\leq$ 150)

N-th Layer	Soil Layer Type	Layer Thickness Li (m)	N-Value (Average)	Υ' (kN/m^3)	$L_i*\Upsilon'*A_p$ (kN)	fi (kN/m^2)	DE	$U*L_i*f_i*DE$ (kN/m^2)
1	Clay	2.000	12	8.0	18.10	120	1	904.78
2	Sand	2.400	40	9.0	24.43	200	1	1809.56
3	Clay	6.500	15	9.0	66.16	150	1	3675.66
4	Rock	4.400	25	10.0	49.76	150	1	2488.14
5	Rock	1.000	50	10.0	11.31	150	1	565.49

DE = is the factor for liquefaction. DE = 1 (no reduction of skin fricion. Liquefaction has been assessed separately. It has been concluded that no liquefaction potential in the specific site.

Effective weight of soil to be replaced by the pile,Ws 169.76 kN
Effective weight of the pile with soil inside, W 253.34 kN
Nominal skin friction of pile 9443.63 kN

Result of Nominal Bearing Capacity of Single Pile, R_n : 15098.49 kN
Result of Factored Resistance of Single Pile,R_R : **9560.68** kN
Result of Factored Axial Pull-out Resistance of Single Pile, P_R **-4975.15** kN

Verification for Lateral Displacement

Displacement Demand (mm)	Displacement Capacity (mm)	C/D	Verification
9.88	12	1.21	OK

Capacity =1% x Dpile. See commentary

Verification for Maximum Axial Resistance

Axial Load Demand (kN)	Axial Load Capacity (kN)	C/D	Verification
6184.30	9560.68	1.55	OK

By checking Pmax= 22022.99 kN
PNi-max= 7186.76 kN
PNi-min = 727.28 kN

Verification for Maximum Axial Pull-out Resistance.

Axial Pull-out Demand (kN)	Axial Pull-out Capacity (kN)	C/D	Verificatio
-608.33	-4975.15	8.18	OK

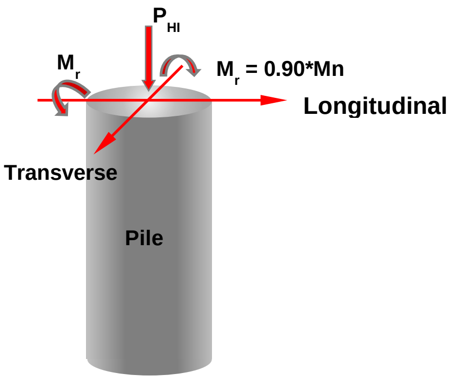
Commentary

BSDS pp 4-15: "In JRA, the reference displacemnt at the linear range is recommended to be one percent(1%) of the foundation width (<= 50mm), which is taken as the allowable displacemen required from the substructure. However, under earthquake loading this value is taken as a reference and may not necessarily be adhered to and may reach as much as 5% of the foundation width."

7.8.5 Verification of Pile structural resistance

Maximum design moment			
Rigid Pile Head		~~Hinged Pile Head~~	
l_m	M_m	l_m	M_m
m	kN-m	m	kN-m
2.27	-1971.50	2.39	-1814.95

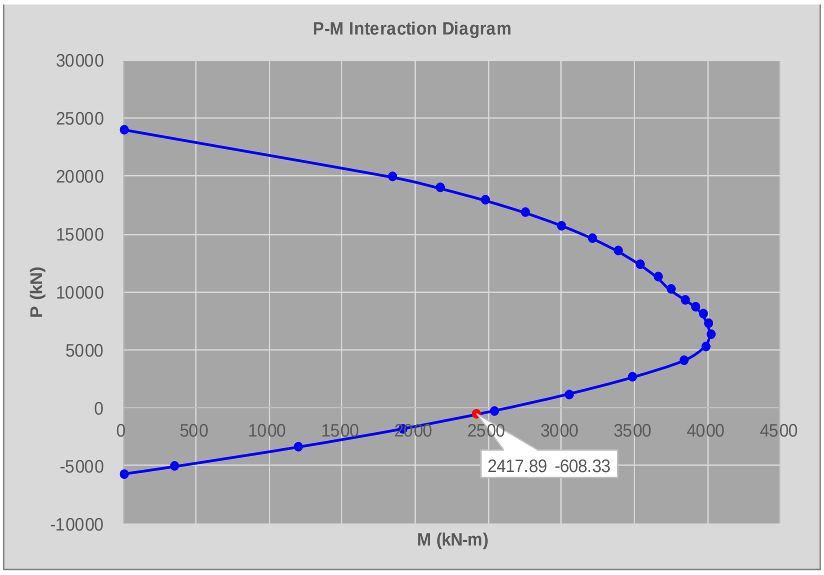
Note: The connection is rigid. Ignore values for hinged pile head.



Define Reinforcement of Pile :

Diameter of Longitudinal Bars	28	mm
Diameter of Hoops/spiral	16	mm
Reinforcement concrete cover to spiral/hoops	75	mm

Calculation for Pile Flexural Resistance for Longitudinal Direction



P-M Interaction Diagram of Pile

Maximum axial force that will result for maximum moment on the pile, P_{Hi} :	**-608.33**	kN
Flexural resitance factor, Ø :	**0.90**	factor
Ultimate flexural resistance, $M_r = ØM_n$:	**2417.89**	kN-m

Commentary

•The minimum axial force is the critical force.

Commentary

Calculation for Pile Shear Resistance for Longitudinal Direction

The nominal shear resistance, V_n, shall be determined by :

$V_n = V_c + V_s$ **DGCS Equation 12.5.3.2-1**

where :

$V_c = 0.083*\beta*Sqrt(f'_c)*b_v*d_v$ **DGCS Equation 12.5.3.2-3**

$V_s = [\ A_v*f_y*d_v*(cot\theta+ cot\alpha)sin\alpha\ ]\ /\ s$ **DGCS Equation 12.5.3.2-4**

b_v	effective web width, mm	1200	mm
d_v	effective shear depth, mm	823.61	mm
s	spacing of transverse reinforcement, mm	80	mm
β	factor indicating ability of diagonally cracked concrete to tra	2.0	factor
θ	angle of inclination of diagonal compressive stresses	45.0	deg
α	angle of inclination of transverse reinforcement	90.0	deg
A_v	area of shear reinforcement within a distance "s"	402.12	mm^2
D	external diameter of the pile, mm	1200	mm
D_r	diameter of pile passing the centers of the longitudinal reinfo	990	mm
$\emptyset$	shear resitance factor for normal weight concrete	0.90	factor

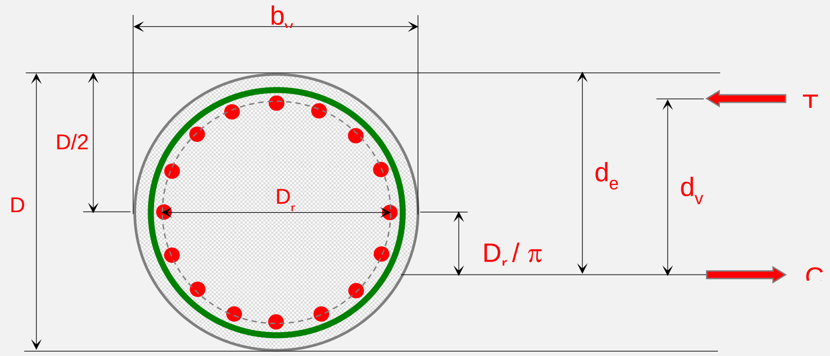


Illustration of Terms b_v, d_v and d_e for Circular Sections

Shear strength provided by the concrete, V_c :	868.14 kN
Shear strength provided by the reinforcements. V_s :	1718.07 kN
Nominal shear resistance, V_n :	2586.22 kN
Ultimate shear resistance, $V_r = \emptyset V_c$:	2327.60 kN

Verification for Single Pile

Flexural Resistance		C/D	Verification		Shear Resistance		C/D	Verification
Demand	Capacity				Demand	Capacity		
kN-m	kN-m				kN	kN		
1971.50	2417.89	1.23	OK		1850.64	2327.60	1.26	OK

Commentary

Verification of Minimum Required Longitudinal Reinforcement

The longitudinal reinforcement shall be verified according to the following :

where :

A_a total area of longitudinal reinforcement, mm^2

A_s cross-sectional area of single longitudinal reinforcement, mm^2

A_g gross area of pile, mm^2

A. The longitudinal reinforcement shall not be less than 0.01

$$\rho_s = A_a / A_g > 0.01$$

•DGCS 12.7.11

B. The longitudinal reinforcement shall be more than 0.04 times the gross section area

$$\rho_s = A_a / A_g < 0.04$$

•DGCS 12.7.11.2 for Zone 3 & 4

Pile Diameter	Bar Diameter	Number of Bars	A_s	A_a	A_g	ρ_s	Verification		
m	m	No.	m^2	m^2	m^2	ratio		A	B
								$\rho_s > 0.01$	$\rho_s < 0.04$
1.20	0.028	24	0.00062	0.01478	1.1310	0.013		OK	OK

Verification of Minimum Required Transverse Reinforcement

The ratio of spiral reinforcement to total volume of concrete core masured out-to-out of spirals shall be :

A. The greater of :

$\rho_{s1} = 0.12*(f'_c/f_y)$ and $\rho_{s2} = 0.45*[\,(A_g/A_c) - 1\,]*(f'_c/f_y)$ (for circular shape only)

where :

A_g gross area of pile, mm^2

A_c area of core measured to the outside diameter of the spiral, mm^2

B. Checking from provided confinement where As represent spiral leg on one (1) side:

$$\rho_{s3} = \frac{4*A_s}{D_r*s} > \text{greater of } \rho_{s1}, \rho_{s2}$$

where :

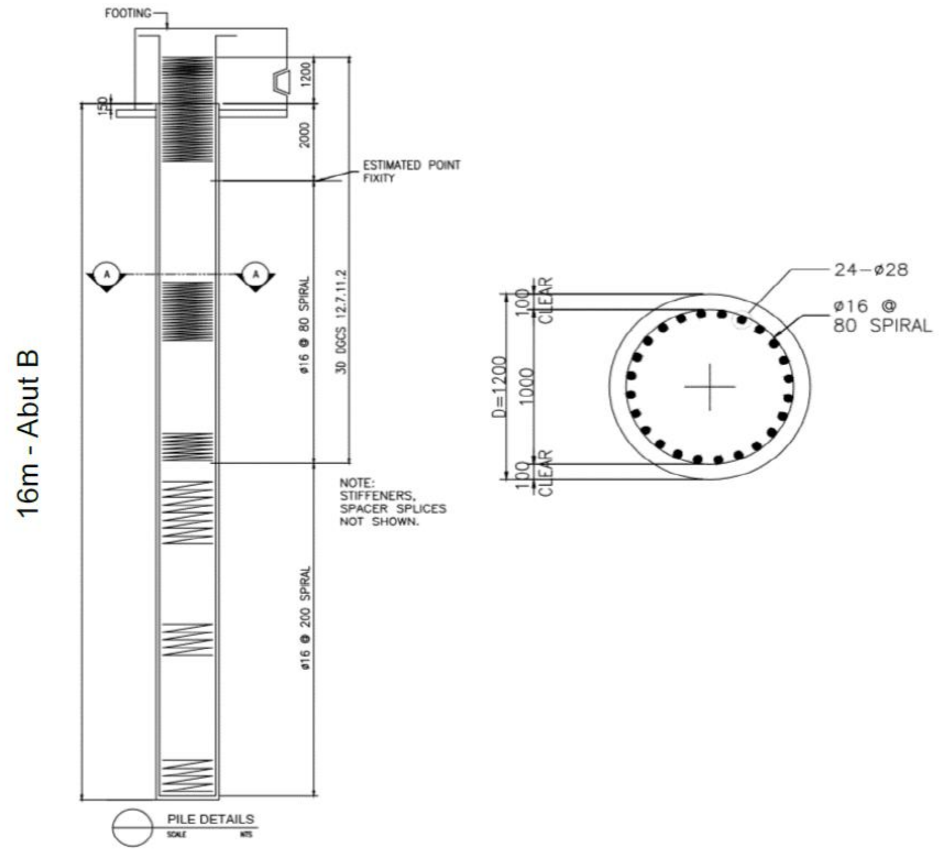
A_s area of shear reinforcement, mm^2

D_r diameter of pile passing the centers of the longitudinal reinforcement, mm

Pile Diameter	A_g	A_c	A_s	ρ_{s2}	$\rho s_1 =$ $0.12*(f'_c/$	ρ_s	ρ_{s3}	Verification
m	m^2	m^2	m^2	ratio	ratio	use max.	/erificatio	
1.20	1.13	0.87	0.00020	0.00931	0.00810	0.00931	0.01015	OK

Summary of results			Commentary
	Number of Piles	6	
	Length	16m	
	Diameter	1200mm	
	Reinforcemnt	24-28diam	
	Spiral Reinf.	16@80	

7.8.6 Pile Details



•Typical pile details. Miscellaneous details not shown.

CHAPTER 8: UNSEATING PREVENTION SYSTEMS

Chapter 8 Unseating Prevention Systems

Example Bridge

(1) Bridge Type 2 span continuous steel box girder

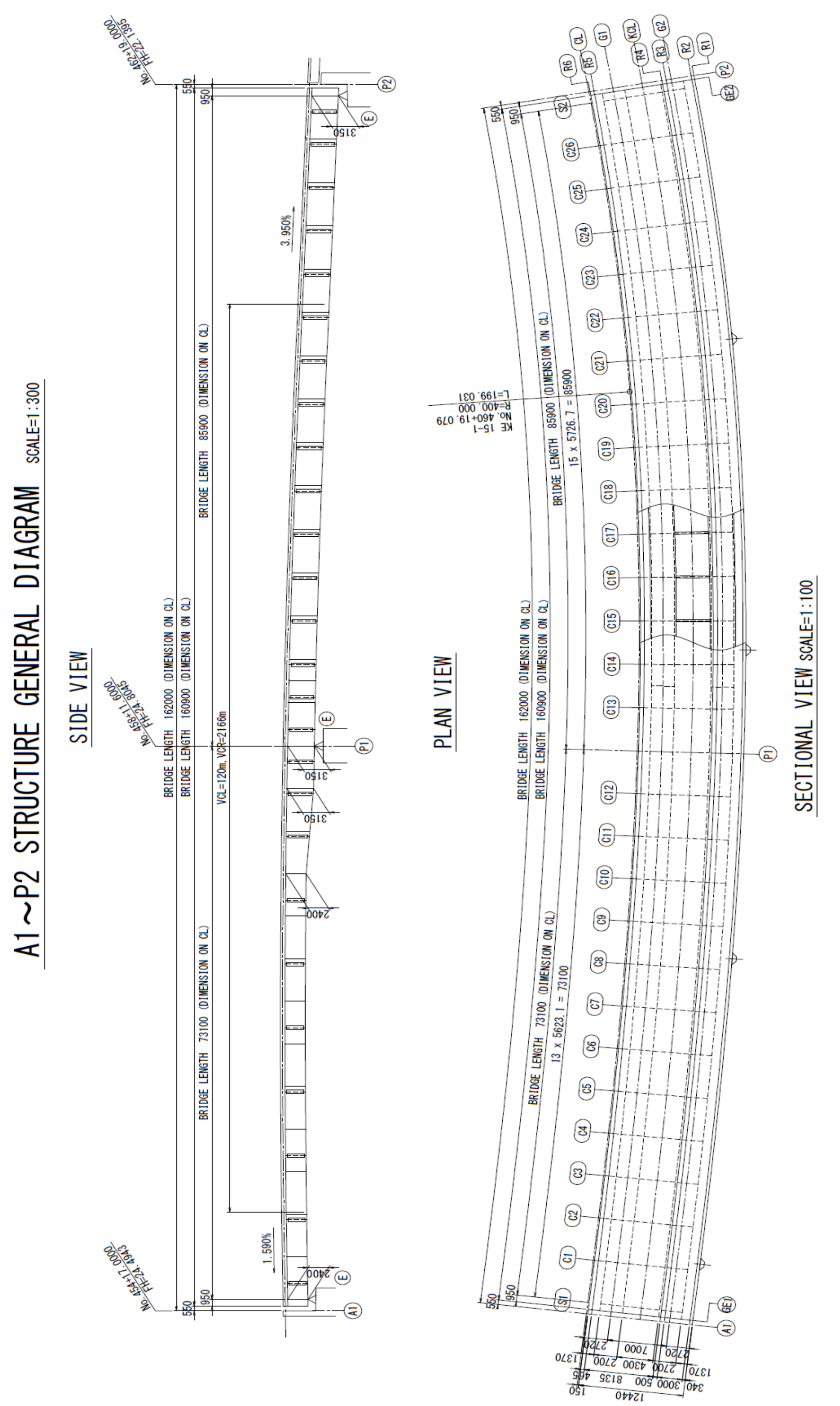


Figure 8-1 Example Bridge (a)

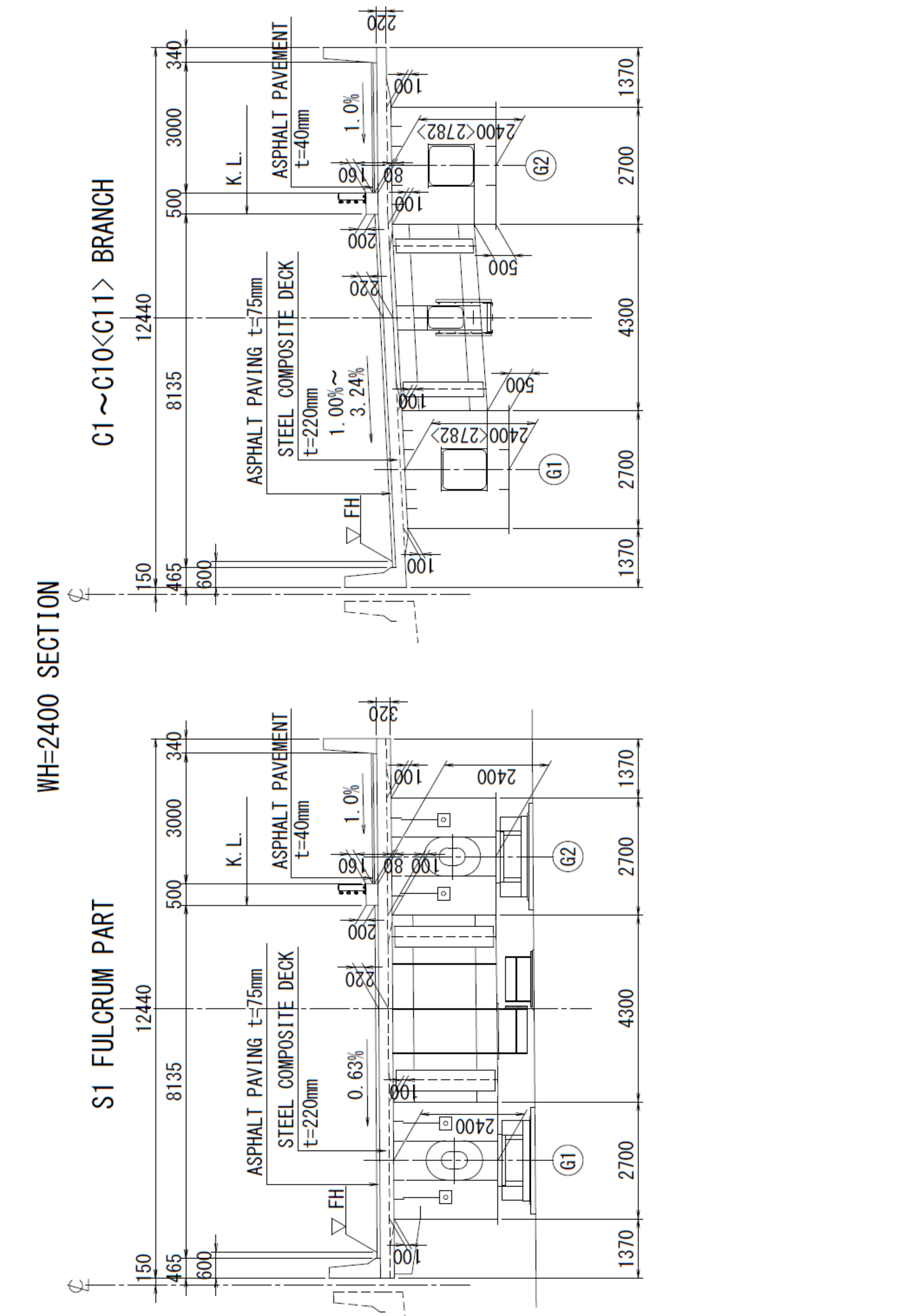


Figure 8-2 Example Bridge (b)

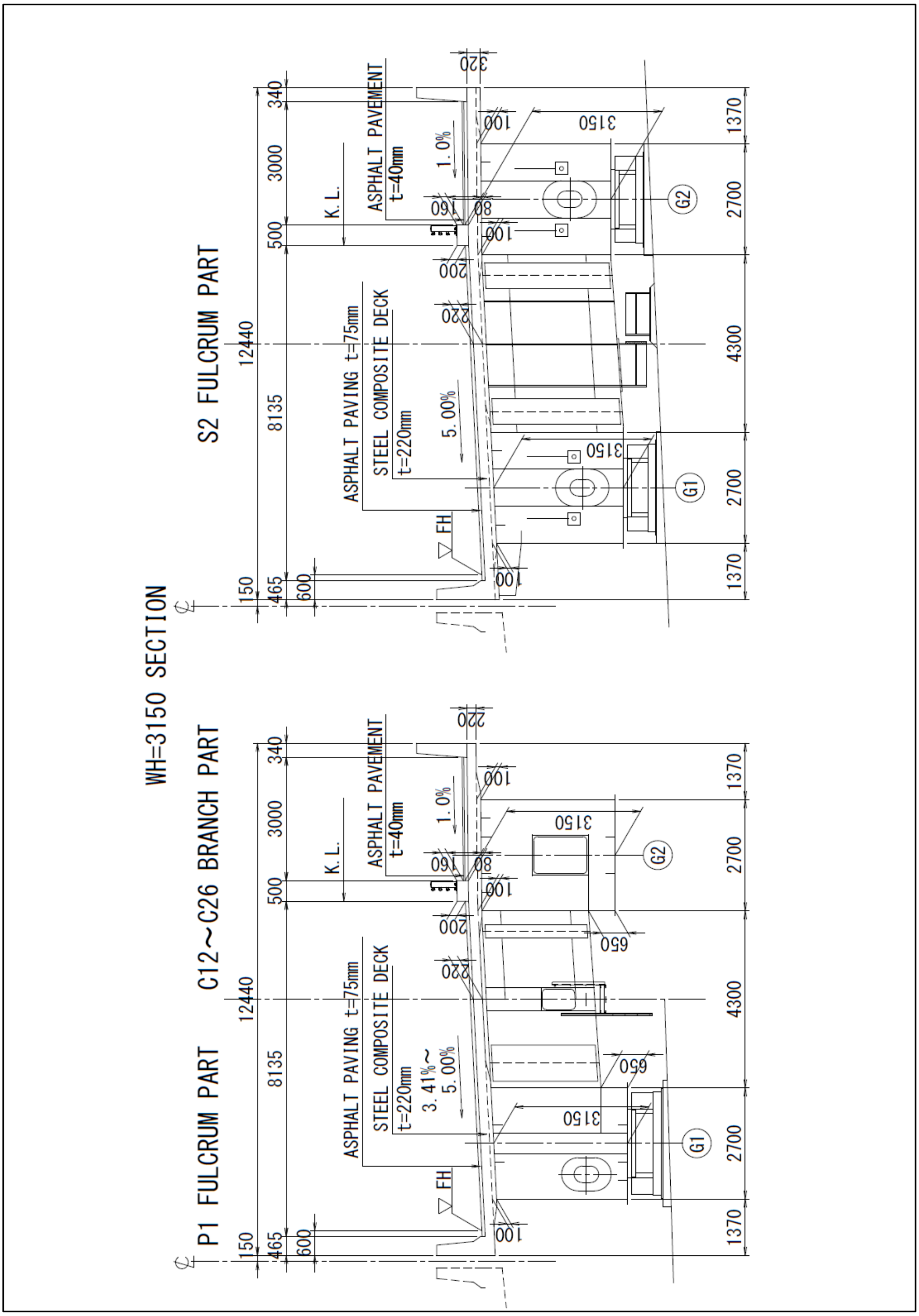


Figure 8-3 Example Bridge (c)

Ground Type: TypeII

8.1 Seat Length

$S_E = u_R + u_G \ \geqq \ S_{EM}$ -- (8.1-1) ~~¥~~

$S_{EM} = 0.70 + 0.005l$ -- (8.1-2) ~~¥¥~~

l: Effective span length(m) =85.9m

S_{EM}=1.130m

$u_G = \ \varepsilon_G \ L$= 0.00375*85.9=0.322m

u_R = Maximum relative displacement between the superstructure and top of the abutment. 0.90m

S_E=0.900+0.322=1222mm>S_{EM}＝1130ｍ

S_{Ed} ：1500ｍｍ （Designed）

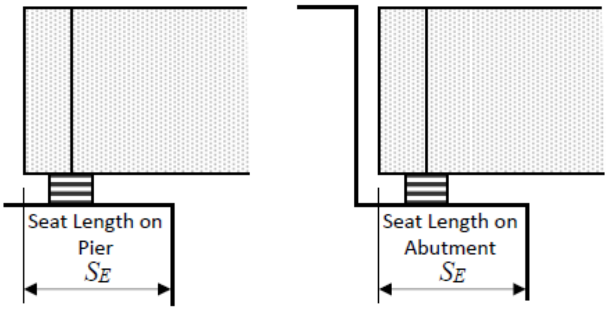


Figure 8.1-1 (a) Girder End Support

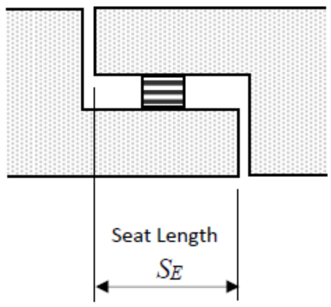


Figure 8.1-2 (b) Intermediate Joint (Hinge/Bearing Joint)

8.2 Unseating Prevention Devices（Longitudinal）

Unseating prevention devices are designed based on JRA Vol 5 2012.Adopted device is the connection method with use of PC cable which is anchored at parapet of abutment and lateral beam which is installed inside the box of main girder at A1 abutment and P2 pier.

Reaction of dead load kN

	RD
A1	5240.0
P2	6960.20

Design example is shown about P2 pier.

1. **Design Condition**
   1) **Design Load**

      Design seismic force applied to PC cable

      $H_F = P_{LG}$ (but $H_F \ \leqq \ 1.5R_d$) = 10440.3 kN → 10440.3 kN

      R_d: Reaction of dead load = 6960.2 kN (1.5 × 6960.2 = 10440.3 kN)

      P_{LG}: Longitudinal horizontal strength of supporting substructure – kN

Design seismic force P for 1 PC cable

$P = H_F \div n \times \sqrt{(1^2 + (\tan \theta v)^2 + (\tan \theta h)^2)}$

$= 10440.3 \div 4 \times \sqrt{(1 + (\tan 0.0)^2 + (\tan 0.0)^2} = 2610.1$ kN

where:

n: number of cables 4

θv: Cable setting angle (vertical) 0 °

θh: Cable setting angle (horizontal) 0 °

2) Design Gap

Design gap Sp of unseating prevention device shall be assured with the movement amount corresponding to the allowable strain and shall not exceed the value of seat length multiplied by design displacement coefficient.

Design minimum gap

$S_{F(min)} = \Sigma t \times \gamma = 228 \times 2.5 = 570$ mm

where:

Total rubber thickness, Σt = 228mm

γ: strain factor = 250%

Design maximum gap

$S_{F(max)} = C_F \times S_E = 0.75 \times 2450 = 1838$ mm

where:

C_F: Design displacement coefficient = 0.75

S_E: Seat length = 2450 mm

Determination of design gap

$S_{F(min)} \leqq S_F \leqq S_{F(max)}$

$570 \leqq S_F \leqq 1838$ then SF = 600 mm OK

2. Determine of fabrication length of spring

$N = b + \sigma 1 \div ns + 2 \times \phi + \sigma 2$

N	:	Fabrication length round up by 50mm unit.
b	:	Setting amount (mm) (S_F/N_S)
$\sigma 1$	:	Expansion and contraction amount due to temperature change (mm)
ϕ	:	diameter of spring (mm) n_S: number spring
$\sigma 2$	:	Minimum compression amount is 100 mm during installation

$$N = b + \sigma1 \div ns + 2 \times \phi + \sigma2$$
$$= 300 + 60 \div 2 + 2 \times 13.0 + 100$$
$$= 456.0 \text{ mm} \rightarrow 500 \text{ mm}$$

b	:	300	mm
$\sigma1$	:	60.0	mm
ϕ	:	13.0	mm
$\sigma2$	:	100	mm or more
n_S	:	2	

3. Design of PC cable and its buffer

1) PC cable

F360TD is adopted for PC cable yield strength Py = 2962 kN

P = 2610 kN < Pa = Py = 2962 kN OK

2) Buffer

Synthesized rubber textile fiber added (hardness 80º±5º) is used for buffer placed at anchor part of PC cable. Allowable bearing stress of rubber is 24N/mm2 considering increase ratio of allowable 1.5.

(1) Girder side

Buffer and bearing plate, diameter (D) = 390 mm

Hole diameter (d) = 117 mm Circular shape

Steel beam is adopted for anchoring.

Hole diameter is 135mm

(i) Bearing stress of buffer

Bearing area $Ab = (D^2 - d'^2) \times \pi \div 4 = 105145 \text{ mm}^2$

Bearing stress of buffer

$\sigma b = P \div Ab = 24.8 \text{ N/mm}^2 \leqq \sigma ba = 1.5 \times 24 = 36 \text{ N//mm}^2$ OK

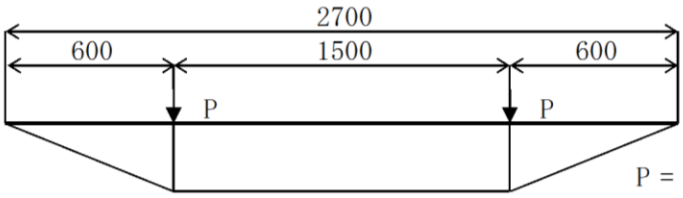
4. Calculation of lateral beam (Anchor beam)

(a) **Calculation of main part lateral beam**

It is designed as the simple beam with supporting span 2.7 m (web plate distance).

Plate thickness is 22mm assuring enough rigidity.

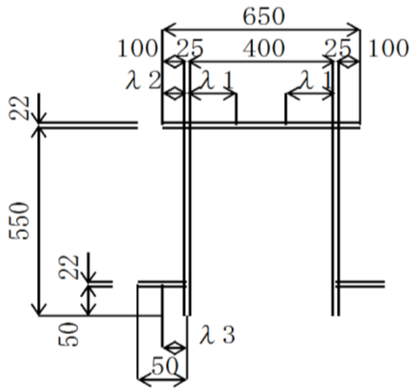
(i) Calculation of sectional force:



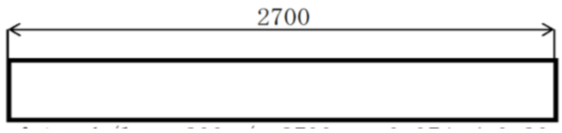
P = 2610.1 kN

Mmax = 2610.1 x 0.600 = 1566.1 kN·m
Smax = 2610.1 = 2610.1 kN

(ii) Section calculation



Equivalent span length



λ1 = b/1 = 200 / 2700 = 0.074 < 0.30
= (1.1 - 2×0.074) × 200 = 190 mm
λ2 = b/1 = 100 / 2700 = 0.037 < 0.05
= Effective width = 100mm
λ3 = b/1 = 50 / 2700 = 0.019 < 0.05
= Effective width = 50mm

Position of maximum force

	A (mm2)	y (mm)	Ay (mm3)	Ay^2orI (mm4)
< SM490Y > 1 - PL 630 x 22 :	13860	286.0	3963960	1133692560
< SM490Y > 2 - PL 550 x 25 :	27500	—	—	693229167
< SM490Y > 2 - PL 50 x 22 :	2200	-214.0	-470800	100751200
A =	43560		3493160	1927672927
				-278784000
			I =	1648888927 mm4

e = 3493160 / 43560 = 80 mm
yu = 275 + 22 - 80 = 217 mm
ylf = 275 - 50 + 80 = 305 mm
ylw = 275 + 80 = 355 mm

σu = 1566.1 x 10^6 / 1648888927 x 217 = 206.1 N/mm2 < σa = 357.0 N/mm2
σlf = 1566.1 x 10^6 / 1648888927 x 305 = 289.7 N/mm2 < σa = 357.0 N/mm2
σlw = 1566.1 x 10^6 / 1648888927 x 355 = 337.2 N/mm2 < σa = 357.0 N/mm2
τ = 2610.1 x 10^3 / 27500 = 94.9 N/mm2 < τa = 204.0 N/mm2
σa = 1.7 x 210.0 = 357.0 N/mm2 , τa = 1.7 x 120.0 = 204.0 N/mm2

Resultant stress : (337.2 / 357.0) ^2 + (94.9 / 204.0) ^2 = 1.11 < 1.2

(iii) Welding with web plate of main girder

Size of flange: S = K shape groove weld

Web S = 2610.1 x 10^3 / 4 x 550 x 0.707 x 204.0 = 8.23 mm

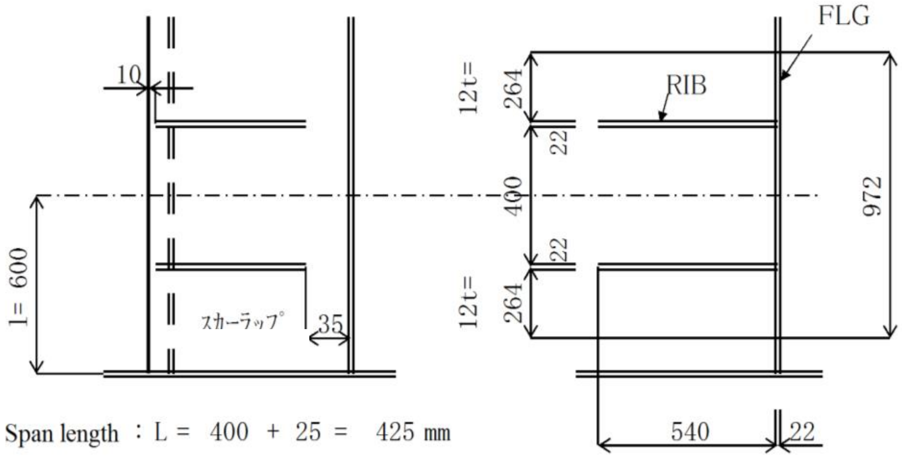
S' = $\sqrt{}$ 2t = $\sqrt{}$ (2 x 25) = 7.07 mm

as above 9mm fillet weld

Size of rib: S = K shape groove weld with web of main girder and filet weld with more than $\sqrt{}$2t

(b) Calculation of reinforcement rib at anchor part of PC cable

Verify stresses assuming simple girder supported by 2 webs of lateral beam.



Span length : L = 400 + 25 = 425 mm

(i) Calculation of sectional force

M = P•L / 4 = 2610.1 x 0.425 / 4 = 277.0 kN•m

S = P / 2 = 2610.1 x ½ = 1305.0 kN

(ii) Stress calculation

	A (mm2)	y (mm2)	Ay (mm3)	Ay^2 or I (mm4)
< SM490Y > 1 - PL 972 x 22 :	21384	281.0	6008904	1688502024
< SM490Y > 2 - PL 540 x 22 :	23760	—	—	577368000
	45144		6008904	2265870024
				-798552216
			I =	1467317808 mm4

e = 6008904 / 45144 = 133 mm

yu = 270 + 22 - 133 = 159 mm

yl = 270 + 0 + 133 = 403 mm > yu

σ = 277.0 x 10^6 / 1467317808 x 403 = 76.1 N/mm2 < σa = 357.0 N/mm2

τ = 1305.0 x 10^3 / (540 - 35) x 22 x 2 = 58.7 N/mm2 < τa = 204.0 N/mm2

σa = 1.7 x 210.0 = 357.0 N/mm2 , τa = 1.7 x 120.0 = 204.0 N/mm2

Resultant stress : (76.1 / 357.0) ^2 + (58.7 / 204.0) ^2 = 0.13 < 1.2

(iii) Weld design with lateral beam

Fillet weld Size of Flange: S = √2t or more

Fillet weld Size of Web: S = 1305.0 x 10^3 / 4 x (540-35) x 0.707 x 204.0=4.48 mm

S' = √ 2t = √ (2 x 22) =6.63 mm

From above, Fillet weld size is 7 mm

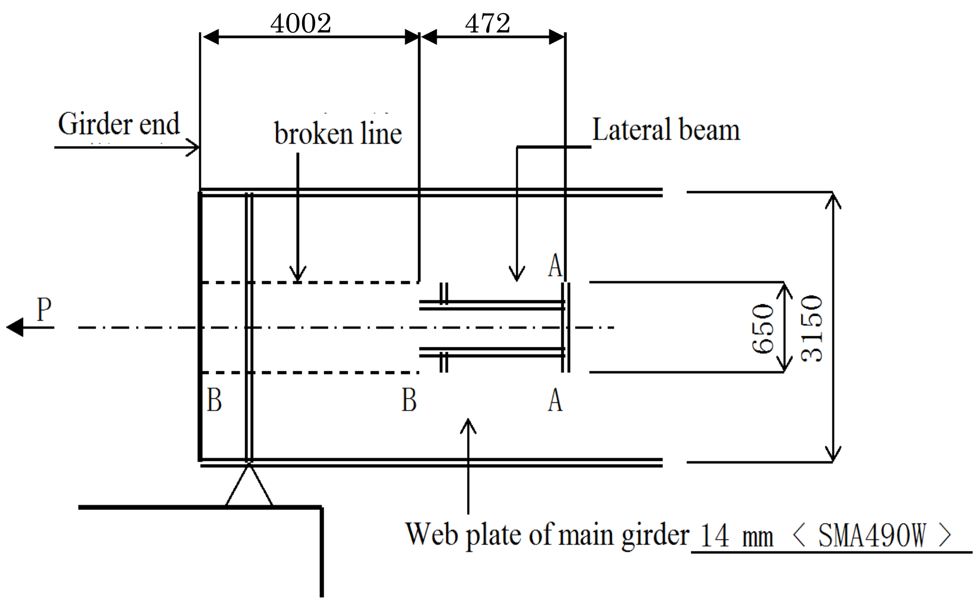
(c) Verification of Web plate of main girder

Check the plate when design earthquake P acts on failing bridge prevention structure.

2 cases are considered

1st one is broken by tensile force at section A-A

2nd one is broken by shear at section B-B (refer to figure below)



Verification of tensile stress at A-A section

Design seismic force of unseating prevention device arranged on web plate are assumed to be distributed uniformly.

σ = P / Hw × Wt

= 2610.1 × 10^3 / 650 × 14 = 286.8 N/mm2

< σa = 1.7 × 210.0 = 357.0 N/mm2

where: Hw = width of reinforcement plate (650mm)

Wt = Thickness of web of main girder (t=14mm)

Verification of shear stress on B-B section

τ = P / 2 × Wl × Wt

= 2610.1 × 10^3 / 2 × 3978 x 14 = 23.4 N/mm2

< τa = 1.7 × 120.0 = 204.0 N/mm2

where: Wl = Broken length at B-B section (3978mm)

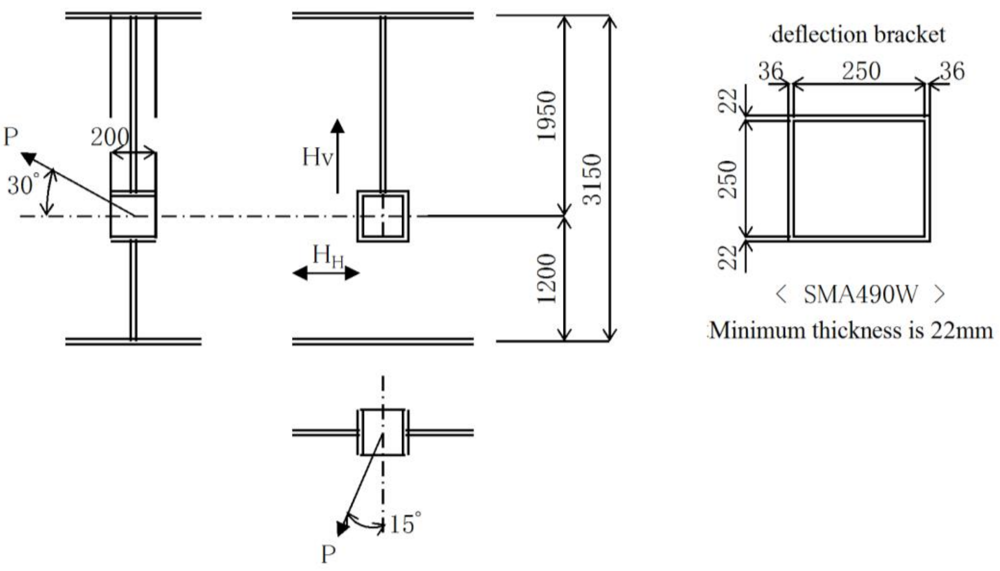
Wt = Thickness of web of main girder (t=14mm)

5. Verification of deflection bracket for unseating prevention device

Increase of allowable stress is 1.7

Acting force generated from cable is as follows assume 30°leaning in vertical direction

assume 30°leaning in horizontal direction



(a) Applying Force

Cable Force : P = 2610.1 from previous calculation

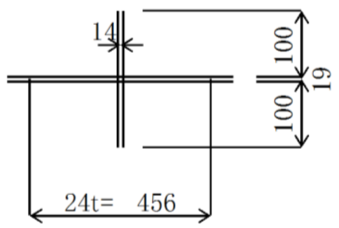
Vertical Force : Hv = 2610.1 × sin 30° = 1305.1 kN

Horizontal Force: H_H = 2610.1 × sin 15° = 675.5 kN

(b) Verification in vertical direction

Verified assuming a column of cross shape composed of web of lateral beam and stiffener

Effective buckling height is all height (H = 3150mm)



Force = 1305.1 kN

＜SMA490W＞

2 −	Stiff	PL :	100 x 16	=	3200	mm2	(As)
1 −	Web	PL :	456 x 19	=	8664	mm2	(Aw)
				A =	11864	mm2	

A ＞ 1.7・As= 5440 mm2

Verification of axial compressive stress

σ c = 1305.1 x 10ˆ3 / 5440 = 239.9 N/mm2 ＜ 1.7 σ ca = 270.3 N/mm2

I = 219 ˆ3 x 14 / 12 = 12254036 mm4

r = √(12254036 / 11864) = 32.14

l/r = (3150 x 1/2) / 32.14 = 49.0 ＞ 15

σ cag =Table 3.2.2 in close 3.2 in JRA = 159.0 N/mm2

b/t = 100 / 14 = 7.1 ＜ 10.5 (但し、＜ 16)

σ cal = Table 4.2.3 in close 3.2 in JRA = 210.0 N/mm2

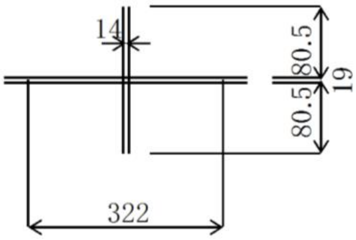
σ cao = upper limit of σcag = 210.0 N/mm2

σ ca = σ cag ・ σ cal / σ cao = 159.0 N/mm2

Verification of bearing stress

Effective section of web : Disadvantageous side of 24t of web thickness or whole width of attached bracket

Effective section of stiffener : Disadvantageous side of stiffener width or attached bracket width + 4mm both side



<SMA490W>

2 −	Stiff	PL :	80	x	14	=	2240 mm2	(As)
1 −	Web	PL :	322	x	19	=	6118 mm2	(Aw)
						A =	8358 mm2	

Scallop of bracket shall be berried

σ b = 1305.1 x 10^3 / 8358 = 156.1 N/mm2 < 1.7 σ ba = 535 N/mm2

Verification of weld

Applying force is assumed to be triangular distribution in consideration for safety

Effective weld length does not include deflection bracket and scallop

S = 2 × 1305.1 × 10^3 / 4 × (1000) × 0.707 × 120.0 × 1.7

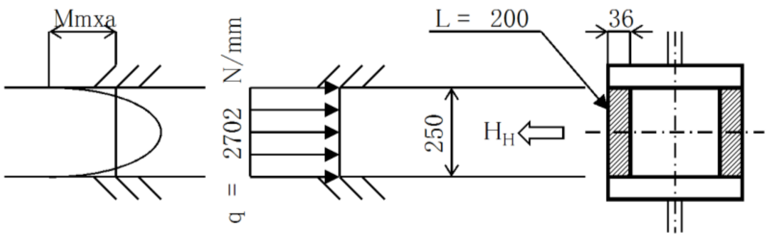
= 4.52 mm < S' = $\sqrt{2t}$ = $\sqrt{2 \times 19}$ = 6.16 mm

As above, size of fillet weld is 7 mm.

(c) Verification in Horizontal direction

Verification for following force along the web plate (hatched part)

Minimum thickness is 22mm



- Bending moment

Applying force:	H_H =	675.5	kN
Span length:	LB =	250	Mm
Distributed load:	q =	2702	N/mm
	Mmxa =	-q × LB^2 / 12 = 14072917 N.mm	

- Shear force

S = H_H = 675.5 × 10^3 / 2 = 337750 N

- Verification of Stress

Use section: t = 36 mm <SMA490W>

Section: A = 200 × 36 = 7200 mm2

Section Modulus: $Z = 200 \times 36^2 / 6 = 43200$ mm3

Bending Stress: $\sigma = 14072917 / 43200 = 325.8$ N/mm2 $< 1.7\ \sigma a = 357$ N/mm2

Shear Stress: $\tau = 337750 / 7200 = 46.9$ N/mm2 $< 1.7\ \tau a = 204.$N/mm2

(d) Bearing stress of concrete on the parapet of abutment

Buffer and bearing plate are assumed to be circular, diameter is 390mm and hole diameter is 117mm

Box out pipe is used for anchor part of concrete

Outer diameter of box out (d2') = 140 mm (VP125)

(i) Bearing stress of buffer

Bearing area $Ab' = (D^2 - d2'^2) \times \pi \div 4 = 104065$ mm^2

Bearing stress of buffer

$\sigma b = P \div Ab' = 25.1$ N/mm$^2 \leqq \sigma ba = 1.5 \times 24 = 36$ N/mm^2 OK

(ii) Bearing stress of concrete

$\sigma b = P \div Ab' = 25.1$ N/mm$^2 > \sigma ba = 18.0$ N/mm^2 Stiffener is necessary

where: σba: allowable bearing stress in case of partial loading

$\sigma ba = 1.5 \times (0.25 + 0.05 \times Ac \div Ab) \times \sigma$

$= 1.5 \times (0.25 + 0.05 \times 16.8) \times 24$

$= 18.0$ N/mm^2 (but, $\sigma ba \leqq 1.5 \times 0.5\sigma$)

Ac: Effective area of concrete in case of partial loading

$Ac = (Dc^2 - d2'^2) \times \pi \div 4 = 1751752$ mm^2

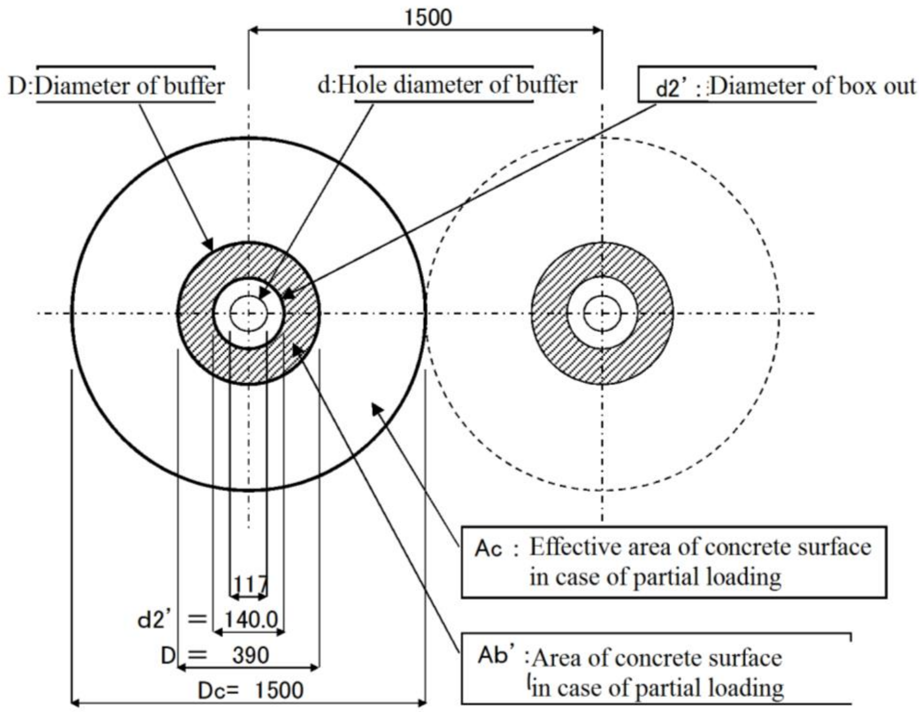
Ac is circle area with diameter of half of distance of 2 cable

Dc = 1500 mm Dc $\leqq 5 \times D$) in condition

Ab': Area of concrete face subjected with partial loading

$Ab' = (Dc^2 - d2'^2) \times \pi \div 4 = 104065$ mm^2

therefore: $Ac \div Ab' = 16.8$



As bearing stress of concrete exceeds the allowable bearing stress, stiffener is installed for the purpose of distribution.

Effective bearing diameter is 440 x 440 in relation of outer diameter of buffer and thickness of rib plate.

Reverification of bearing stress of concrete after installation of stiffener

Bearing Area Ab' = D^2 - $d2'^2$ x π ÷ 4 = 178206 mm^2

σb = P / Ab' = 14.7 N/mm^2 ≦ σba = 18.0 N/mm^2 OK

σba Allowable bearing stress in case of partial loading

σba = 1.5 x (0.25 + 0.05 x Ac ÷ Ab') x σ

= 1.5 x (0.25 + 0.05 x 12.5) x 24

= 18.0 N/mm^2 (but, σba ≦ 1.5 x 0.5σ)

Ac: Effective bearing area on concrete face in case of partial loading

Ac = Dc^2 – ($d2'^2$ x π ÷ 4) = 2234606 mm^2

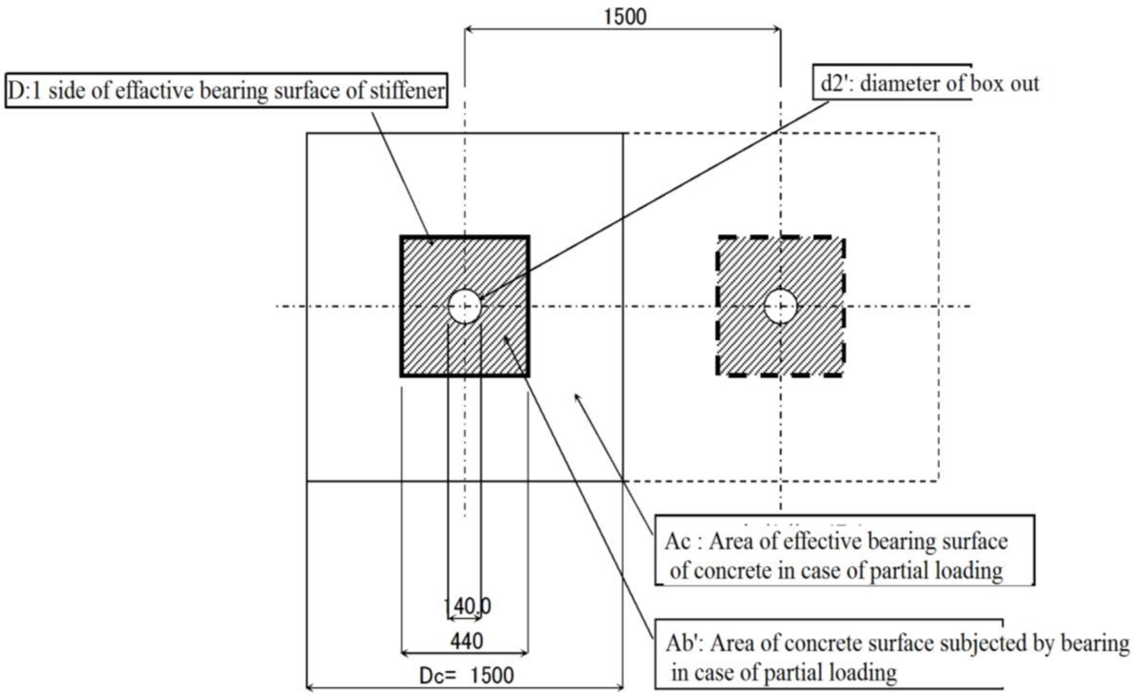
Ac is the area of rectangle with distance between center of cable as a side

Dc = 1500 mm (but, Dc ≦ 5 x D)

Area of concrete surface subjected by bearing in case of partial loading

Ab' = D^2 – ($d2'^2$ x π ÷ 4) = 178206 mm^2

Then Ac ÷ Ab = 12.5



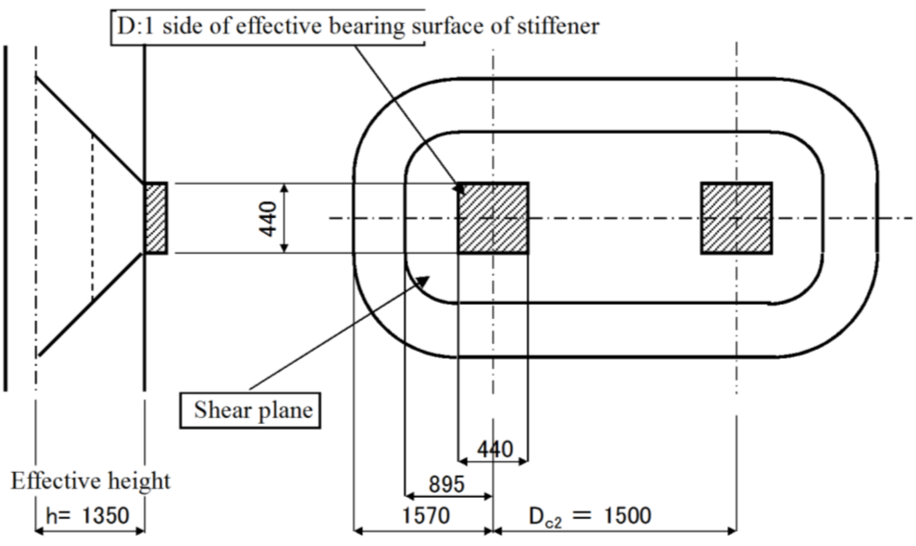
6. Design of parapet of abutment

1) Verification of punching shear

Increase factor of allowable is 1.5

Cable number for verification n = 2

Distance of cable Dc' = 1500mm



Shear resistance area

$$A = (D \times 4 + h \times \pi + Dc \times 2) \times h$$
$$= (440 \times 4 + 1350 \times \pi + 1500 \times 2) \times 1350$$
$$= 12151553 \text{ mm}^2$$

where: h: Effective height = h1-h2 = 1500-150 = 1350 mm
h1: thickness of concrete 1500 mm
h2: Cover of re bar = 150 mm

Punching shear stress

$$\tau = P \times 2 \div A = 2611000 \times 2 \div 12151553$$
$$= 0.43 \text{ N/mm}^2 \leqq \tau a = 1.5 \times 0.90 = 1.35 \text{ N/mm}^2 \quad \text{OK}$$

8.3 Unseating Devices (Transverse)

- **Al Abutment**

1. Outline

The bridge is curves bridge applicable to provision 16.1(4)1), then transverse displacement prevention structure shall be installed.

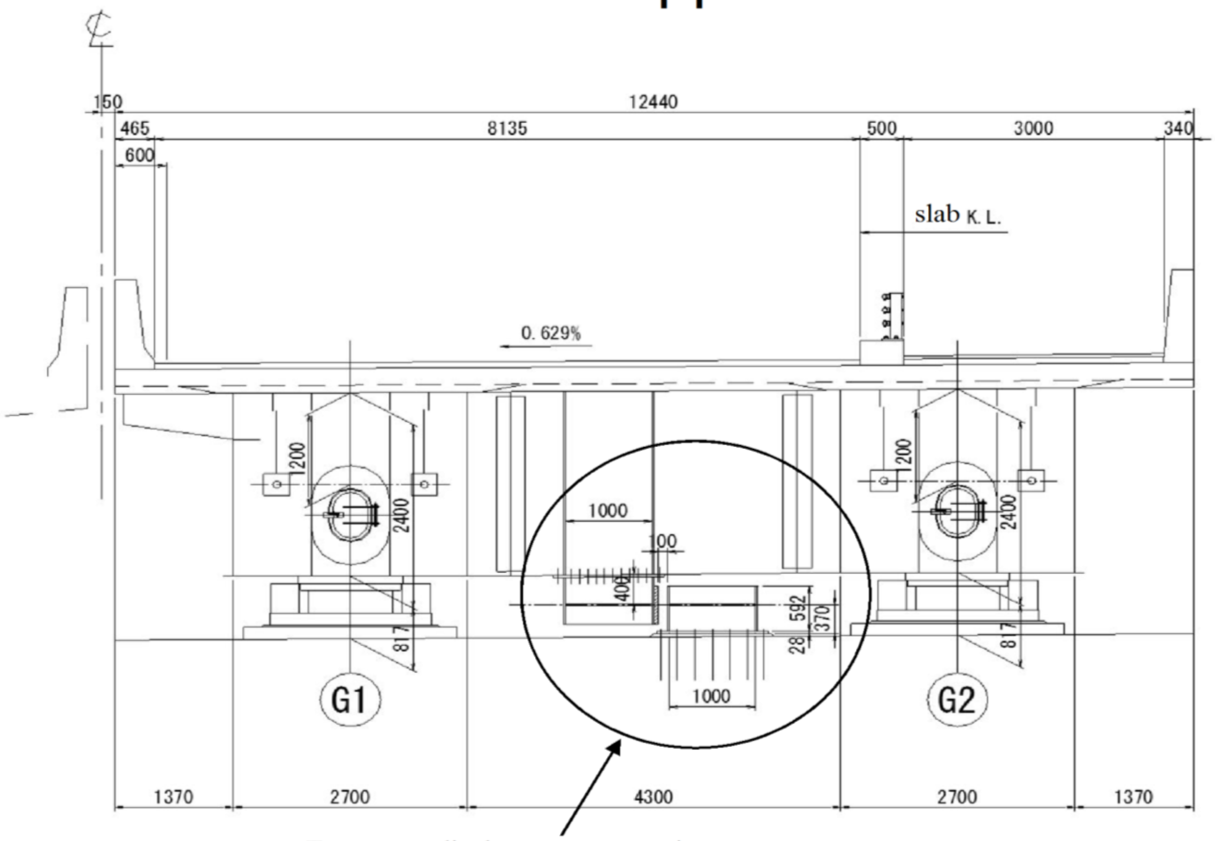
The gap in-between bracket is 100mm in consideration for transverse direction associated with longitudinal movement during earthquake.
Displacement during earthquake is derived from dynamic analysis.

2. Design force

$$Hs = 3 \cdot kh \cdot Rd/n \quad \text{For Sl support}$$
$$= 4087.2 \text{ kN}$$

kh = 0.26
Rd = 5240 kN/pier
n = 1 installation number

S1 Support



Transverse displacement prevention structure

3. Calculation of bracket on upper structure side

   (1) Design of buffer rubber

   Used rubber is chloroprene rubber (hardness 55±5°)

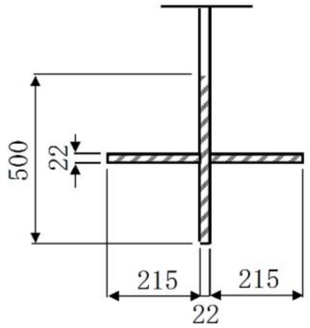
   Necessary sectional area: Areq = $4087.2 \times 10^3 / 12.0 / 1.5 = 227067$ mm^2

   Used sectional area: A = $500 \times 500 = 250000$ mm^2 > Areq

   (2) Design of bracket

   - Mounting part of buffer

     Design reaction = 4087.2 kN



effective width of section is 12t and within buffer rubber size

(SMA490W)				A (mm^2)
2 -	215	×	22	9460
1 -	500	×	22	11000
			ΣA=	20460

I = 169299915 mm^4
r = 91 mm
L = 1000 mm

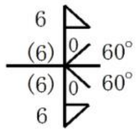
L/r = 11 < 15
b/t = 9.8 < 10.5
σ cag = 210 N/mm^2 , σ cao = 210 N/mm^2
σ cal = 210 N/mm^2
σ ca = 210 N/mm^2 × 1.7 = 357 N/mm^2

Stress σ c = 4087.2 × 10^3 / 20460 = 199.8 < σ ca= 357 N/mm^2

Weld of rib

necessary size
Sreq = 2 × 4087.2 ×10^3/(4 × 1.732 × 1000 × 204)= 5.8 mm

from above, weld is as shown right .

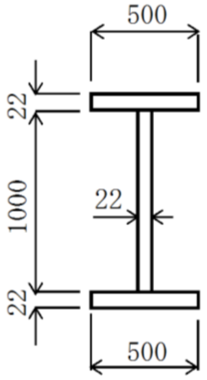


(3) Bracket

Mounting part to lateral beam calculation

Conversion value to normal time (1/1.7)

Sectional force M = Hs ・e = 4087.2 × 0.450 / 1.7 = 1081.9 kN・m
S = Hs = 4087.2 = 2404.2 kN



distance of fixing point L= 900 mm

(SMA490W)				A (mm^2)	y (mm)	Ay^2+I (mm^4)
2 - FLG	500	×	22	22,000	511	5,745,549,333
1 - WEB	1000	×	22	22,000		1,833,333,333
				44,000		7,578,882,666

yu = 522 mm L/b = 6 < 30
yl = −522 mm Aw/Ac = 2.00 ≦ 2

Jstress $\sigma t = M / I \cdot yu = 74.5$ N/mm² $< \sigma ta = 210.0$ N/mm²

$\sigma c = M / I \cdot yl = -74.5$ N/mm² $< \sigma ca = 198.5$ N/mm²

$\tau = S / Aw = 109.3$ N/mm² $< \tau a = 120.0$ N/mm²

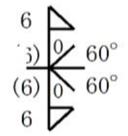
Resultant stress $(74.5 / 210.0)^2 + (109.3 / 120)^2 = 0.96 < 1.2$

weld between web and base plate

Necessary fillet weld size

$$Sreq = 2404.2 \times 10^3 / (2 \times 1.732 \times 1000 \times 120) = 5.8 \text{ mm}$$

As above, weld is as shown right



Weld between flange and base plate is full penetration as shown below

(4) Design of mounting part of bracket

(a) High tension bolt is used for mounting bolt (m22(S10TW, 2 row are arranged on both sides of neutral axis of bolt group).

Allowable force of bolt is determined by 1.7 times of normal time

Lever reaction force generated by tensile force is considered

Verification for friction connection (Resultant force is considered in case that whole number of bolts are effective.

Force applying to 1 bolt

$\rho = P \div n_b = 4087200 \div 56$

$= 72986 \text{ N} \leqq \rho_a = 1.7 \times 48704 = 82797 \text{ N}$

ρ_a: Allowable force for 1 friction high tension bolt (1 plane friction strength: inorganic zinc rich paint is considered

n_b: Bolt number = 56 (whole bolt number)

(b) Verification of bolt tension

Verify the tensile force of a bolt by calculating secondary moment of inertia around the neutral axis of bolt group.

Bending moment

$M = P \times L = 4087200 \times 400.0 = 1634880000 \text{ N}\cdot\text{mm}$

Arrangement of bolt

Row No	Number n (mm)	pitch (mm)	distance y (mm)	n•y (no. mm)	ye (mm)	n• ye² (mm²)
1 row	6	0	0	0	581.0	2025366
2 row	6	140	140	840	441.0	1166886
3 row	4	98	238	852	343.0	470596
4 row	4	98	336	1344	245.0	240100
5 row	4	98	434	1736	147.0	86436
6 row	4	98	532	2128	49.0	9604
7 row	4	98	630	2520	-49.0	9604
8 row	4	98	728	2912	-147.0	86436
9 row	4	98	826	3304	-245.0	240100
10 row	4	98	924	3696	-343.0	470596
11 row	6	98	1022	6132	-441.0	1166886
12 row	6	140	1162	3972	-581.0	2025366
13 row	0	0	0	0	0.0	0
14 row	0	0	0	0	0.0	0
15 row	0	0	0	0	0.0	0
Total	56	1162		32536		7997976

Painted part shows compressive bolts ye: distance from neutral axis (+tension, - compressive)

Neutral axis of bolt group

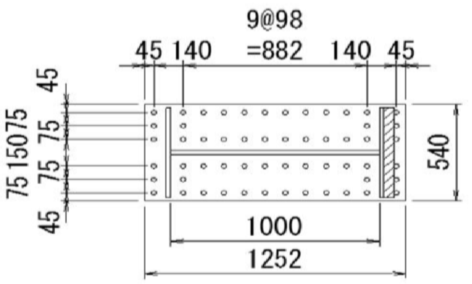
$$e = \frac{\Sigma\ ny}{\Sigma\ n} = 581.0\ \text{mm}$$

Σ ny: n•y Sum

Σ n: total number

Bolt tension (lever reaction is ignored)

$$\rho_t{}' = \frac{M}{\Sigma\,(n \cdot ye^2)} \times ye_{max} = \frac{1634880000}{7997976} \times 581.0 = 118763\ \text{N}$$



Calculation of lever reaction coefficient

In consideration for lever reaction generated by tension force, calculate the lever reaction coefficient regarding “a” part based on the “Draft of design guideline of tension connection with use of high-tension bolt”

nf:	Bolt number resisting load as the tension connection	=	12 mm
n’:	Bolt number arranged on the one side of T flange/2 (longitudinal direction)	=	6 mm
c :	Distance of ete of T web direction ≦ 3.5b	=	75 mm
e :	Bolt edge distance in T web direction	=	45 mm
w:	Length of T flange (n’-1) c + 2e	=	500 mm
t :	Thickness of T flange 1.0d (Base plate thickness	=	28 mm
tw:	Thickness of T web (thickness of rib)	=	22 mm
tc:	Base plate thickness where T flange is connected ≧ t	=	19 mm
d :	Nominal bolt diameter	=	22 mm
d’:	Bolt hole diameter d+3	=	25 mm
Ab:	Axial sectional area $(d/2)^2\pi$	=	380.1 mm^2
c’ :	Distance of ete of bolt in T flange direction	=	140 mm
b :	Distance between bolt center to surface of T web (c’-tw/2)	=	59.0 mm
a :	Distance between bolt center to end of T flange	=	45 mm
s :	Weld size of flange and web (leg length of groove fillet weld)	=	5.5 mm
b’:	Distance between bolt center to center of fillet weld of T web/2	=	56.3 mm

$$\phi = a \,/\, b' = 45 \,/\, 56.3 = 0.80$$

$$\eta = \frac{24\ n' \cdot Ab \cdot b'^3}{w \cdot t^3\,(t + tc)} = \frac{24 \times 6 \times 380.1 \times 56.3^{\ 3}}{500 \times 28^{\ 3} \times (\ 28 + 19\ )} = 18.9$$

Therefore $\eta \cdot \phi^3 - \phi^2 - 2\phi - 1 = 0$ $\qquad \phi = 0.49$

以上より、 $\phi = 0.49 \leqq \phi = 0.80$

$$pu = \frac{1}{2} \times \frac{1}{(1+\phi)^2 - 1} = \frac{1}{2} \times \frac{1}{(\ 1 + 0.49\ )^2 - 1} = 0.41$$

Load applying to 1 bolt in consideration of lever reaction

$$\rho t = \rho t'\ (1 + py) = 118763 \times (\ 1 + 0.58\ )$$
$$= 187646\ N \leqq \rho_{ta} = 1.7 \times 160000 = 272000\ N$$

where: ρ_{ta}: Allowable force per 1 high tension bolt for tension connection

(c) Verification of base plate thickness

σy: Yield stress of T flange = 355 N/mm^2 (SMA490W)

σu: Tensile strength of T flange = 490 N/mm^2

By: Yielding bolt axial force = 273 kN

py: Coefficient of lever reation at yielding of axial force

$$py = \frac{(11 + Pu)\,Pu}{10 - (1 + Pu)^2} = \frac{(\ 11 +\ 0.41\ ) \times\ 0.41}{10\ -\ (\ 1\ +\ 0.41\ )^2} =\ 0.58$$

$$k = 0.5 + 0.9\,\sigma u / \sigma y = 0.5 + 0.9 \times 490 / 355 = 1.74$$

$$\delta = 1 - n' \cdot d' / w = 1 - 6 \times 25 / 500 = 0.70$$

Necessary baseplate thickness

$$t1 = \sqrt{\frac{6\,n' \cdot By \cdot py \cdot a}{\delta \cdot w\,(1+py)\,k \cdot \sigma y}}$$

$$= \sqrt{\frac{6 \times 6 \times 273 \times 1000 \times 0.58 \times 45}{0.70 \times 500 \times (\ 1 + 0.58\ ) \times 1.74 \times 355}} = 27.5 \text{ mm}$$

$$t2 = \sqrt{\frac{6\,n' \cdot By\,(b' - a \cdot py)}{w\,(1+py)\,k \cdot \sigma y}}$$

$$= \sqrt{\frac{6 \times 6 \times 273 \times 1000 \times (\ 56.3 - 45 \times 0.58\ )}{500 \times (\ 1 + 0.58\ ) \times 1.74 \times 355}} = 24.7 \text{ mm}$$

$$t = 28 \text{ mm} \geqq t1\ ,\ t2$$

(d) Reduction of allowable sear stress of High-Tension Bolt

(1) From Eq. 7.3.10 in provision 7.3.7 JRA

$$\rho a = \rho a2 \cdot (n \cdot N - T) / (n \cdot N) = 48704 \text{ N}$$

ρa : Allowable shear force per 1 bolt (N)

$\rho a2$: Allowable bolt force of 1 1 bolt as a friction connection (N) 54000 N

n : Total number of bolt at connection part 56

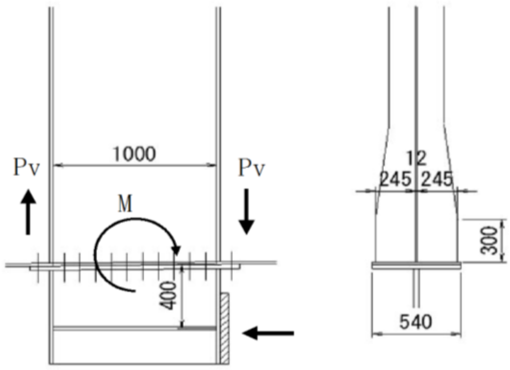
N : Initial induction axial force of bolt (N) 205000 N

T : Tensile force applying to connection part (N) 1125876 N

4. Reinforcement to Lateral Beam

Reinforcement rib is installed for the coupling force due to bending moment caused by horizontal force.

Where, this reinforcement rib is used to double with vertical stiffener of lateral beam.



Acting bending moment and coupling force

M = 1634.9 kN・m

Pv= 1634.9 / 1.000 = 1634.9 kN

Reinforced section (SMA400W)

2 −	245 × 22	10780			I =	231927681	mm^4
1 −	288 × 12	3456		(24・t)	r =	127.6	mm
	ΣA=	14236	mm^2		L =	3150	mm

L/r = 24.7 > 18

b/t = 11.1 > 12.8

σ cag = 134.5 N/mm^2 , σ cao = 140 N/mm^2

σ cal = 185.5 N/mm^2

σ ca = σ cag・σ cal／σ cao = 178.2 N/mm^2 × 1.7 = 303 N/mm^2

axial compressive stress

σ c = 1634.9 $\times 10^3$ / 14236 = 115 N/mm^2 < σ ca= 303 N/mm^2

Weld between reinforcement rib and web of lateral beam

$\sqrt{2t} = \sqrt{(2 \times 22)} = 6.6$ mm

Necessary weld length when fillet weld size is 7mm Lreq

Lreq = 2 × 1634.9 $\times 10^3$ /(4 × 6 × 0.707 × 80 × 1.7)

= 1416.9 mm < WH

As above mentioned, required welding length is less than the girder height of lateral beam, it is extended as the vertical stiffener.

5. Verification of lateral beam

Section calculation is carried out assuming lateral beam are subjected with axial force from lateral displacement prevention device in addition to dead load.

Md = 412.8 kN・m Confer to design of lateral beam

Sd = 189.5 kN

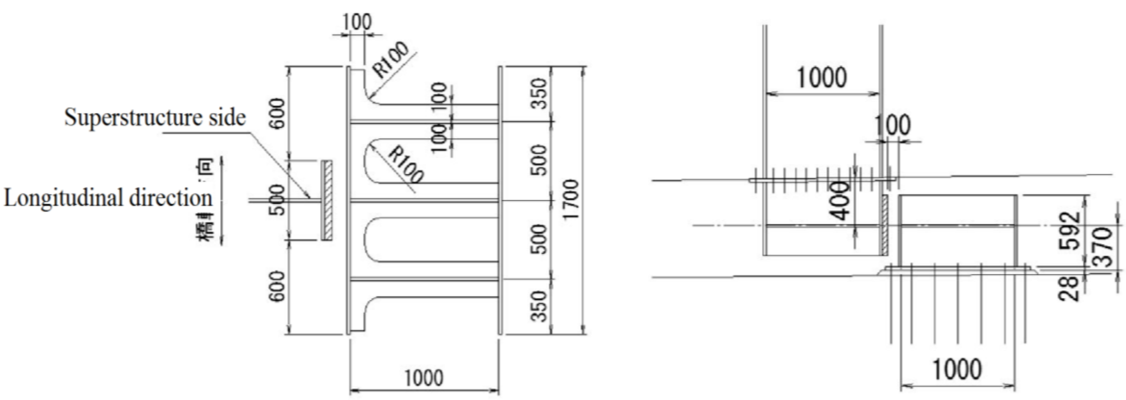
Ne = 4087 kN / 1.7 = 2404 kN (converted value of normal time)

Section S1

```
Section S1                          (※{weathering steel ia adapted.

Sectional force   Loading case name      MY          SZ          NX          MZ          SY          MX
   (    1)CASE.1                       412.80      189.49    -2404.00        0.00        0.00        0.00

                  |section ĵ     Tmin  Material quality  <dimension>                  <Section shape
      1-FLG     300x  10> 9.0(SM490Y )  |AS=       41688  mm2                    <---300--->T= 10
      1-WEB    2369x  12>11.3(SM490Y )  |JX=0.00000271  m4                       |    (UF2)   |(UF)
      1-FLG     540x  19>16.5(SM490Y )  |AW/AS= 0.68>0.4                    _____UUUUUUUUUUU
                              < in plane    >   < out of plane>                          W  (UW)
      sectional force        =         41688          41688  mm2         web height       W
      δ                      =         208.3            0.0  mm            2369          RW  (CW)
      Moment of inertia      =  0.03035920     0.00027216  m4             T= 12            W
      Buckling length        =        4.3000         4.3000  m             |____           W  (LW)
  Radius of giration of area =      853.3745        80.7990  mm                      LLLLLLLLLLL
      slenderness ratio)     =        5.0388        53.2185                          |    (LF2)   |(LF)
    Euler's buckling stress  =         77745            697  N/mm2                   <---540--->T= 19
  Effective width coefficient =      1.00000        1.00000
Verification of distance of vertical stiffener  (Horizontal stiffener is 1 tier)  σ =  76.6、τ =   2.2  Decision eq.   0.11<1.0

    < Allowable stress   (N/mm2)    loading case name =CASE.1                    upper flange   web    Lowe flange
  allwable stress in consideration for local buckling related chapter 4.2 JRA Vol 5 ( σ cal)   110.9   210.0   119.1
        allowable stress without consideration for local buckli ng                ( σ cag)   152.7   152.7   152.7
        upper limit of allowable stress without consideration for local buckling  ( σ cao)   210.0   210.0   210.0
        Allowable axial compressive stress                       ( σ ca=σ cag*σ cal/σ cao)    80.6   152.7    86.6
   Allowable bending compressive stress ( σ bagy)L  4.300  L/B= 14.3<27   8.0<27           134.4   210.0   187.7

    < Stress   >   《CASE.1》   σ NX= -57.7< 80.6
  factor depending on stress gradient:   U-FLG=1.0000、L-FLG=1.0000)
      |        σ MY     σ MZ     σ NX   Σ σ     σ cal|   τ SZ    τ SY    τ MX    Σ τ    τ a|<1.0  <1.2|
      |UF      -19.1     0.0    -57.7   -76.8<111|     0.0     0.0     0.0     0.0<120|0.86  0.13|
      |UF2     -19.1     0.0    -57.7   -76.8<111|     1.3     0.0     0.0     1.3<120|0.86  0.13|
      |UW      -18.9     0.0    -57.7   -76.6<210|     2.2     0.0     0.0     2.2<120|0.47  0.13|
      |CW        0.0     0.0    -57.7   -57.7<210|     8.3     0.0     0.0     8.3<120|0.38  0.08|
      |LW       13.3     0.0    -57.7   -44.4<210|     5.3     0.0     0.0     5.3<120|0.31  0.05|
      |LF2      13.5     0.0    -57.7   -44.1<119|     1.7     0.0     0.0     1.7<120|0.60  0.04|
      |LF       13.5     0.0    -57.7   -44.1<119|     0.0     0.0     0.0     0.0<120|0.60  0.04|
```

6. Calculation of bracket in substructure side

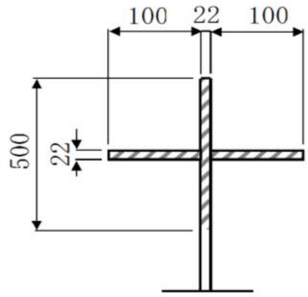
Substructure side bracket is associated with the longitudinal movement of superstructure. As movement amount is 600 mm during earthquake, bracket of substructure is arranged with interval of 500 mm to double with superstructure bracket.



(1) Design of web and rib

Effective section width is least of member size, buffer rubber size, 12t

Design reaction = 4087. 2 kN



(SMA490W)					A (mm^2)
2 −	100	×	22		4400
1 −	500	×	22		11000
				ΣA=	15400

I = 20058588 mm^4

r = 36. 1 mm

L = 1000 mm

L/r = 27. 7 > 15

b/t = 4. 5 < 10. 5

σ cag = 191 N/mm^2 , σ cao = 210 N/mm^2

σ cal = 210 N/mm^2

σ ca = 191 N/mm^2 × 1. 7 = 325 N/mm^2

Stress σ c = 4087. 2 × 10^3 / 15400 = 265. 4 < σ ca= 325 N/mm^2

Weld of rib

Required size

Sreq = 2 × 4087. 2 ×10^3 /(4 × 1. 732 × 1000 × 204)= 5. 8 mm

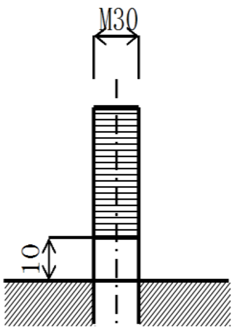
from above Weld is groove fillet 6mm 。

(2) Calculation of anchor bolt

Sectional forces for design are the applying force of bending moment and shear force at the bottom of base plate

- Calculation of sectional force

M = 889. 6 kN·m , S = 2404. 2 kN



- Section calculation

Used section ; : DB-D 32 (SD345 : M 30)

Area of embeded part : A1 = 794. 2 mm^2

Section narea of the thread : A2 = 560. 6 mm^2 Effective diameter

Allowable tensile stress : σ a = 200. 0 N/mm^2

Allowable shear stress : τ a = 115. 0 N/mm^2 (σ a×$\sqrt{3}$)

Allowable bolt force : ρ ta1 = 200. 0 × 560. 6 = 112120 N/ Bolt (σa)

ρ ta2 = 115. 0 × 794. 2 = 91333 N/ Bolt (τa)

Position of Neutral Axis (calculated with origin is 1st row of bolt)

	Number per row	Distance of Row	Accumulated distance	Moment of Area	Distance from Neutral Axis	Moment of Inertia around Neutral Axis
	n (mm)	p (mm)	yi (mm)	n• yi	e-yi (mm)	n• (e-yi)^2
1st Row	8	0.0	0.0	0	592.0	2803712
2nd Row	4	192.0	192.0	768	400.0	640000
3rd Row	4	200.0	392.0	1568	200.0	160000
4th Row	4	200.0	592.0	2368	0.0	0
5th Row	4	200.0	792.0	3168	-200.0	160000
6th Row	4	200.0	992.0	3968	-400.0	640000
7th Row	8	192.0	1184.0	9472	-592.0	2803712
Σn =	36		Σn•yi =	21312	ΣI =	7207424

1st row is the farthest tensile bolt row

Eccentricity: e = Σn•yi / Σn = 592.0 mm (From 1st row to neutral axis)

Verification of maximum bending tensile stress

ρ t1 = 889.6 × 10^6 / 7207424 × 592.0 = 73070 N/bolt

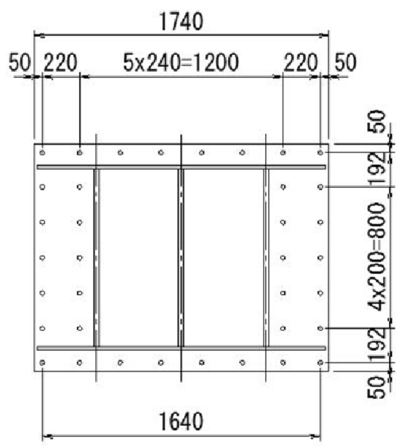
< ρ ta1 = 112120 N/bolt

Verification of shear force

ρ t2 = 2404.2 × 10^3 / 36 = □6783 N/bolt < ρ ta2 = 91333 N/bolt

Verification of resultant force

k = (73070 / 112120) ² + (66783 / 9133) ² = 0.96 < 1.2



(3) Calculation at anchoring part

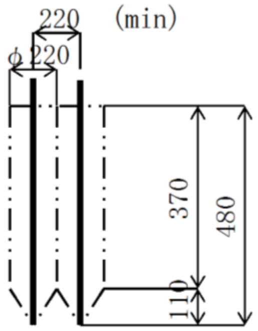
Tension bolt force	73070 N/bolt
Adopter bolt diameter	DB-D 32
Embeded length of bolt	= 480 mm (15D) = 15D = 480 mm
Concrete strength	σck = 24 N/mm
Allowable bonding stress	τ a = 1.60 N/mm^2
Allowable punching shear stress	τ ca = 0.90 N/mm^2

- Verification of bond stress between concrete and anchor bolt

  $\rho = 73070 / (32 \times \pi \times 480) = 1.51$ N/ mm^2 < τ a = 1.60 N/ mm^2

- Verification of pull out shear stress of concrete

Per 1 bolt



shear resistance area

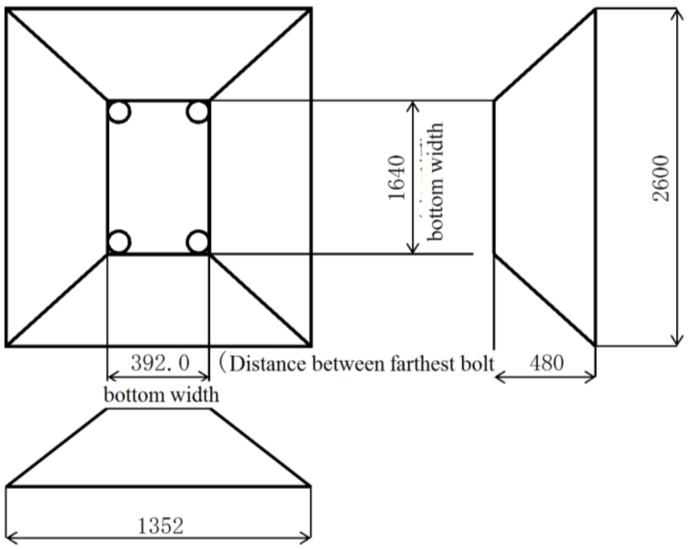
$$a1 = 220 \times \pi \times 370 = 255726 \text{ mm}^2$$
$$a2 = 220 / 2 \times \pi \times \sqrt{2} \times 220 / 2 = 53759 \text{ mm}^2$$
$$\Sigma a = 309485 \text{ mm}^2$$

Pull out Shear stress

$$\tau t = 73070 / 309485 = 0.24 \text{ N/mm}^2 < \tau ca = 0.90 \text{ N/mm}^2$$

Tensile Bolt Group

Resistance area is 45-degree distributed area from tip of bolt



Shear resistance area

$a = 1352 \times 2600 = 3515200$ mm^2

Bolt force acting to tension bolt group

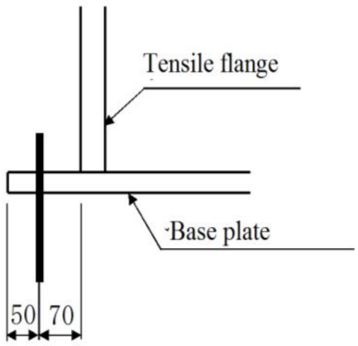
$\Sigma \rho t = 73070 \times 16 = 1169120$ N

Pull out shear stress

$\tau c = 1169120 / 3515200 = 0.33$ N/mm^2 < τ ca = 0.90 N/mm^2

(4) Calculation of Base plate

Base plate thickness is determined by bending moment generated by bolt axial force supported by tensile flange



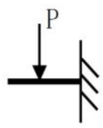
Maximum bolt axial force and applying bending moment

$P = \rho t1 \cdot n = 73070 \times 8 = 584560$ N

$M = P \cdot a = 584560 \times 70 = 40919200$ N·mm

Base plate thickness required

Material quality SMA490W



$$
\begin{aligned}
treq &= \sqrt{[\,6 \cdot M / (\alpha \cdot \sigma a \cdot B)\,]} \\
&= \sqrt{[\,6 \times 40919200 / (1.0 \times 210 \times 1740)\,]} \\
&= 25.9 \text{ mm} \rightarrow 28 \text{ mm}
\end{aligned}
$$

B = 1740 mm (·Base plate width

$\alpha$is arbitrary coefficient

8.4 Settling Prevention Devices

Outline

Load considered is vertical dead load and not horizontal dead load.

Strength limit stress of steel is 1.7 times of allowable stress and capacity of concrete member is obtained as concrete area multiplied by specified design strength. Minimum thickness of structural steel plate is 22mm.

(1) End support

Design regarding P2 where lager reaction occurs.

a) Design Force

Rd = 6975.5 kN/Pier Use P2 support

n = 4 places

P = Rd / n = 1743.9 kN

b) Necessary size of buffer rubber

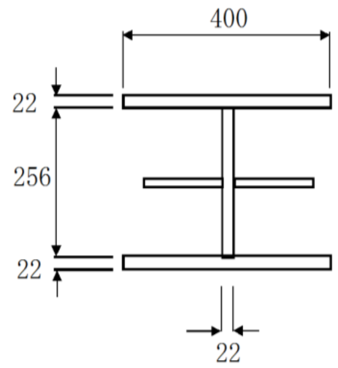
Using material is equivalent to chloroprene rubber (hardness 55)

Necessary area Areq = 1743.9 × 10^3 / 1.5 / 12 = 96883 mm^2

Used area A = 300 × 400 = 120000 > 96883 mm^2

c) Design of Bracket

- Mounting section of buffer



Design force P = 1743.9 kN

(SMA490W) Minimum thickness is 22mm

		A (mm^2)	
2 –	400 × 22	17600	center rib is ignored
1 –	256 × 22	5632	
	ΣA =	23232 mm^2	

σ = 1743.9 $\times 10^3$ / 23232 = 75.1 N/mm^2
< σ a= 1.7 × 205.8 = 350 N/mm^2

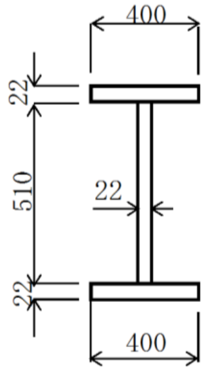
L = 2378 mm
r = $\sqrt{(I / A)}$ = 126.5 mm

L/r =	18.8	> 15	σ cag=	205.8	N/mm^2
b/t =	8.6	<10.5	σ cal=	210.0	N/mm^2
			σ ca' =	205.8	N/mm^2

- Section calculation of mounting portion to main girder

Converted value to normal time (1/1.7)

Sectional force M = 348.8 kN•m (1743.9 × 0.340 × 1/1.7)
S = 1025.8 kN



Distance between fixed point L= 680 mm

(SMA490W)		A (mm^2)	y (mm)	Ay^2+I (mm^4)
2 – FLG	400 x 22	17,600	266	1,246,015,467
1 – WEB	510 x 22	11,220		243,193,500
		28,820		1,489,208,967

yu = 277 mm L/b = 1.7 < 30
yl = −277 mm Aw/Ac = 1.275 > 2

Stress						
σ t	= M / I • yu	=	64.9 N/mm^2	<	σ ta=	210.0 N/mm^2
σ c	= M / I • yl	=	−64.9 N/mm^2	<	σ ca=	210.0 N/mm^2
τ	= S / Aw	=	91.4 N/mm^2	<	τ a=	120.0 N/mm^2

Resultant stress $(64.9 / 210.0)^2 + (91.4 / 120)^2 = 0.68 < 1.2$

Weld between web and base plate

Necessary size of fillet weld

Sreq = $1025.8 \times 10^3 / (2 \times 0.707 \times 510 \times 120)$ = 11.9 mm

$\sqrt{2t}$ = 6.6 mm

From above, size of fillet weld is 12mm.

d) Design of mounting part of bracket

High tension bolt is used for mounting bolt (M22(S10TW,2 row are arranged on both sides of neutral axis of bolt group).

Allowable force of bolt is determined by 1.7 times of normal time

Lever reaction force generated by tensile force is considered

(i) Verification for friction connection (Resultant force is considered in case that whole number of bolts are effective.

Force applying to 1 bolt

$\rho = P \div n_b = \quad 1743900 \div 28$

$= \quad 62282 \div N \leqq \rho_a = 1.7 \times 44443 = 75553$ N

ρ_a : Allowable force for 1 friction high tension bolt (1 plane friction strength: inorganic zincrich is considered n_b: bolt number = 28 (all bolt number)

(ii) Verification of tensile force of bolt

Verify the tensile force of a bolt by calculating secondary moment of inertia around the neutral axis of bolt group

Bending moment

$M = P \times L = 1743900 \times 340.0 = 592926000$ N・mm

Bolt arrangement

Row No	Bolt No	pitch (mm)	distance y (mm)	n•yi (no. mm)	ye (mm)	n• ye² (mm²)
1 row	4	0	0	0	330.0	435600
2 row	4	130	130	520	200.0	160000
3 row	4	100	230	920	100.0	40000
4 row	4	100	330	1320	0.0	0
5 row	4	100	430	1720	-100.0	40000
6 row	4	100	530	2120	-200.0	160000
7 row	4	130	660	2640	-330.0	435600
8 row	0	0	0	0	0.0	0
9 row	0	0	0	0	0.0	0
10 row	0	0	0	0	0.0	0
11 row	0	0	0	0	0.0	0
12 row	0	0	0	0	0.0	0
13 row	0	0	0	0	0.0	0
14 row	0	0	0	0	0.0	0
15 row	0	0	0	0	0.0	0
Total	28	660		9240		1271200

Painted part shows the compression bolt. Ye: Distance from neutral axis (+ tension side, - compression side)

Neutral axis of bolt group

$$e = \frac{\Sigma\, ny}{\Sigma\, n} = 330.00 \text{ mm}$$

Σ ny: n•y Total

Σ ny: No. Total

Bolt tension (lever reaction is ignored)

$$\rho_t' = \frac{M}{\Sigma\,(n \cdot ye^2)} \times ye_{max} = \frac{592926000}{1271200} \times 330.0 = 153922 \text{ N}$$

(calculation of lever reaction force)

In consideration for lever reaction generated by tension force, calculate the lever reaction coefficient regarding "a" part based on the "Draft of design guideline of tension connection with use of high-tension bolt"

nf:	Bolt number resisting load as the tension connection	=	8
n':	Bolt number arranged on the one side of T flange/2 (longitudinal direction)	=	4
c :	Distance of ete of T web direction ≦ 3.5b	=	100 mm
e :	Bolt edge distance in T web direction	=	40 mm
w:	Length of T flange (n'-1) c + 2e	=	400 mm
t :	Thickness of T flange ≧ 1.0d (Base plate thickness	=	24 mm
tw:	Thickness of T web (Rib thickness)	=	22 mm
tc:	Base plate thickness where T flange is connected ≧ t	=	14 mm
d :	Nominal bolt diameter	=	22 mm
d':	Bolt hole diameter d+3	=	25 mm
Ab:	Axial sectional area $(d/2)^2\pi$	=	380.1 mm^2
c' :	Distance of ete of bolt in T flange direction	=	130 mm
b :	Distance between bolt center to surface of T web (c'-tw/2)	=	54.0 mm
a :	Distance between bolt center to end of T flange	=	40 mm
s :	Weld size of flange and web (leg length of groove fillet weld)	=	5.5 mm
b':	Distance between bolt center to center of fillet weld of T web b-s/	=	51.3 mm

ϕ = a / b' = 40 / 51.3 = 0.78

$$\eta = \frac{24\, n' \cdot Ab \cdot b'^3}{w \cdot t^3\,(t + tc)}$$

$$= \frac{24 \times 4 \times 380.1 \times 51.3^3}{400 \times 24^3 \times (24 + 14)} = 23.4$$

Therefore: $\eta \cdot \phi^3 - \phi^2 - 2\phi - 1 = 0$ $\phi = 0.45$

$\phi = 0.45 \leqq \phi = 0.78$

$$Py = \frac{1}{2} \times \frac{1}{(1+\phi)^2 - 1}$$

$$= \frac{1}{2} \times \frac{1}{(1+0.45)^2 - 1} = 0.45$$

Load applying to 1 bolt in consideration of lever reaction

$\rho t = \rho t' (1 + Py) = 153922 \times (1 + 0.65)$

$= 253971 \text{ N} \leqq \rho_{ta} = 1.7 \times 160000 = 272000 \text{ N}$

ρ_{ta}: Allowable force per 1 high tension bolt for tension connection

(iii) Verification of base plate thickness

σy : Yielding stress of T flange = 355 N/mm (SMA490Wmaterial)

σu : Tensile strength of T flange = 490 N/mm

By : Yielding bolt axial force = 273 kN

Py : lever reaction coefficient at yielding axial force

$$\frac{(11 + Pu)\,Pu}{10 - (1 + Pu)^2} = \frac{(11 + 0.45) \times 0.45}{10 - (1 + 0.45)^2} = 0.65$$

$k = 0.5 + 0.9\,\sigma u / \sigma y = 0.5 + 0.9 \times 490 / 355 = 1.74$

$\delta = 1 - n' \cdot d' / w = 1 - 4 \times 25 / 400 = 0.75$

Necessary base plate thickness

$$t1 = \sqrt{\frac{6\,n' \cdot By \cdot Py \cdot a}{\delta \cdot w\,(1+Py)\,k \cdot \sigma y}}$$

$$= \sqrt{\frac{6 \times 4 \times 273 \times 1000 \times 0.65 \times 40}{0.75 \times 400 \times (1 + 0.65) \times 1.74 \times 355}} = 23.7 \text{ mm}$$

$$t2 = \sqrt{\frac{6\,n' \cdot By\,(b' - a \cdot Py)}{w\,(1+Py)\,k \cdot \sigma y}}$$

$$= \sqrt{\frac{6 \times 4 \times 273 \times 1000 \times (51.3 - 40 \times 0.65)}{400 \times (1 + 0.65) \times 1.74 \times 355}} = 20.2 \text{ mm}$$

$t = 24 \text{ mm} \geqq t1 , t2$

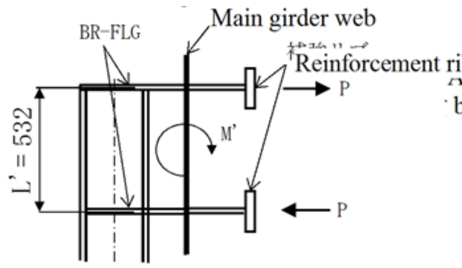
(iv) Reduction of allowable sear stress of High Tension Bolt

From Eq 7.3.10 in provision 7.3.7 JRA

$\rho a = \rho a2 \times (n \times N - T) / (n \times N) = 44443 \text{ N}$

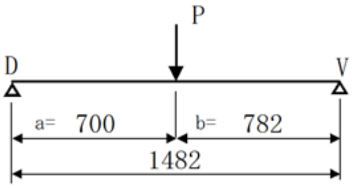
ρa :	Allowable shear force per 1 bolt (N)	
$\rho a2$:	Allowable bolt force of 1 as a friction connection (N)	54000 N
n :	Total number of bolts at connection part	28 No.
N :	Initial induction axial force of bolt	205000 N
T :	Yensile force applying to connection part (N)	1015884 N

(v) Reinforcement calculation (Calculate at G2 where stiffener distance of main girder is largest inside the girder

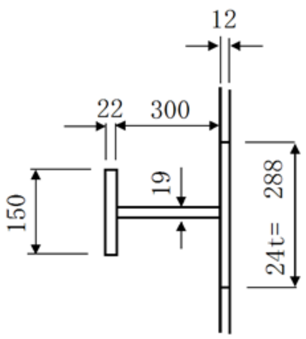


Acting force P is the coupling force due to eccentric bending at in between bracket flange

M' = 592.9 kN·m
L' = 0.532 m
P = 1114.5 kN



Design sectional force
M = 411.7 kN·m (P·a·b/L)
S = 588.1 kN (P·b/L , a<b)
L = 1.482 m



(SM490Y)		A (mm^2)	y (mm)	Ay (mm^3)	Ay^2+I (mm^4)
1 −	150 × 22	3300	161	531300	85672400
1 −	300 × 19	5700	−	−	42750000
1 −	288 × 12	3456	−156	−539136	84146688
		12456		−7836	212569088
					−4484
					212564604

e = −0.6 mm
y = 172.6 mm

σ = 411.7 × 10^6 / 212564604 × 172.6 = 334.3 N/mm^2
< 1.7× 210 = 357 N/mm^2

τ = 588.1 × 10^3 / 5700 = 103.2 N/mm^2
< 1.7× 120 = 204 N/mm^2

(334.3 / 357)^2 + (103.2 / 204)^2 = 1.13

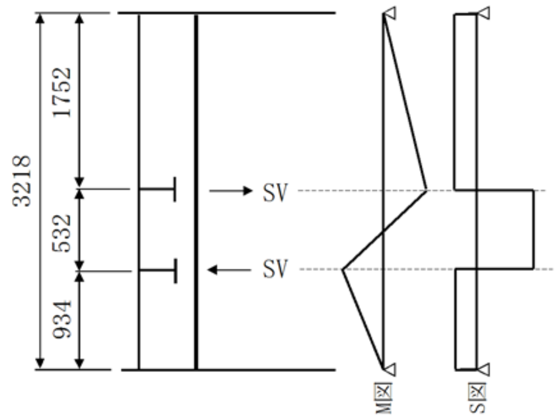
Weld at supporting point at reinforcement rib.

Necessary fillet weld leg length

Sreq = 588.1 × 10^3 / (2 × 0.707 × 300 × 204)
= 6.8 mm ⟶ 7 mm

Reinforcement calculation at supporting point of reinforcement rib.

As the reinforcement rib is the connection structure between diaphragm of end support and vertical stiffener reinforcement calculation of vertical stiffener is verified.

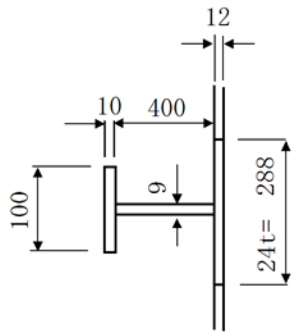


$SV = P \cdot a / L = 526.4$ kN

$M_{max} = 157.4$ kN·m

$S_{max} = 439.4$ kN

Reinforced section at stiffener



	(SM490Y)	A (mm^2)	y (mm)	A (mm^3)	$Ay^2 + I(mm^4)$
1-	100 × 10	1000	205	205000	42033333
1-	400 × 9	3600	-	-	48000000
1-	288 × 12	3456	-206	-711936	146700288
		8056		-506936	236733621
	e = -62.9				-31872839
	y = 272.9				204860782

$$\sigma = 157.4 \times 10^6 \;/\; 204860782 \times 272.9 = 209.7 \text{ N/ mm}^2$$

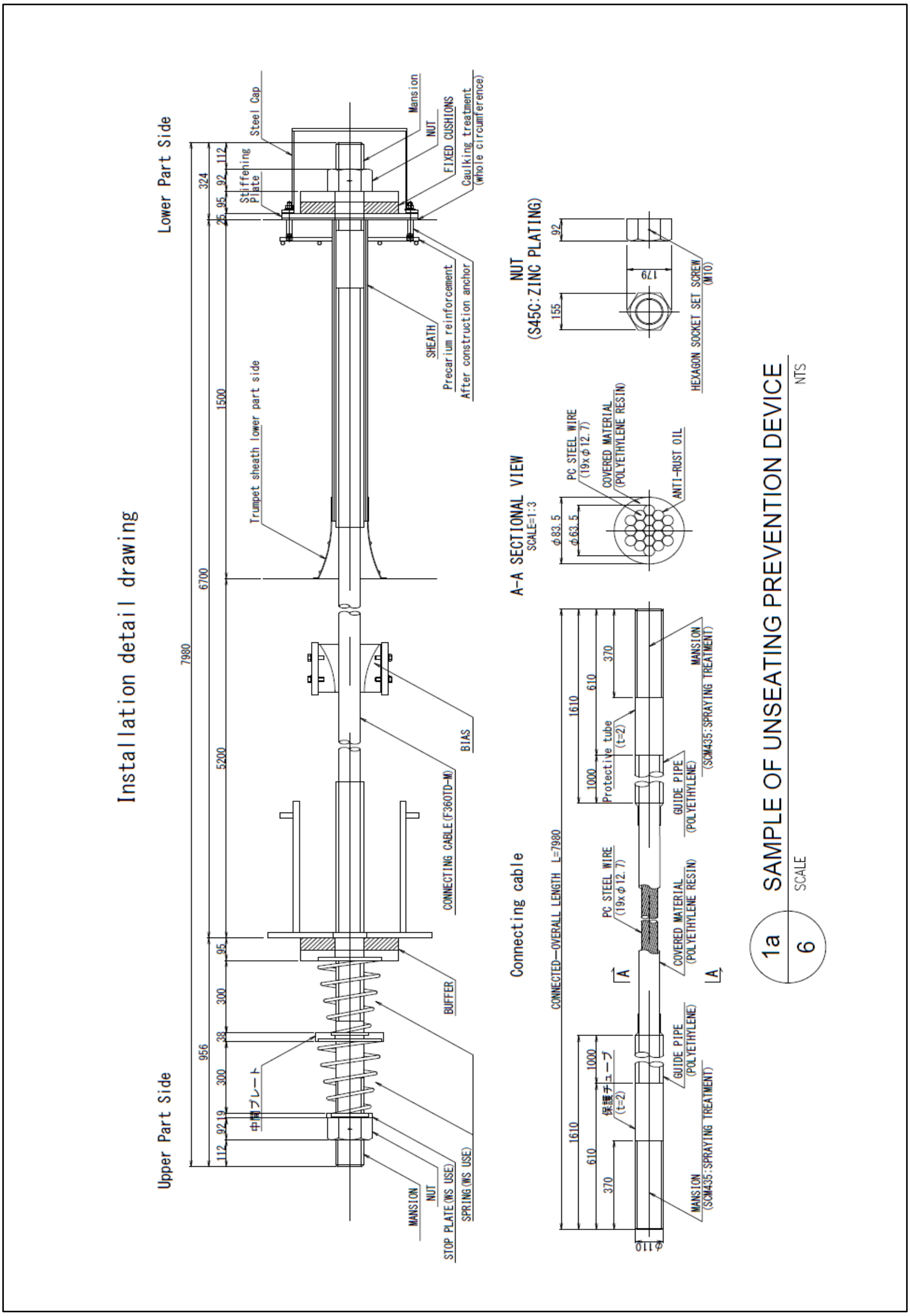
$$<1.7 \times 210 = 357 \text{ N/ mm}^2$$

$$\tau = 439.4 \times 10^3 \;/\; 3600 = 122.1 \text{ N/ mm}^2$$

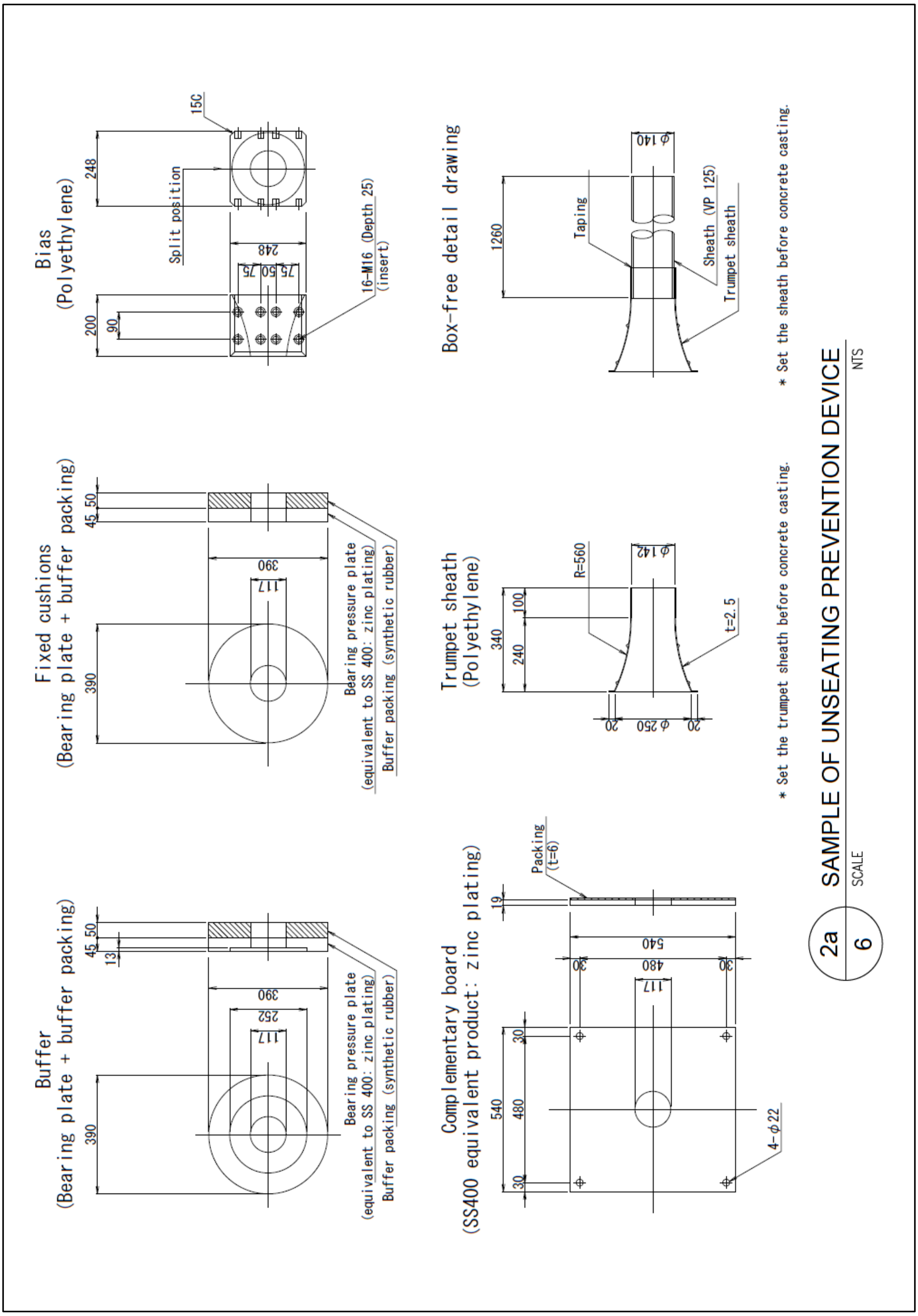
$$<1.7 \times 210 = 240 \text{ N/ mm}^2$$

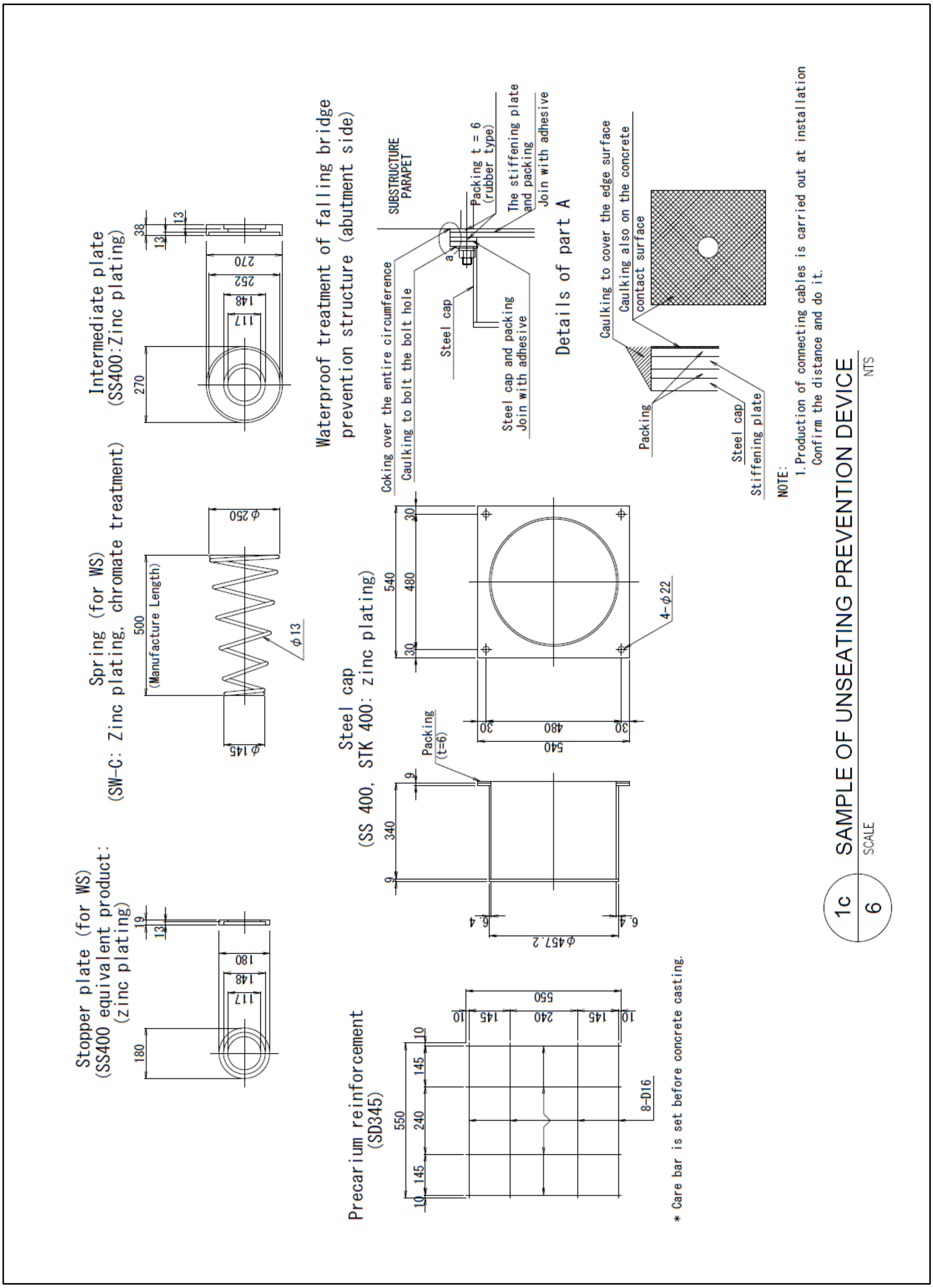
$$(209.7 / 357)^2 + (122.1 / 204)^2 = 0.7$$

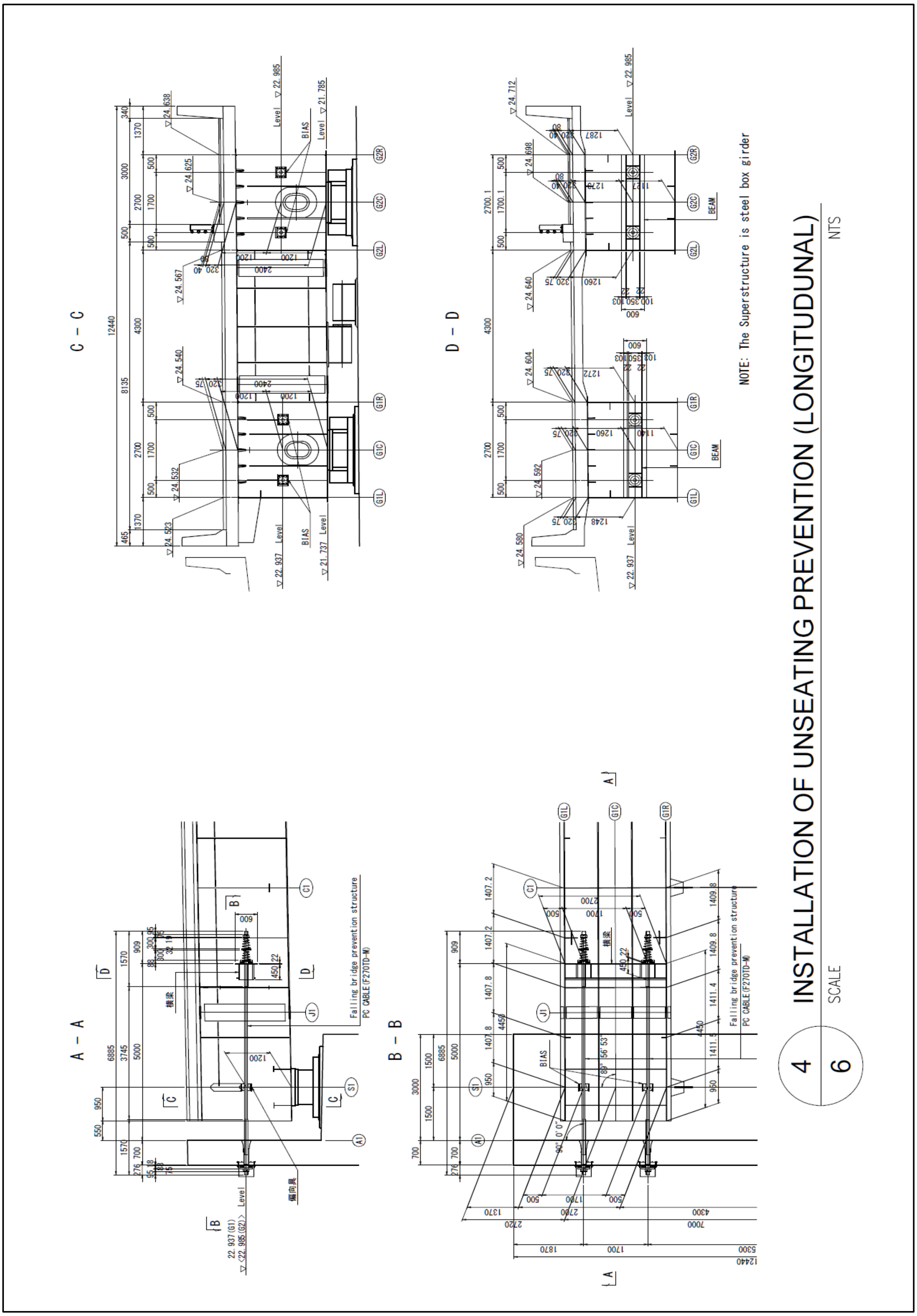
$$< 1.2$$

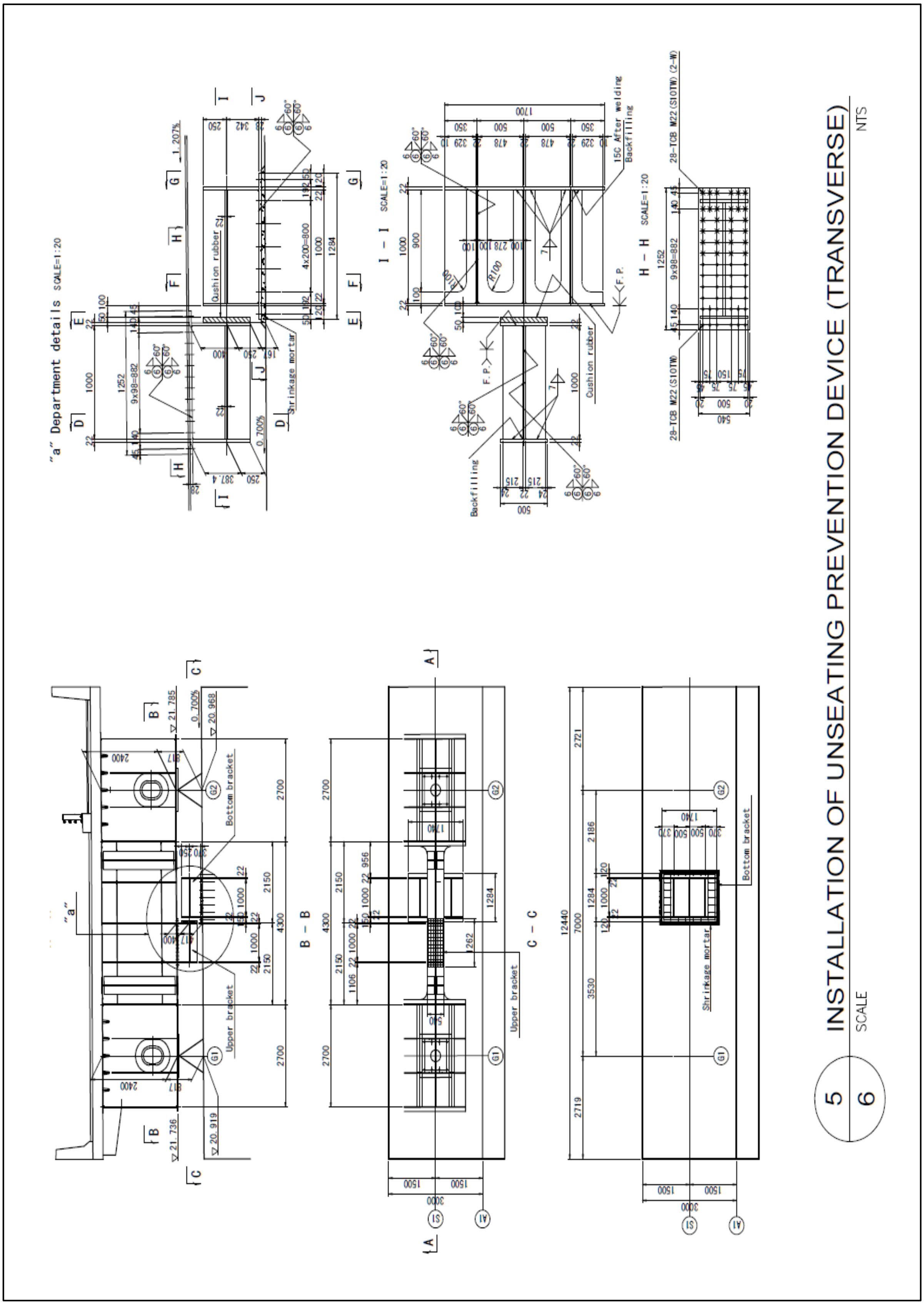


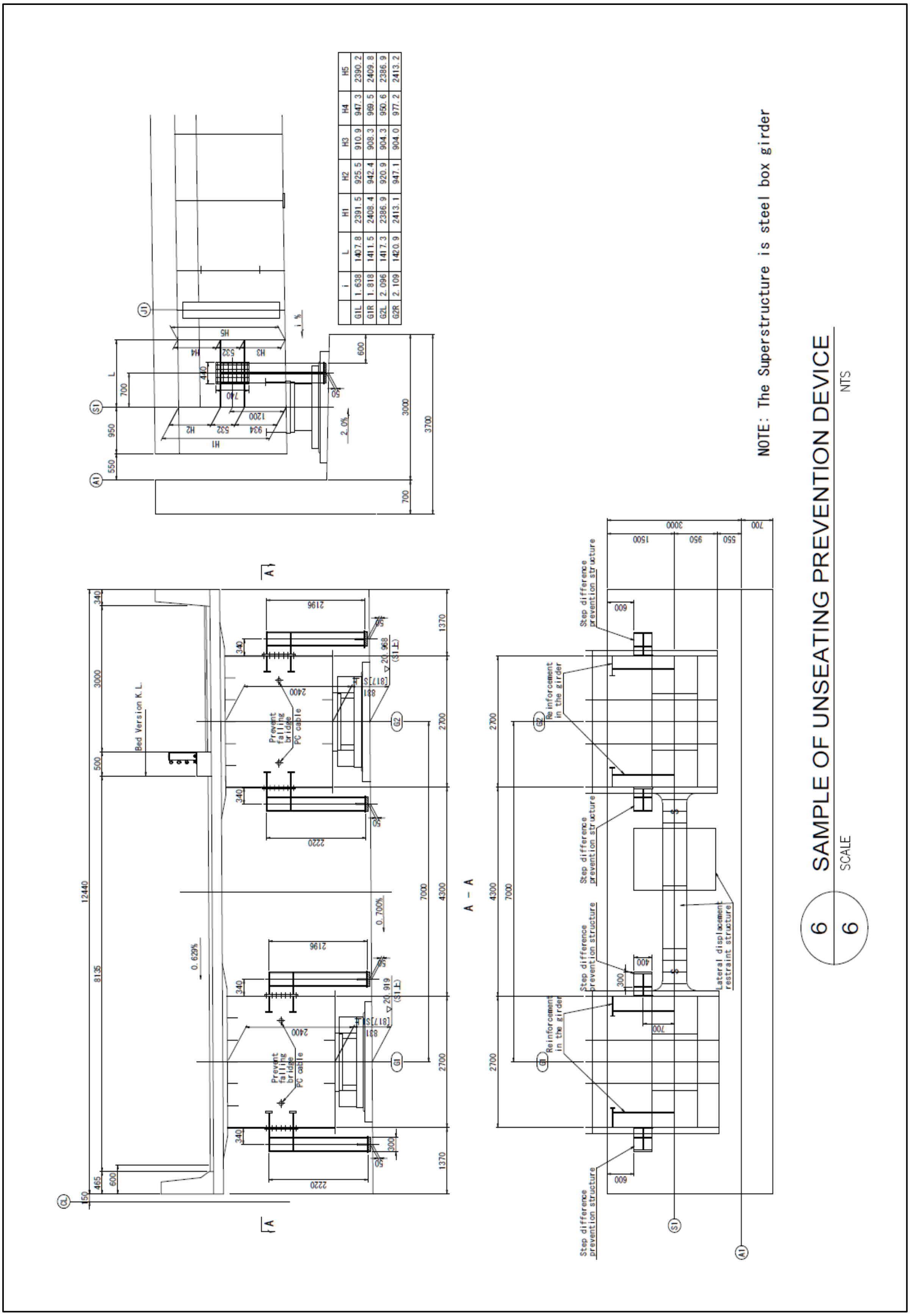
Installation detail drawing
Upper Part Side
Lower Part Side
7980
956
6700
5200
1500
324
MANSION
NUT
STOP PLATE(WS USE)
SPRING(WS USE)
中間プレート
BUFFER
CONNECTING CABLE(F360TD-M)
BIAS
Trumpet sheath lower part side
SHEATH
Precarium reinforcement
After construction anchor
Steel Cap
Stiffening Plate
Mansion
NUT
FIXED CUSHIONS
Caulking treatment (whole circumference)
Connecting cable
CONNECTED—OVERALL LENGTH L=7980
PC STEEL WIRE (19xφ12.7)
COVERED MATERIAL (POLYETHYLENE RESIN)
GUIDE PIPE (POLYETHYLENE)
MANSION (SCM435:SPRAYING TREATMENT)
保護チューブ (t=2)
Protective tube (t=2)
A-A SECTIONAL VIEW
SCALE=1:3
φ83.5
φ63.5
ANTI-RUST OIL
NUT
(S45C:ZINC PLATING)
HEXAGON SOCKET SET SCREW (M10)
1a
6
SAMPLE OF UNSEATING PREVENTION DEVICE
SCALE
NTS


Buffer
(Bearing plate + buffer packing)
Bearing pressure plate
(equivalent to SS 400: zinc plating)
Buffer packing (synthetic rubber)
Fixed cushions
(Bearing plate + buffer packing)
Bias
(Polyethylene)
Split position
16-M16 (Depth 25)
(insert)
15C
Complementary board
(SS400 equivalent product: zinc plating)
Packing
(t=6)
4-φ22
Trumpet sheath
(Polyethylene)
R=560
t=2.5
Box-free detail drawing
Taping
Sheath (VP 125)
Trumpet sheath
* Set the trumpet sheath before concrete casting.
* Set the sheath before concrete casting.
SAMPLE OF UNSEATING PREVENTION DEVICE
SCALE
NTS
2a
6


Stopper plate (for WS)
(SS400 equivalent product:
(zinc plating)
Spring (for WS)
(SW-C: Zinc plating, chromate treatment)
500
(Manufacture Length)
Intermediate plate
(SS400:Zinc plating)
Steel cap
(SS 400, STK 400: zinc plating)
Packing
(t=6)
4-φ22
Precarium reinforcement
(SD345)
8-D16
* Care bar is set before concrete casting.
Waterproof treatment of falling bridge
prevention structure (abutment side)
SUBSTRUCTURE
PARAPET
Packing t = 6
(rubber type)
The stiffening plate
and packing
Join with adhesive
Coking over the entire circumference
Caulking to bolt the bolt hole
Steel cap
Steel cap and packing
Join with adhesive
Details of part A
Caulking to cover the edge surface
Caulking also on the concrete
contact surface
Packing
Steel cap
Stiffening plate
NOTE:
1.Production of connecting cables is carried out at installation
Confirm the distance and do it.
SAMPLE OF UNSEATING PREVENTION DEVICE
SCALE
NTS
1c
6


A – A
B – B
C – C
D – D
NOTE: The Superstructure is steel box girder
Falling bridge prevention structure
PC CABLE (F270TD-M)
BIAS
BEAM
INSTALLATION OF UNSEATING PREVENTION (LONGITUDUNAL)
NTS
SCALE
4
6


INSTALLATION OF UNSEATING PREVENTION DEVICE (TRANSVERSE)
SCALE
NTS
5
6
"a" Department details SCALE=1:20
I - I SCALE=1:20
H - H SCALE=1:20
B - B
C - C
Upper bracket
Bottom bracket
Cushion rubber
Shrinkage mortar
Backfilling
15C After welding Backfilling
28-TCB M22 (S10TW)
28-TCB M22 (S10TW) (2-W)



NOTE: The Superstructure is steel box girder
SAMPLE OF UNSEATING PREVENTION DEVICE
SCALE
NTS
6
6
A - A
Bed Version K.L.
Prevent falling bridge PC cable
Step difference prevention structure
Reinforcement in the girder
Lateral displacement restraint structure

	i	L	H1	H2	H3	H4	H5
G1L	1.638	1407.8	2391.5	925.5	910.9	947.3	2390.2
G1R	1.818	1411.5	2408.4	942.4	908.3	969.5	2409.8
G2L	2.096	1417.3	2386.9	920.9	904.3	950.6	2386.9
G2R	2.109	1420.9	2413.1	947.1	904.0	977.2	2413.2

CHAPTER 9: DESIGN EXAMPLE OF SEISMIC ISOLATED BRIDGE WITH HIGH DAMPING LAMINATED RUBBER BEARING (HDR)

Chapter 9 Design Example of Seismic Isolated Bridge with High Damping Laminated Rubber Bearing (HDR)

Isolation bearings can be used to design and retrofit bridges to avoid structural damage during the most severe earthquake. The primary goal in a seismic isolation strategy is to decouple a structure from the earthquake ground motions. This strategy has been used for various bridge systems where the inertia effects of the vibrating superstructure are separated from the substructure at the interface between superstructure and substructure. This reduces the forces transmitted to the substructure columns, piers, and foundations. The earthquake energy is absorbed by heat in the isolation bearing that provides protection for the substructure.

This chapter is devoted to the applications of seismic isolation design for bridge structures. Seismic isolation analysis and design of three span bridge has been presented as an example. The used of current Bridge Seismic Design Specification (BSDS, 2013) and the Highway Bridge Seismic Isolation Design Specification (HBSIDS, 2019), the state of the practice and implementation of seismic isolation are discussed. The basic concepts, modeling and analysis methods, design, and evaluation are then explained. A design example is given for illustration purposes.

9.1 Procedure

The analysis and design procedure in this example was presented according to the following steps.

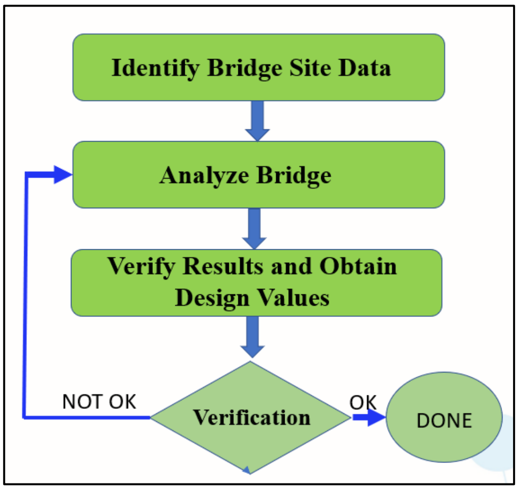


Figure 9.1-1 Seismic Isolation Design General Procedure

1. Identification of bridge data includes the following:
   - Bridge properties
   - Seismic Hazard on site
   - Required performance of isolated bridge.
2. In the analysis, following shall be defined comprehensively based on the actual condition and parameters:
   - Bridge complete model
   - Selection of isolator type and initially design for static condition to have required characteristic strength
   - Analysis method to be used.

3. After the analysis, verification of the results or response is necessary to obtain the design actions. Followings are the common structural response or actions need to be check:

    - Bearing forces and displacements
    - Hysteretic behavior of bearing isolator
    - Damping of structure
    - Design forces and displacements (for both superstructure and substructures)
    - Check if the required performance is satisfied.

4. Finally, after all the bridge target performance has been satisfied, verification of bearing stability according to its required allowable values (i.e allowable stress, allowable strain, etc..) need to be satisfied also, otherwise, iteration is necessary until all the requirements has been satisfied.

9.2 Design Condition of Example Bridge and Seismic Hazard

9.2.1 Description

A three-span continuous prestressed concrete girder bridge with a single column pier is shown in Error! Reference source not found.. This bridge is classified as essential bridge (OC2) and no skewed (regular bridge). The bridge carries two traffic lanes and superstructure width of 10.5 m. Superstructure is made of typical type 5 AASHTO girder spaced at 2.5 m. center-to-center. All columns are supported with 8 cast-in-place 1.2 diameter piles. The rectangular high damping laminated rubber bearing (HDR) are installed between the top of coping and lower end of end-diaphragm.

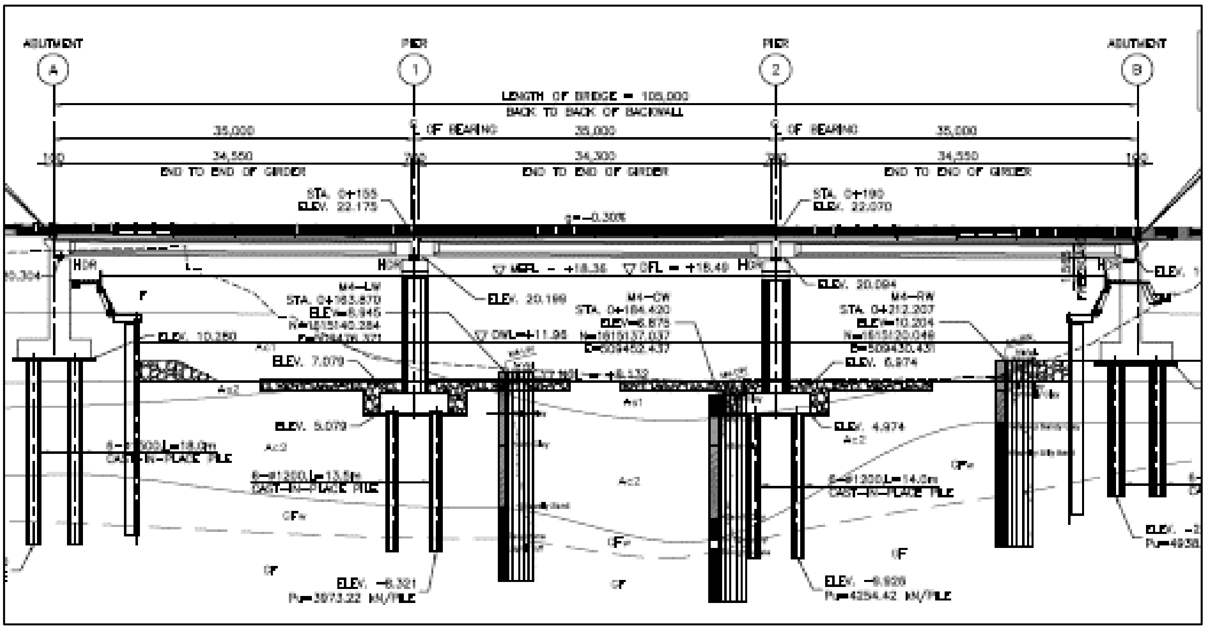


Figure 9.2-1 General Elevation of Sample Bridge

9.2.2 Seismic hazard

Seven (7) pairs of spectrally matched site-specific acceleration time history ground motion was prepared shown in **Figure 9.2-2**. The site is classified as Type II (Medium). The provisions for the generation of earthquake ground motion for dynamic analysis in Section 4.2 of this guideline referred to BSDS 2013.

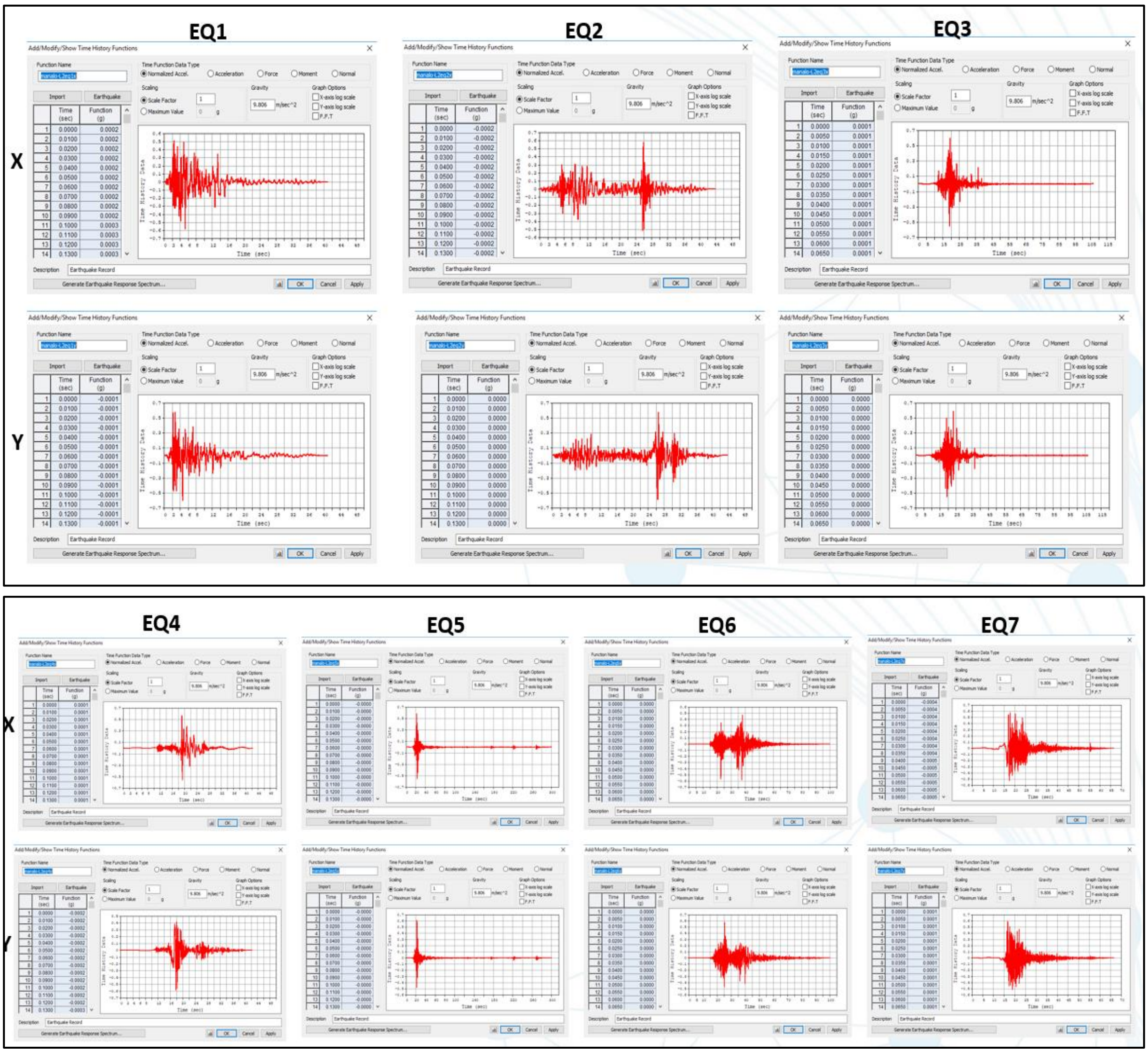


Figure 9.2-2 Seven Sets of Site-Specific Acceleration Time History Ground Motions

9.2.3 Required performance of Isolated Bridge

The required seismic performance of bridge as explained in Section 3 of this guidelines. In commentary RBSIDG mention that "*Because, a seismic isolation bearing is a member that can absorb the energy of Level 2 Earthquake Ground Motion without being damaged, a seismically isolated bridge has a structural form that is suitable for use as a bridge that needs to be restored quickly after the occurrence of an earthquake*". Meaning, that after large earthquake bridge remains at its elastic condition (SPL1), the primary plastic behavior is permissible only at bearing location as shown in **Figure 9.2-3**.

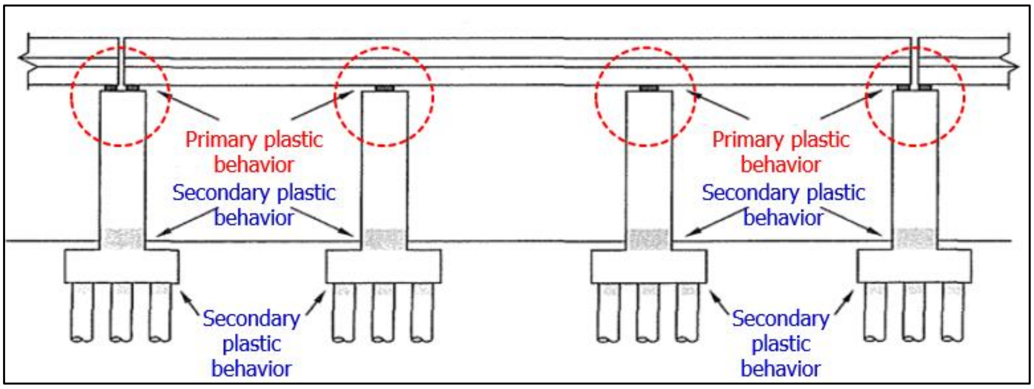


Figure 9.2-3 Permissible Plastic Hinge Location for Seismic Isolated Bridge

9.3 Analysis

Analysis procedure and analytical model was defined in Section 4.3 of this guideline. Two analysis method were recommended to be used in dynamic analysis of bridge: the response spectrum method and time history response analysis method.

9.3.1 Global Analysis Model

In modeling of bridge, Midas Civil 2018 software was utilized as a tool in this example. The bridge was modelled as grillage/3d Finite element model according to its actual geometry and properties as shown in **Figure 9.3-1**.

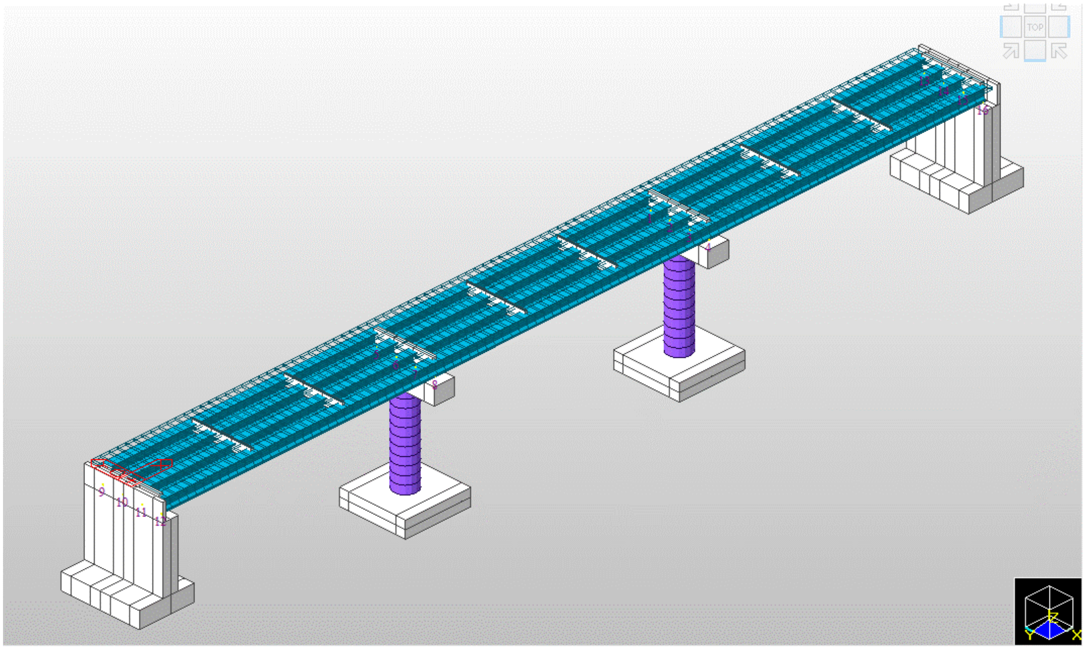


Figure 9.3-1 3D Bridge Mathematical Model

9.3.1.1 Material Properties

The material properties used for an elastic analysis are usually: modulus of elasticity, shear modulus, Poisson's ratio, the coefficient of thermal expansion, the mass density and the weight density.

9.3.1.2 Loadings

In general, there are two types of loads in bridge design: permanent loads and Transient loads. In this example, permanent load includes: Deadloads including the self-weight of a whole bridge and superimposed dead loads such as railings, wearing surface, etc. and the transient load such as earthquake ground motion.

9.3.1.3 Support Conditions

In these examples, the simplified dynamic analysis model “lumped spring model was adopted during the analysis as explained in Section 4.3.3 in BSDS 2013.
From borehole data at Pier foundation the following average N-value based on soil layer was obtained and the corresponding soil spring stiffness was calculated according to **Figure 9.3-2**.

FOR PIER 1

Layer symbol	Layer type	Layer thickness Li (m)	N-value	Vsi (m/s)	Cv	VsD (m/s)	γt (kN/m3)	G_D (kN/m2)	νD	E_D (kN/m2)
Ac	Clay	12.00	17	257	0.8	205	18.0	77188	0.5	231564
GFW	Clay	1.00	50	292	0.8	233	20.0	110793	0.5	332379

FOR PIER 2

Layer symbol	Layer type	Layer thickness Li (m)	N-value	Vsi (m/s)	Cv	VsD (m/s)	γt (kN/m3)	G_D (kN/m2)	νD	E_D (kN/m2)
Ac	Clay	11.00	15	247	0.8	197	18.0	71281	0.5	213843
GFW	Clay	1.00	50	292	0.8	233	20.0	110793	0.5	332379
GF	Sand	1.00	50	295	0.8	235	20.0	112704	0.5	338112

Pile spring stiffness, P1
Longitudinal/Transverse Direction

Type	Stiffness	Unit
Ass	3,771,748	(kN/m)
Asr,Ars	-4,774,365	(kN/rad)
Arr	37,961,700	(kN*m/rad)
Avv	3,236,400	(kN/m)

Pile spring stiffness, P2

Longitudinal/Transverse direction

Type	Stiffness	Unit
Ass	3,519,762	(kN/m)
Asr,Ars	-4,559,278	(kN/rad)
Arr	37,594,500	(kN*m/rad)
Avv	3,236,400	(kN/m)

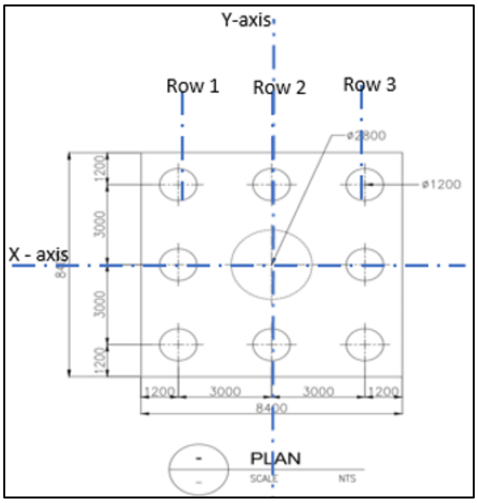
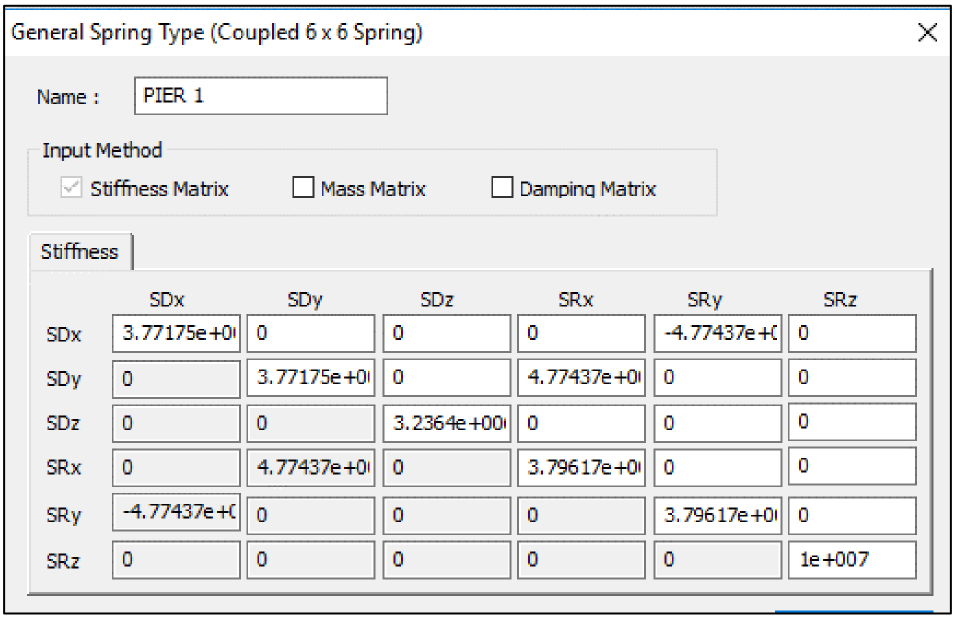


Figure 9.3-2 Piles Foundation Plan

The computed spring stiffness in this example both directions are same since the configuration of pile foundation as well as the number of piles is the same as shown in **Figure 9.3-3**.

Consideration of off diagonal spring stiffness (Asr, Ars) was also employed in modelling of spring foundation.



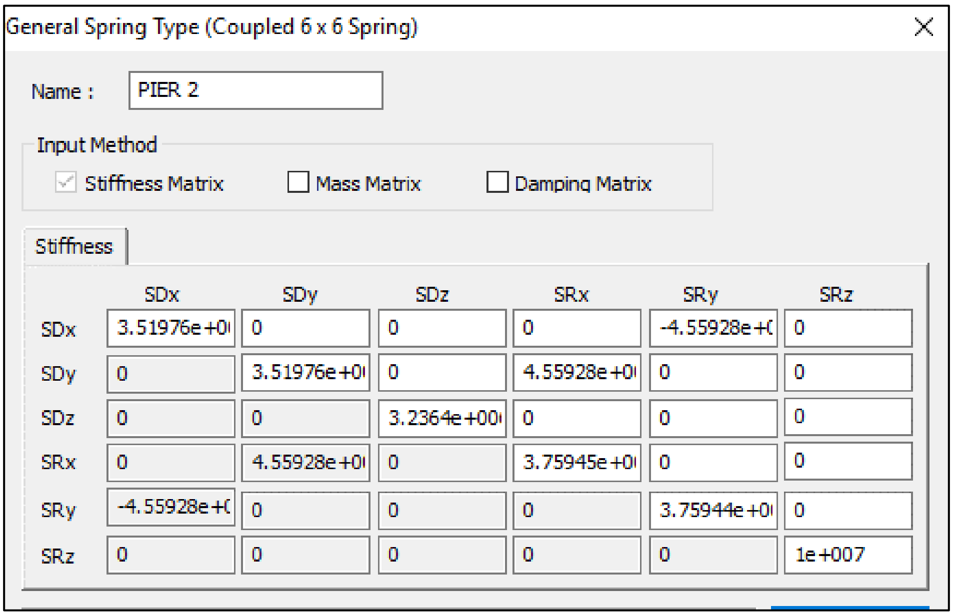


Figure 9.3-3 Soil Spring Stiffness Input in Midas Civil

9.3.2 Bearings (HDR)

High damping laminated rubber bearing (**Figure 9.3-4**) is a seismic isolation bearing using the rubber material imparting damping property to the rubber itself. Therefore, it functions as the seismic isolation bearing which combines the horizontal spring characteristics which generates the restoring force by rubber itself and the history damping performance for energy absorption.

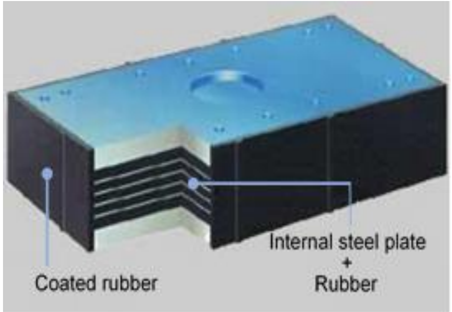


Figure 9.3-4 High Damping Laminated Rubber Bearing (HDR)

9.3.4 Mechanical Properties of HDR

In this example, G8 type rubber were chosen as given in Table 5.7.1 of this guideline. Breaking elongation of this type is 550% and Shear elastic modulus of 0.8 MPa. The allowable values to be used for the design as given in the following table:

Table 9.3-1 Allowable Value of Rubber Material

Type			Allowable value
Compressive Stress	Maximum Compressive Stress	$6 \leqq S_1 < 8$	$\sigma_{maxa} = 8.0$ (N/mm^2)
		$8 \leqq S_1 < 12$	$\sigma_{maxa} = S_1$ (N/mm^2)
		$12 \leqq S_1$	$\sigma_{maxa} = 12.0$ (N/mm^2)
	Amplitude of Stress	$6 \leqq S_1 < 8$	$\Delta\sigma_a = 5.0$ (N/mm^2)
		$8 \leqq S_1$	$\Delta\sigma_a = 5.0 + 0.375 \times (S_1 - 8.0)$ (N/mm^2) Maximum: 6.5 (N/mm^2)
Shear Strain	Normal Condition		$\gamma_{sa} = 70(\%)$
	Wind Condition		$\gamma_{wa} = 120(\%)$
	Seismic Condition (Level 2)		$\gamma_{ea} = 200(\%)$
Local Shear Strain	Ultimate Local Shear Strain		$\gamma_{ta} = \gamma_u / f_a$ γ_u : Breaking elongation shown in **Error! Reference source not found.** f_a : 1.5
Tensile Stress	Normal Condition		$\sigma_{ta} = 0$ (N/mm^2)
	Wind Condition	G8	$\sigma_{ta} = 1.2$ (N/mm^2)
		G10 and above	$\sigma_{ta} = 1.5$ (N/mm^2)
	Seismic Condition	G8	$\sigma_{ta} = 1.6$ (N/mm^2)
		G10 and above	$\sigma_{ta} = 2.0$ (N/mm^2)

Note) S_1: Primary shape factor of laminated rubber bearing calculated by Equation 6.2.5 BSDS

9.3.5 Dynamic Characteristics of HDR

The nonlinear historical characteristics of HDR was modeled by the bilinear model as shown in **Figure 9.3-6**, and the primary stiffness, secondary stiffness and yield load in the bilinear model was calculated according to Chapter 8.2 of this guideline.

In order to determine the initial characteristics of rubber bearing iterative solutions is required.

1. Determination Initial Characteristics of HDR (First Iteration)
   The first step is to choose the type of rubber to be use and compute for the bearing reaction, R_{max} based on service condition. In this example, rectangular shape has been chosen.

– Calculation of bearing reaction:

For the Abutment bearing:
R_{dl} = 730 kN (Total deadload reaction at single bearing support at the abutment)
R_{ll} = 225 kN (Governing max. live load reaction at single bearing support at the abutment)
R_{max} = 955 kN (Total reaction at a single bearing support at the abutment)

For the Pier bearing:
R_{dl} = 1460 kN (Total deadload reaction at single bearing support at Pier)
R_{ll} = 450 kN (Governing max. live load reaction at single bearing support at Pier)
R_{max} = 1910 kN (Total reaction at a single bearing support at Pier)

- Determination of Initial dimension

Initially, obtain the reasonable section by setting the allowable compressive strength equal to actual compressive stress. However, shape factor S_1 is necessary in able to define σ_{maxa} in **Table 9.3-1** and S_1 also can be determine by means of section properties of rubber bearing. So from here, assumptions may be required and a few iterations may be necessary.

Assume S_1 >12, so σ_{max} = 12 N/mm^2

$$\sigma_{max} = \frac{R_{max}}{A_e} = \sigma_{maxa}$$ (HBSIDS Eq. 6.3.1)

$A_e = \frac{(1910*1000)}{12} = 159{,}167\ mm2$ (Effective Compression Area)
Say 400 x 400 mm (a = 400 mm, b= 400 mm)

- Estimation of rubber layer thickness, A_e

To estimate the rubber thickness estimation of structure displacement may be required. In AASHTO guidelines for seismic Isolation section 7.1 simplified method has been used to estimate displacement of rubber bearing.

$$d = \left(\frac{g}{4\pi^2}\right)\left(\frac{S_{D1}T_{eff}}{B_L}\right)$$ (GSID C7.1)

Where:

$$B_L = \left(\frac{\xi}{0.05}\right)^{0.3}$$

d is the estimated structure displacement. One way to make this estimate is to assume the effective isolation period, Teff = 2.0 sec., take the viscous damping ratio to be 5% and calculate the displacement.
Then;
$d = 0.25 S_{D1}$ (m)
$d = 0.25 \times 0.698 = 0.175m$ (175 mm)
Setting $\gamma_s \leq \gamma_{sa}$, Design shear strain is equal to allowable shear strain in Table 1.2.1.

$\gamma_{sa} =$ 200% (Earthquake Condition)

$$\gamma_{ea} = \frac{d}{\sum t_e}$$ (HBSIDS Eq. 6.3.29)

$\sum t_e = \frac{175*2}{2} = 175$ mm (Estimated displacement of rubber bearing)

Try 12 mm thickness of rubber layer and 3.2mm thick of steel plate

Say 12 layers of rubber and 11 layers of steel plate in between rubber layer
Initial thickness of HDR, $\sum t_e = 179.2\ mm$
The Properties of HDR for first trial:
Type: G8
a =400 mm, b= 400mm
Thickness = 179.2 mm

At Pier Bearing using the same procedure assuming the same thickness as the abutment bearing obtained:
Try 600 x 600 mm HDR at pier location

- Determination of Characteristics of HDR

Characteristics of HDR including the primary and secondary stiffness in this example are based on its actual testing, the coefficient used according to Tables given in this guideline.

Calculation of Primary Stiffness for abutment bearings:
The primary stiffness, K1 of HDR was calculated using the equation below

$$K_1 = \frac{G_1(\gamma_{uB}) \cdot A_e}{\sum t_e} \qquad \text{(HBSIDS Eq. 8.2.1)}$$

Where:

$$G_1(\gamma_{uB}) = a_0 + a_1 \cdot \gamma_{uB} + a_2 \cdot \gamma_{uB}{}^2 + \cdot \cdot \cdot + a_i \cdot \gamma_{uB}{}^i \qquad \text{(HBSIDS Eq. 8.2.5)}$$

The coefficient, α_i of HDR are given in Table 8.2.1 of this guideline.

For HDR G8 Rubber: α_0 = 13.606, α_1 = -14.281, α_2 = 8.7294, α_3 = -2.1797, α_4 = 0.20376

$$\gamma_{uB} = \frac{175}{179.2} = 0.98$$

$G_1(\gamma_{uB}) = 13.606 + (-14.281) * .98 + 8.7294 *. 98^2 + (-2.1797) *. 98^3 + 0.20376 *. 98^4 = 6.13$
Ae = 160000 mm^2

Therefore:
$K_1 = 6.13 \frac{N}{mm^2} * \frac{160000 mm^2}{179.2 mm} = 5318.5\ kN/m$ Primary stiffness

The primary stiffness, K2 of HDR was calculated using the equation below:

$$K_2 = \frac{G_2(\gamma_{uB}) \cdot A_e}{\sum t_e} \qquad \text{(HBSIDS Eq. 8.2.2)}$$

$$G_2(\gamma_{uB}) = b_0 + b_1 \cdot \gamma_{uB} + b_2 \cdot \gamma_{uB}{}^2 + \cdot \cdot \cdot + b_i \cdot \gamma_{uB}{}^i \qquad \text{(HBSIDS Eq. 3.6)}$$

The coefficient, b_i of HDR are given in Table 8.2.2 of this guideline.
For HDR G8 Rubber: b_0 = 1.5104, b_1 = -1.5854, b_2 = 0.96921, b_3 = -0.24207, b_4 = 0.02264

$$G_2(\gamma_{uB}) = 1.5104 + -1.5854 * .98 + 0.96921 *. 98^2 + -0.24207 *. 98^3 + 0.02264 *. 98^4 = 0.68 N/mm\hat{}2$$

$K_2 = 0.68 * \frac{160000}{179.2} = 608\ kN/m$ Secondary Stiffness

Calculation of yield load, Q_y of HDR bearing.

$$Q_y = \tau_y \cdot A_e \qquad \text{(HBSIDS Eq. 3.3)}$$

Where:

$\tau_d(\gamma_{uB})$: Yield stress intensity of HDR

$$\tau_d(\gamma_{uB}) = \gamma_{uB}[G_e(\gamma_{uB}) - G_2(\gamma_{uB})] \qquad \text{(HBSIDS Eq. 8.2.8)}$$

$$G_e(\gamma_{uB}) = c_0 + c_1 \cdot \gamma_{uB} + c_2 \cdot {\gamma_{uB}}^2 + \cdot \cdot \cdot + c_i \cdot {\gamma_{uB}}^i \qquad \text{(HBSIDS Eq. 8.2.7)}$$

Coefficient c_1 is according to Table 8.2.3 (HDR is i=4)

For HDR G8 Rubber: c_0 = 2.3686, c_1 = -2.7376, c_2 = 1.7359, c_3 = -0.47343, c_4 = 0.0048822

$$G_e(\gamma_{uB}) = 2.3686 + -2.7376 * .98 + 1.7359 *.98^2 + -0.47343 *.98^3 + 0.004882 *.98^4$$
$$= 0.912N/mm\text{^}2$$

And

$$\tau_d(\gamma_{uB}) = .98 * (.912 - .68) = .227 \text{ ....Shear stress intensity}$$
$$\tau_y(\gamma_{uB}) = \frac{G_1(\gamma_{uB})}{G_1(\gamma_{uB}) - G_2(\gamma_{uB})}\tau_d(\gamma_{uB}) = \frac{6.13}{6.13 - .68} * .227 = .255$$
$$Q_y = \tau_y \cdot A_e = .255 * \frac{160000}{1000} = 41.032\ kN \text{ ..Yielding force}$$

The equivalent stiffness K_B and equivalent damping constant h_B of high damping laminated rubber bearing when modeling by the equivalent linear method shall be calculated by Equation (C8.2.1) and Equation (C8.2.3) of this guideline, respectively.

$$K_B = \frac{G_e(\gamma_{uB}) \cdot A_e}{\sum t_e} \qquad \text{(HBSIDS Eq. 8.2.1)}$$

$$K_B = .912 * \frac{160000}{179.2} = 816\ kN/m \text{ ....Equivalent Stiffness}$$

$$h_B(\gamma_{uB}) = d_0 + d_1 \cdot \gamma_{uB} + d_2 \cdot {\gamma_{uB}}^2 + \cdot \cdot \cdot + d_i \cdot {\gamma_{uB}}^i \qquad \text{(HBSIDS Eq. 8.2.3)}$$

Coefficient d_i is according to Table C8.2.1 (HDR is i=2)
For HDR G8 Rubber: d_0 =0 .21615, d_1 = -0.047991, d_2 = 0.0045171

$$h_B(\gamma_{uB}) = .21615 + -.047991 * .98 + .0045171 *.98^2 = 0.173 \text{......Equivalent Damping Constant}$$

Stiffness ratio, r = K2/K1= 0.114

For the bearing at Pier doing the same procedure, obtained the following results:

K1 = 11,966.67 kN/m

K2 = 1369.3 kN/m

Stiffness ratio, r = 0.114
Yielding Force, Qy = 92.32 kN
Equivalent Stiffness, K_B = 1836.5 kN/m
Equivalent damping constant, hb = .1735

Characteristics of Vertical Spring (Compression Spring Constant) of HDR.

$$K_v = \frac{EA_e}{\sum t_e}$$ (HBSIDS Eq. 6.2.3)

Where:

$$E = \alpha \cdot \beta \cdot S_1 \cdot G_e$$ (HBSIDS Eq. 6.2.3)

E is the Longitudinal modulus of rubber bearing.
α = 45 is the coefficient according to type Table 6.2.1 of this guideline.
β = 1.0 is the coefficient according to planar shape Table 6.2.1 of this guideline.
Then,

$$E = \alpha \cdot \beta \cdot S_1 \cdot G_e = 45 * 1 * 8.33 * 0.8 = 300\ N/mm\text{^}2$$

Therefore:

$$K_v = \frac{EA_e}{\sum t_e} = 300 * \frac{160000}{179.2} = 267857\ kN/m$$ For the Abutment Bearing

Kv = 904017 kN/m For Pier Bearing

The summary of initial Characteristics of HDR are shown in **Table 9.3-2** below. Those values, will be used in the analysis as the initial input for the in modeling linear and non-linear characteristics of HDR bearing.

Table 9.3-2 Characteristics of High Damping Rubber (G8) Bearing (Initial Input Value)

Abutment (LRB-A)			
	Vertical	**Longitudinal**	**Transverse**
Direction	**Dz**	**Dx**	**Dy**
NonLinear	No	Yes	Yes
Linear Properties			
Effective Stiffness,Ke (N/mm)	267857.14	816.2	816.2
Equivalent Damping,h_b	0.174	0.174	0.174
Non-Linear Properties			
Stiffness, K (N/mm)	-	5318.5	5318.5
Yield Force, Qy (N)	-	41032	41032
Post Yield Strength ratio	-	0.114	0.114
Pier (LRB-P)			
	Vertical	**Longitudinal**	**Transverse**
Direction	**Dz**	**Dx**	**Dy**
NonLinear	No	Yes	Yes
Linear Properties			
Effective Stiffness,Ke (N/mm)	904017.86	1836.5	1836.5
Equivalent Damping,h_b	0.174	0.174	0.174
Non-Linear Properties			
Stiffness, K (N/mm)	-	11966.6	11966.6
Yield Force, Qy (N)	-	92322	92322
Post Yield Strength ratio	-	0.114	0.114

9.3.6 Analysis Method

Since most of the isolation systems are non-linear, it might appear at first sight that only non-linear analysis methods can be used in the design (such as Non-linear time history method). However, if the non-linear can be linearized, equivalent linear (elastic) methods may be used, in which case many methods are suitable for isolated bridges. These methods include:

- Uniform load Method
- Single Mode Spectral method
- Multi-mode Spectral method
- Time-History method

The first three method are elastic methods. The time history method may be either elastic or inelastic. It is required to use for complex structures or where explicit modeling of energy dissipation is required top better represent isolation systems with high level of hysteretic damping. A variation of simplified method such as Uniform load method (ULM) is the displacement-based method of analysis which is particularly useful for performing initial designs and checking the feasibility of isolation for a particular bridge. It may be used as a starting point in design, followed by more rigorous methods as the design progresses.

The Multi-Mode Spectral method as the minimum requirements recommended by this guideline for equivalent linear analysis is the same as specified in the LRFD BSDS (2013) and LRFD Seismic Guide (2011) using the 5% damping ground motion response spectra with the following modifications:

1. The isolation bearings are represented by their effective stiffness values.
2. The response spectrum is modified to incorporate the effect of higher damping of the isolated system. This results in a reduction of the response spectra values for the isolated modes. For all the other modes, the 5% damping response spectra should be used.
3. A typical modified response spectrum is shown in **Figure 9.3-5**.

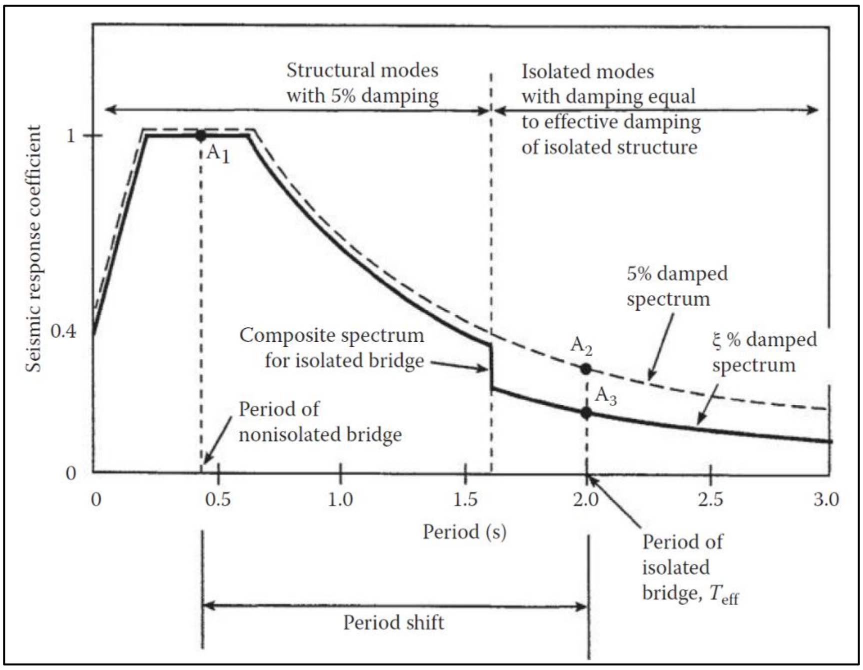


Figure 9.3-5 Modified Design Response Spectrum for Isolated Bridge (Chen Et Al. 2014)

However, using equivalent linear method, the actual behavior of bearing cannot be defined explicitly and the comparison from the actual load test maybe rough. In this example, time history analysis by modal superposition was performed using Midas Civil 2018 as an analysis tool. Sensitivity of the analysis has been explained including the HDR characteristics and hysteretic model and parameters.

Non-linear analysis has been performed by applying non-linear characteristics of bearing. This analysis method is called the "Boundary Non-linear time history analysis". Boundary nonlinear time history analysis, being one of nonlinear time history analyses, can be applied to a structure, which has limited nonlinearity. The nonlinearity of the structure is modeled through General Link of Force Type, and the remainder of the structure is modeled linear elastically. Boundary nonlinear time history analysis is analyzed by converting the member forces of the nonlinear system into loads acting in the linear system. Because a linear system is analyzed through modal superposition, this approach has an advantage of fast analysis speed compared to the method of direct integration, which solves equilibrium equations for the entire structure at every time step. The equation of motion for a structure, which contains General Link elements of Force Type, is as follows:

$$M\ddot{u}(t) + C\dot{u}(t) + (K_s + K_n)u(t) = B_p p(t) + B_N(f_L(t) - f_N(t)) \qquad \text{(Equation 1.2.1)}$$

Where:

M : *Mass* Matrix

C : Damping Matrix

K_S : Elastic Stiffness without General Link element of Force type

K_N : Elastic Stiffness of General Link element of Force type

B_P, B_N : Transformation Matrix

$u(t)$, $\dot{u}(t)$, $\ddot{u}(t)$: Nodal displacement, velocity, and acceleration

p(t) : Dynamic load

$f_L(t)$: Internal forces due to effective stiffness of non-linear components contained in general link elements of force type

$f_N(t)$: True internal forces of non-linear components contained in general link

9.3.6.1 Energy Dissipation

Although the low horizontal stiffness of seismic isolators leads to reduced seismic forces, it may result in larger superstructure displacements. Wider expansion joints and increased seat lengths maybe required to accommodate these displacements. As a consequence, most isolation systems include the energy dissipation mechanism to introduced a significant level of damping into the bridge to limit these displacements to acceptable levels. These mechanisms are frequently hysteretic in nature, which means that there is an offset between the loading and the unloading force-displacement curves under reverse (cyclic) loading. Energy, which is not recovered during unloading, is mainly dissipated as heat from the system. Following Figure shows a bilinear force-displacement relationship for a typical seismic isolator that includes an energy dissipater. The hatched under the curve is the energy dissipated during each cycle of motion of the isolator.

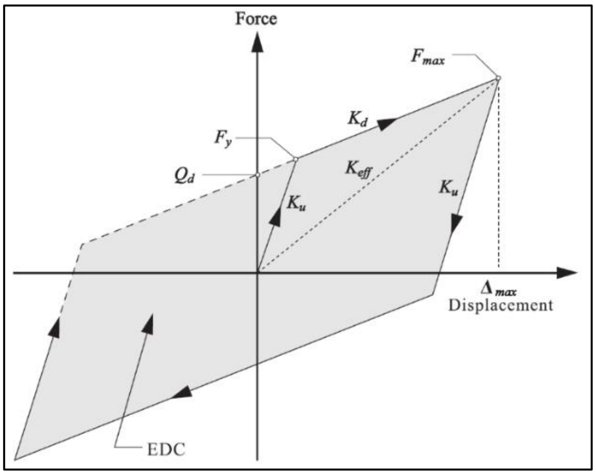


Figure 9.3-6 Billinear Hysteresis Loop (Aashto 1999)

Q_d = Characteristic strength
F_y = Yield force
F_{max} = Maximum force
K_d = Post-elastic stiffness
K_u = Elastic (unloading) stiffness
K_{eff} = Effective stiffness
Δ_{max} = Maximum bearing displacement
EDC = Energy dissipated per cycle = Area of hysteresis loop (shaded)

Analytical tools for these non-linear systems are available using inelastic time history structural analysis software packages. But these tools can be unwieldly use and not always suitable for routine design office use. Simplified methods such as equivalent linear analysis has therefore been developed which use effective elastic properties and an equivalent viscous dashpot to represent the energy dissipation. The effective stiffness, K_e and effective damping constant, h_b in Error! Reference source not found. has been used.

9.3.6.2 Hysteretic System

There are several hysteresis models in modeling the dynamic behavior of high damp rubber that was developed in past years e.g Bouc-Wen model (Wen, 1976) that was developed by Tsai Model (Tsai et al, 2003), Huang Model (Huang et al, 2002) and many more, until now many researchers are still developing and modifying the model of characteristics of high damp rubber bearing. Tsai et al. recently proposed a force -displacement model for HDR bearings based on the Bouc-Wen hysteretic model. The tangent stiffness in the bilinear curve, D (Tsai et al. 2003) may be expressed as:

$$D = \gamma K + (1-\gamma)K\left[A - \left(\alpha\,\mathrm{sgn}\left(\dot{U}Z\right) + \beta\right)Z^2\right] \qquad \text{(Equation 1.2.2)}$$

Where A, α, β, are material constants; recommended values are 1.0, 0.1, 0.9, respectively according to Tsopelas et al., 1994 for modeling HDR. The terms K and Ƴ are the initial stiffness and plastic stiffness ratio, respectively. To verify the tools capability of performing such hysteresis model, in Midas technical manual, and accordingly, the following Equation 1.2.3 were verified.
Hysteretic system based on Wen model consists of 6 independent components having the properties of Uniaxial Plasticity. The system is used to model Energy Dissipation Device through hysteretic behavior. The force-deformation relationship of Hysteretic System by components is as follows:

$$f = r \cdot k \cdot d + (1 - r) \cdot F_y \cdot \mathrm{z} \qquad \text{(Equation 1.2.3)}$$

Where:

k : *Initial* Stiffness

F_y : Yield Strength

r : Post-yield stiffness reduction

d : Deformation between two nodes

z : Internal variable for hysteretic behavior

z is an internal hysteretic variable, whose absolute value ranges from 0 to 1. The dynamic behavior of the variable z was proposed by Wen (1976) and defined by the following differential equation:

$$\dot{z} = \frac{k}{F_y}\left[1 - |z|^s \left\{\alpha\,\mathrm{sgn}\left(\dot{d}z\right) + \beta\right\}\right]\dot{d} \qquad \text{(Equation 1.2.4)}$$

Where:

α, β : *P*arameters determining the shape of hysteretic curve

s : Parameters determining the magnitude of the yield strength transition region

$\dot{d}$: Rate of change in deformation between two nodes

α and β are the parameters determining the post-yield behavior. α+β>0 signifies Softening System, and α+β<0 signifies Hardening System. The energy dissipation due to hysteretic behavior increases with the increase in the closed area confined by the hysteretic curve. In the case of Softening System, it increases with the decrease in the value of (β-α). The change of hysteretic behavior due to the variation of α and β is illustrated in **Figure 9.3-7**. s is an exponent determining the sharpness of the hysteretic curve in the transition region between elastic deformation and plastic deformation, i.e. in the region of yield strength. The larger the value, the more distinct the point of yield strength becomes and the closer it is to the ideal Bi-linear Elasto-plastic System. The change of the transition region due to s is illustrated in **Figure 9.3-8**.

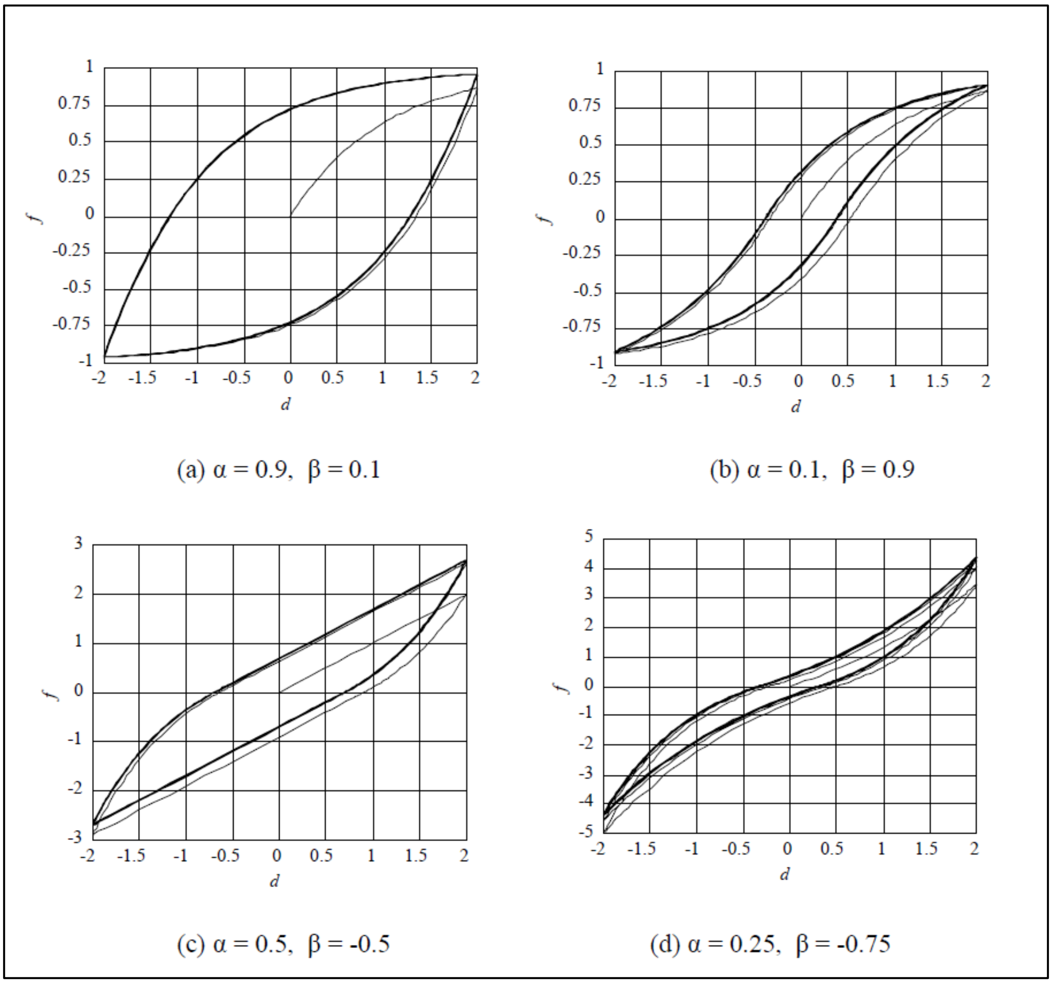


Figure 9.3-7 Force-Deformation Relationship Due to Hysteretic Behavior (R = 0, K = Fy = S = 1.0)

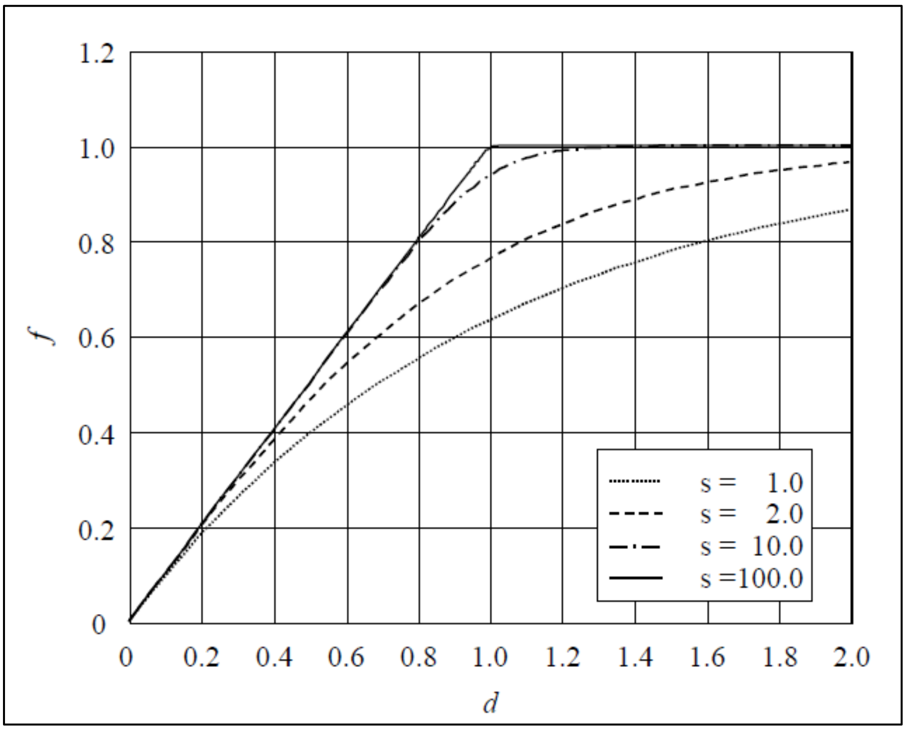


Figure 9.3-8 Transition Region Between Elastic and Plastic Deformations (Yield Region)

9.3.7 Practical Modeling and Analysis

Example bridge mathematical model considering the non-linear boundary element as shown in **Figure 9.3-9**.

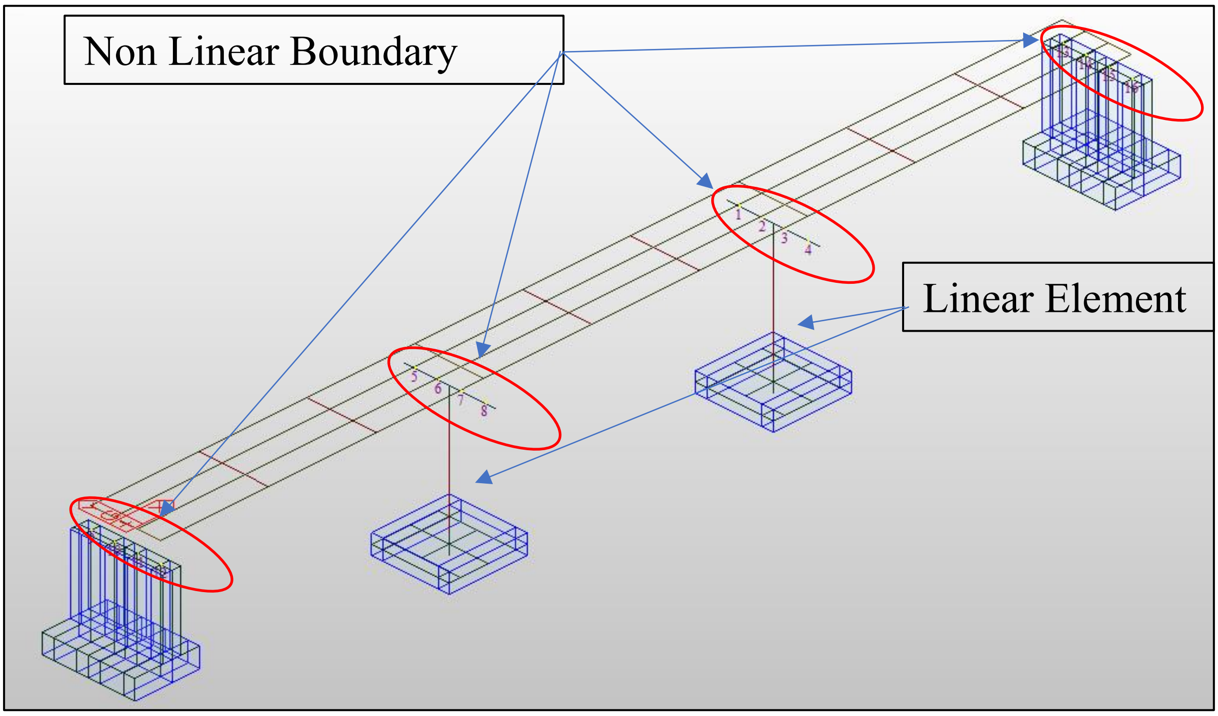


Figure 9.3-9 Example Bridge Mathematical Model

9.3.7.1 Time History Analysis Application

The bridge was modeled according to its actual condition and configuration, considering the target performance according to Chapter 9.1.3 of this guideline. In this example the bridge is classified as Essential bridge or OC2 in which the minimum performance level requirements is SPL2 *or Limited damage for function recovery*, however since the isolation has been applied, we expect much higher performance and the target performance of this bridge is now SPL 1 or *Damage prevention*. Meaning, No structural damage during level 2 earthquake, the behavior of pier still at elastic range. In short, the bridges are expected to perform its normal function or be open to all traffic under level 2 earthquake.

Considering the performance requirements, in these examples, all elements of the bridge was modeled as linear/elastic except that for bearing which is non-linear boundary elements. The bearing support was modeled as a non-linear hysteretic system as explained in Chapter 9.2.3.2 of this guideline.

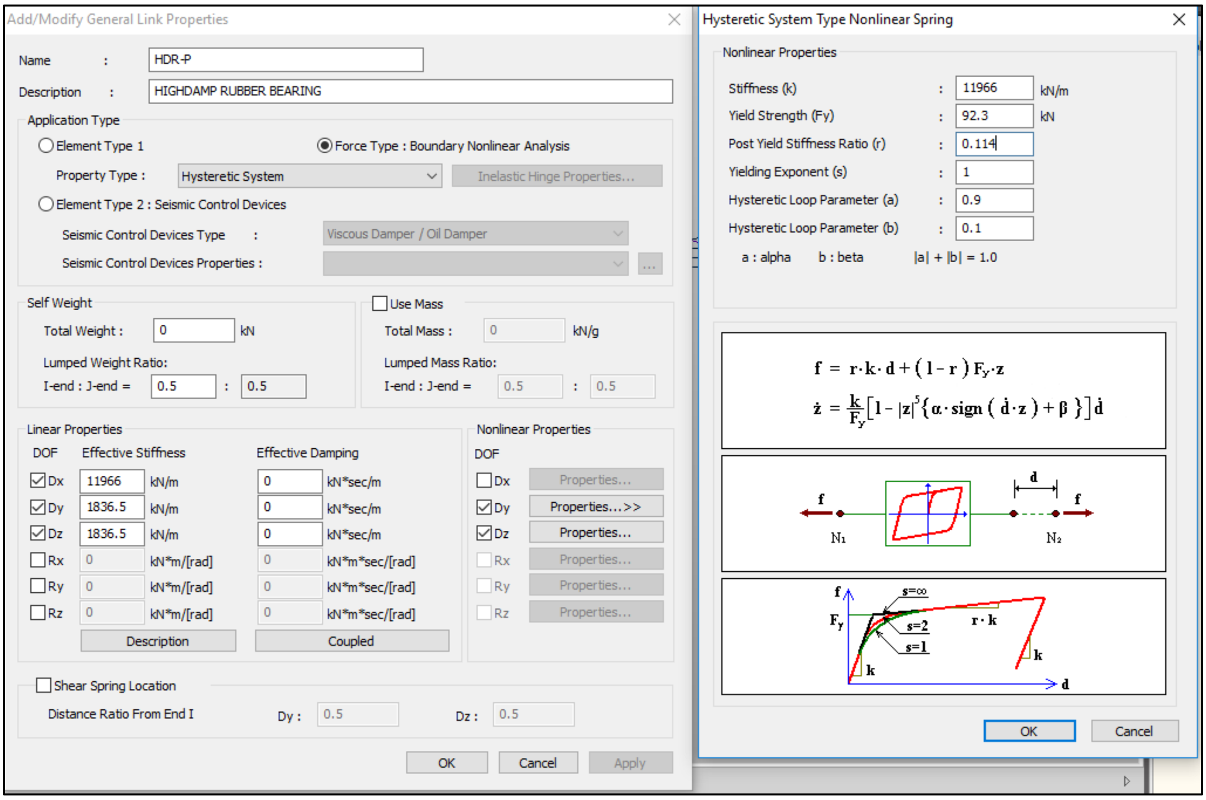


Figure 9.3-10 Sample Bearing Input Parameters for Hysteretic System

9.4 Design Seismic Forces for Verification of Bearing Support

9.4.1 Design Seismic force for verification of bearing support

The design seismic force for verification of bearing was explained in Section 5.4 of this guideline. The downward and upward design vertical seismic forces of the bearing supports shall be calculated from HBSIDS Equation 5.4.1 and 5.4.2, respectively.

$$R_L = R_D + \sqrt{{R_{HEQ}}^2 + {R_{VEQ}}^2} \qquad \text{(HBSIDS Eq. 5.4.1)}$$

$$R_U = R_D - \sqrt{{R_{HEQ}}^2 + {R_{VEQ}}^2} \qquad \text{(HBSIDS Eq. 5.4.2)}$$

Where,

R_L : Downward seismic force used for seismic design of bearing support (kN)

R_U : Upward seismic force used for seismic design of bearing support (kN)

R_D : Reaction force generated at the bearing supports by the dead load of the superstructure (kN).

R_{HEQ} : Horizontal reaction force (kN) generated at the bearing supports.

R_{VEQ} : Vertical seismic force (kN) generated by the design vertical seismic coefficient kv which is obtained from the following Equation.

$$R_{VEQ} = \pm k_V R_D \qquad \text{(HBSIDS Eq. 5.4.3)}$$

k_V : Design vertical seismic coefficient; it is obtained by multiplying the design horizontal seismic coefficient on the ground surface by a factor specified in RBSIDG 5.4.1 the design horizontal seismic coefficient for Level 1 Earthquake Ground Motion is specified in Clause 3.6 and Appendix 3A in BSDS, and the design horizontal seismic coefficient for Level 2 Earthquake Ground Motion is specified in Clause 3.6 and Appendix 3B in BSDS.

From RBSIDG Table 5.4.1 Multiplying coefficient for the design horizontal coefficient for Level 2 EQ Type 2 ground is 0.67.
Then, k_v = γFpga = 0.67*0.883 = 0.5916
Where: Fpga = 0.881 (Calculated from given hazard Map)
The vertical and horizontal seismic forces for the verification of bearing explained in RBSIDG C5.4 as shown in **Figure 9.4-1**.

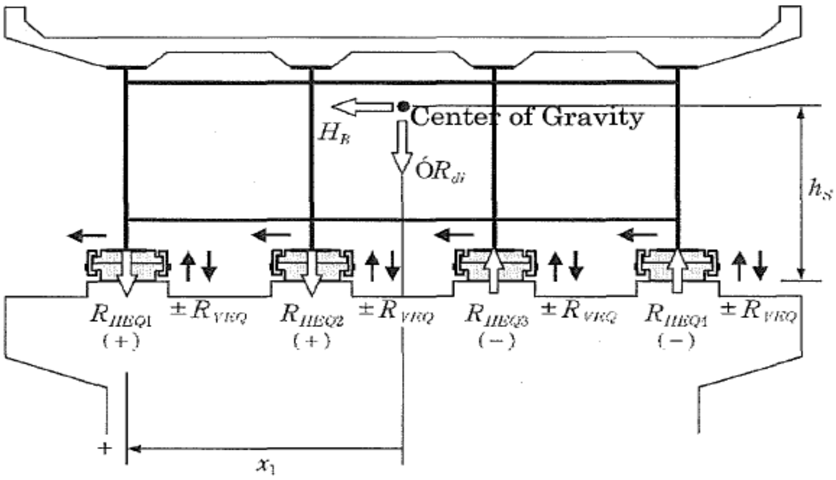


Figure 9.4-1 Vertical Reaction Force R_{heq} Generated in Bearing Support Due to Horizontal Seismic Force & Vertical Reaction Force R_{veq} Generated in Bearing Support Due to Vertical Seismic Force

$$R_{HEQi} = \frac{H_B h_s}{\sum x_i^2} x_i$$

(HBSIDS Eq. 5.4.2)

Where:

$$\left.\begin{aligned} H_B h_s &= \sum \left(R_{HEQi} x_i\right) \\ \sum R_{HEQi} &= 0 \\ R_{HEQi} &= K\left(x_i - x_0\right) \end{aligned}\right\}$$

(HBSIDS Eq. 5.4.1)

Where,

R_{HEQi} : Reaction force generated in the i-th bearing supports when the design horizontal seismic force acts in the transverse direction to bridge axis (kN)

H_B : Design horizontal seismic force of bearing support specified in (1) and (2) RBSIDG C5.4

h_s : Ve1tical distance from the bearing seat surface to the gravity center of the superstructure (m). The maximum value of h_s on the bearing support line shall be used, when there is a level difference in the seat surfaces on one bearing support line.

x_i : Horizontal distance from the gravity center of the superstructure to the i-th bearing support. Both positive and negative values shall be considered.

K : A coefficient representing a proportional relationship. It can be obtained from RBSIDG Equation (C5.4.1).

x_0 : Distance from the balancing point of R_{HEQi} to the center of gravity (m). However, it becomes 0 when the center of gravity is in the center of the symmetrical section in the transverse direction to bridge axis.

Thus, for the example bridge given the dimension, using Equation 5.4.2 R_{HEQ} was computed as shown in table below

No.	x_i(m)	x_i^2	x_0	R_{HEQi}(kN)
1	5	25	0	119.9110629
2	2.5	6.25	0	59.95553143
3	-2.5	6.25	0	59.95553143
4	-5	25	0	119.9110629

No.	x_i(m)	x_i^2	x_0	R_{HEQi}(kN)
1	5	25	0	110.14704
2	2.5	6.25	0	55.07352
3	-2.5	6.25	0	-55.07352
4	-5	25	0	-110.14704

Computed the bearing reaction due to total superstructure weight, R_D = 1461.75 kN
Then the vertical seismic force, R_{VEQ} = 0.5916*1461.75 = 864.77 kN
From Equation 5.4.1 and 5.4.2 the Design downward and upward bearing forces for verification of bearing was obtained:

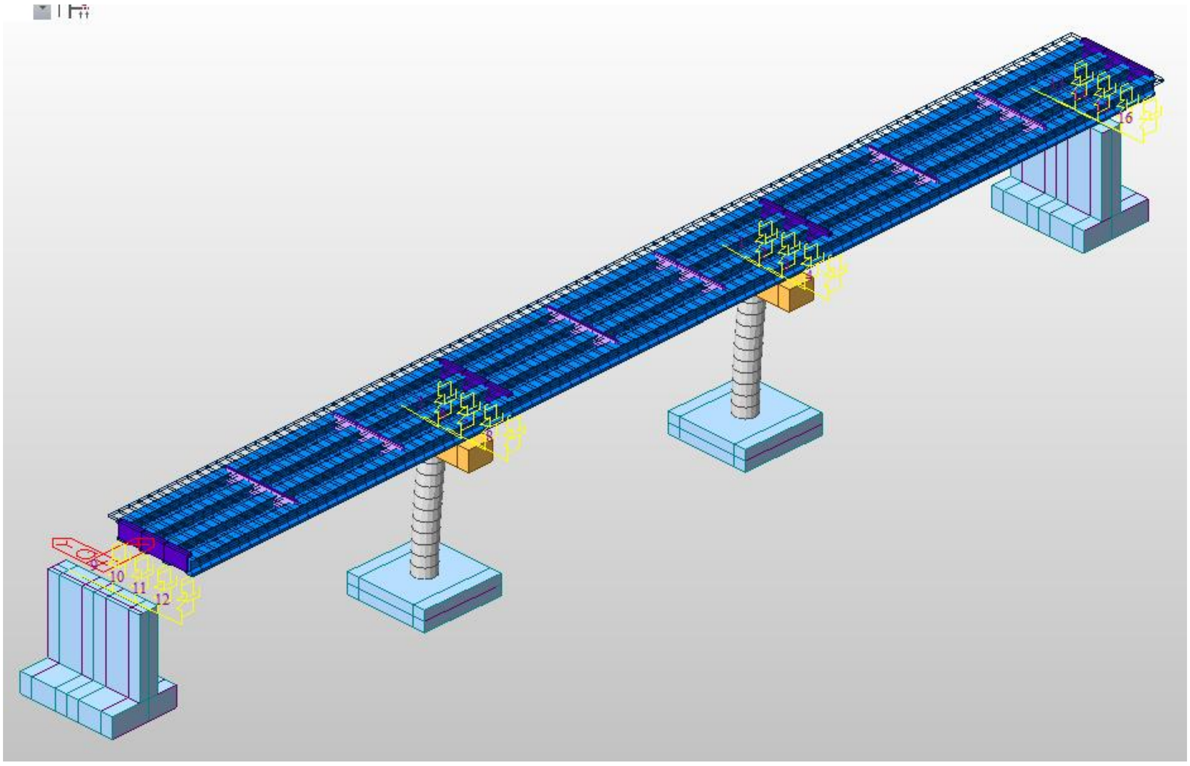
PIER BEARING FORCE	
R_L(kN)	R_U(kN)
2335.133126	588.3668742
2328.937458	594.5625421
2328.937458	594.5625421
2335.133126	588.3668742

ABUTMENT BEARING FORCE	
R_L(kN)	R_U(kN)
1177.234973	284.5150267
1166.923103	294.8268972
1166.923103	294.8268972
1177.234973	284.5150267

9.4.2 Verification of Analysis Output

Based on the results of the analysis from the initial bearing model (First iteration), the results would be as follows:

(1) Fundamental modes of the bridge
1st Mode (Longitudinal Direction)
Period, Tn (secs) = 2.128 secs.



2nd Mode (Transverse Direction)
Period, Tn = 1.0 sec.

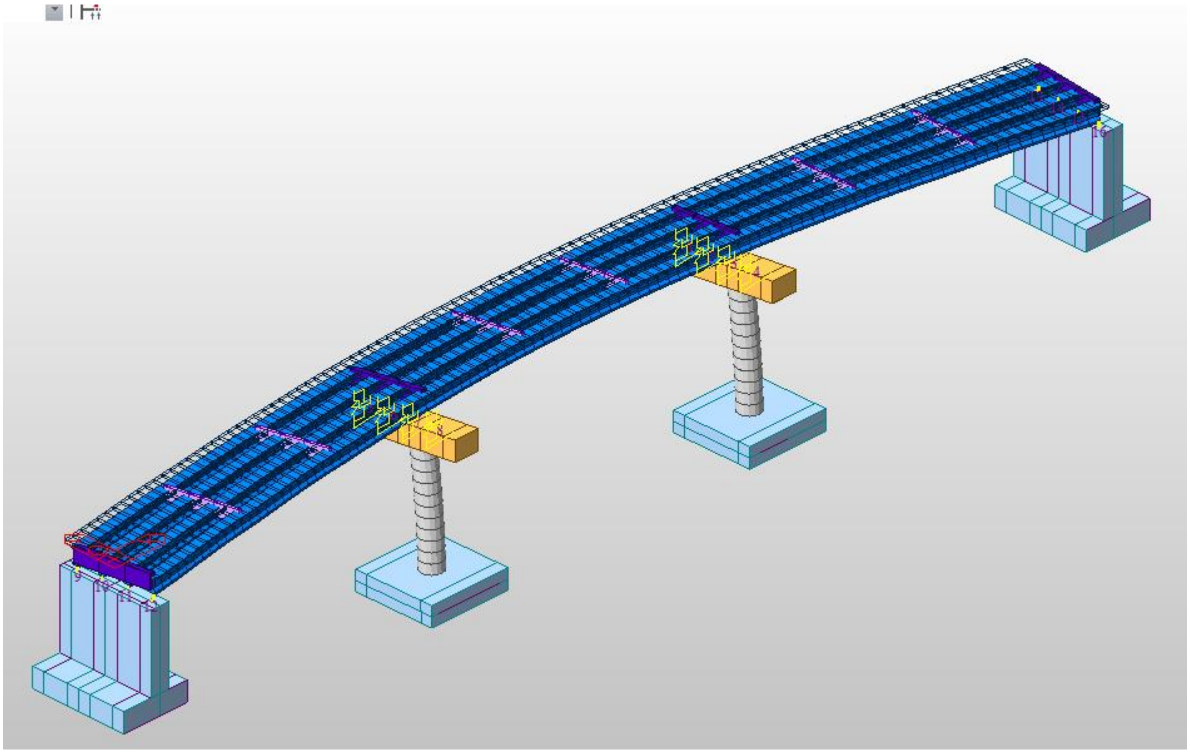


Figure 9.4-2 Fundamental Modes of Bridge

(2) Verification of Bearing Displacement

At Pier Bearing, the mean response of each bearing due to seven (7) input ground motion.

For example, at Bearing 1 at Pier:

EQ1X(max)	0.161837	0.065103	6.2E-05	1.8E-05	-0.004323	0.00133
EQ2X(max)	0.132388	0.061618	0.000103	2.9E-05	-0.004526	0.001499
EQ3X(max)	0.147581	0.080256	7.4E-05	2.1E-05	-0.004708	0.001512
EQ4X(max)	0.134333	0.07727	0.00011	2.9E-05	-0.003345	0.001151
EQ5X(max)	0.15311	0.084072	0.000109	2.9E-05	-0.004973	0.001607
EQ6X(max)	0.126706	0.071961	0.000259	7.1E-05	-0.00307	0.001403
EQ7X(max)	0.162262	0.078688	0.000446	0.000115	-0.004998	0.001546
Mean	**0.16205**	**0.0718955**	**0.000254**	**6.65E-05**	**-0.004661**	**0.001438**

Then the design isolation displacement = 162 mm

Same for the abutment bearings:

EQ1(max)	0.204518	0.000068	0.002986	0.000787	0.000153	0.002633
EQ2(max)	0.189208	0.000066	0.003451	0.000909	0.000115	0.002482
EQ3(max)	0.180622	0.000087	0.003654	0.000972	0.000156	0.003148
EQ4(max)	0.163937	0.000082	0.002622	0.000703	0.000132	0.003222
EQ5(max)	0.188674	0.000094	0.003599	0.000957	0.000191	0.003142
EQ6(max)	0.156008	0.000076	0.003012	0.000796	0.000094	0.002942
EQ7(max)	0.213772	0.000081	0.003698	0.000981	0.000115	0.003066
Mean	**0.185248**	**7.914E-05**	**0.003289**	**0.000872**	**0.0001366**	**0.002948**

And for the abutment bearing, the design displacement is 185 mm

(3) Verification of Superstructure Displacement

Load	DX (m)	DY (m)	DZ (m)	RX (rad)	RY (rad)	RZ (rad)
EQ1(max)	0.20563	0.001402	0.00102	0.00786	0.00149	0.002623
EQ2(max)	0.184656	0.001291	0.001179	0.00909	0.00121	0.002473
EQ3(max)	0.188201	0.001585	0.001228	0.00971	0.00152	0.003136
EQ4(max)	0.166516	0.001718	0.000864	0.00703	0.00127	0.00321
EQ5(max)	0.196613	0.001793	0.00121	0.00957	0.00185	0.003129
EQ6(max)	0.157288	0.001437	0.001025	0.00796	0.00096	0.002932
EQ7(max)	0.214142	0.001602	0.001247	0.00981	0.00113	0.003056
	0.187578	**0.0015469**	**0.00111**	**0.00872**	**0.001347**	**0.002937**

Say 188 mm (Displacement of Superstructure)

(4) Design Bearing Horizontal Seismic Force

For Pier Bearing:

Axial (kN)	Shear-y (kN)	Shear-z (kN)
903.1914	832.715714	286.2757143

For Abutment Bearing

Axial (kN)	Shear-y (kN)	Shear-z (kN)
840.6214	764.91	243.022857

9.5 Design of High-Damping Rubber Bearing

In the design of HDR, two performances need to be verified 1. At Normal Condition and 2. During Earthquake.
Initial Bearing Size = 600 x 600 x 179.2 mm (Ae = 360,000)……………For Pier
Initial Bearing Size = 400 x 400 x 179.2 mm (Ae = 160,000)……………For Abutment
Rubber type: G8, Ge = 0.8 Mpa
Thickness of rubber layer = 12 mm
Thickness of steel plate = 3.2 mm

9.5.1 Design of Pier Bearing

(1) At Normal Condition
Maximum Compressive stress

$$\sigma_{max} = \frac{R_{max}}{A_{cn}} \le \sigma_{maxa}$$

$$Shape\ factor, S1 = \frac{360000}{2(600 + 600) * 12} = 12.5 > 12$$

Therefore: $\sigma_{maxa} = 12\ Mpa\ (Table\ 5.7.2)$

$$\sigma_{max} = \frac{R_{max}}{A_{cn}} = \frac{Rll + Rdl}{360000} = \frac{(1461.75 + 450.88)kN * 1000N}{360000} = 5.31\ Mpa \le \sigma_{maxa}(OKAY)$$

a. Buckling Stability

$$\sigma_{max} \le \sigma_{cra}$$

Where:

$$\sigma_{cra} = \frac{G_e S_1 S_2}{f_{cr}} = \frac{0.8*12.5*3.34}{2.5} = 13.36\ Mpa > \sigma_{max} = 5.31\ \ (OKAY)$$

f_{cr} : Coefficient considering the frequency of load occurring in laminated rubber bearing which shall be set to **2.5.**

$$S_2 = \frac{shorter\ length\ of\ a\ or\ b}{\sum t_e} = \frac{600}{179.2} = 3.34$$

b. Tensile Stress of Internal plate

$\sigma_s \leq \sigma_{sa}$ where: $\sigma_{sa} = 140\ Mpa\ for\ SS400, t \leq 40mm$

$\sigma_s = f_c \cdot \sigma_{max} \cdot \frac{t_e}{t_s} = 2 * \frac{5.31*12}{3.2} = 38.9\ Mpa < 140\ Mpa\ \ (OKAY)$

- Verification of Deformation Performance

c. Shear Strain

Shear strain caused by horizontal displacement by temperature change at normal condition, creeping of concrete and dry shrinkage were verified.

$\gamma_s \leq \gamma_{sa}$

$\gamma_{sa} = 70\%\ at\ Normal\ Condition\ (Table\ 5.7.2)$

$$\gamma_s = \frac{\Delta L_1}{\sum t_e} = \frac{7}{179.2} = 4\% < 70\%\ \ (OKAY)$$

$\Delta L_1 = 7mm$ Design deformation of laminated rubber bearing occurring at normal condition.

d. Rotational Displacement

The rotational displacement caused by the girder deflection by live load were verified.

$$u_r \leq \frac{u_c}{f_v} = \frac{7.5}{1.3} = 5.77 > 2.83\ (OKAY)$$

$u_r = \frac{a \cdot \sin\theta + b \cdot \cos\theta}{2} \cdot \sum \alpha_e$ For Rectangular Section

$\sum \alpha_e$ = 0.00872 rad. (See Deformation results)

$\theta = 0.0872\ rad.$ (5 Degrees Bevel)

$u_r = \frac{a \cdot \sin\theta + b \cdot \cos\theta}{2} \cdot \sum \alpha_e = \frac{[600 \sin(.0872) + 600 \cos(.0872)]}{2} * .00872 = 2.83\text{mm}$

e. Fatigue Durability

The total of localized shear strain caused by maximum vertical reaction, horizontal traveling amount and rotation were verified.

$\gamma_t \leq \gamma_{ta}$

$\gamma_t = \gamma_c + \gamma_s + \gamma_r = .68 + 0.91 + .039 = 163\% < 433\%\ (OKAY)$

$$\gamma_c = 8.5 \cdot S_1 \cdot \frac{R_{max}}{EA_{cn}} = 8.5 * 12.5 * 1912.63 * \frac{1000}{824 * 360000} = 0.68$$

γ_s=7/179.2 =.039

$\gamma_r = 2(1 + a/b)^2 \cdot S_1^2 \cdot \alpha_e = 2 * (1+1)^2 * 12.5^2 * .00872/12 = 0.91$

α_e Rotational angle per rubber layer

$$E = (3 + 2/3 \cdot \pi^2 \cdot S_1^2)G_e = \left(3 + \frac{2}{3} * \pi^2 * 12.5^2\right)0.8 = 824\ Mpa$$

$$\gamma_{ta} = \frac{\gamma_u}{f_a} = \frac{650}{1.5} = 433.33\%$$

Where:

f_a Safety coefficient of localized shear strain at normal condition, set to which shall be 1.5.

(2) At the Condition of Earthquake

1. Verification of the Vertical Force Support Function

a. Buckling Stability

Buckling stability against downward force during earthquake were verified by Equation below.

$\sigma_{ce} = \frac{R_L}{A_{ce}} \leq \sigma_{cra}$ R_L = Downward Seismic force

Where:

$$\sigma_{cra} = \frac{G_e \cdot S_1 \cdot S_2}{f_{cr}} = \frac{0.8 * 12.5 * 3.34}{1.5} = 22.27\ Mpa$$

$$\sigma_{ce} = \frac{R_L}{A_{ce}} = 2335 * \frac{1000}{360000} = 6.49\ Mpa < 22.27\ Mpa\ \ (OKAY)$$

f_{cr}: Coefficient considering the frequency of the load occurring in laminated rubber bearing, which shall be set to 1.5.

b. Tensile Stress Intensity

Tensile stress intensity caused by the upward force at the condition of earthquake were verified by Equation below.

$\sigma_{te} = \frac{R_U}{A_{te}} = 594 * \frac{1000}{360000} = 1.65\ MPa > \sigma_{ta} = 1.6Mpa$ Say almost Okay, but section not fully adequate. There are two ways to add the capacity 1. By changing rubber type to G10, the allowable now become 2.0 Mpa and 2. By adding dimension, e.q instead of 600 x 600 mm, say 650 x 650 mm considering 12.5 mm Cover, the effective dimension becomes 625 x 625 mm. The Effective Area, Ae now becomes 390,625 mm^2.

In this example, the increase of section has been chosen, and since the dimension was change, Re-analysis is required by changing the input bearing parameters to the new one. This is the second iteration, using the same procedure in the previous analysis and design.

$\sigma_{ta} = 1.6Mpa\ (Table\ 5.7.2)$

- Verification of Deformation Performance in Horizontal Direction
  For the horizontal displacement at the condition of earthquake, the shear strain was verified.

$\gamma_{se} \leq \gamma_{ea} = 200\%$

$\gamma_{se} = \frac{u_B}{\sum t_e} = \frac{162}{179.2} = .904\ or\ 90.4\% < 200\%\ (OKAY)$

$\gamma_{ea} = \frac{u_a}{\sum t_e}$

$u_B = 162\ mm$ Design displacement of HDR at time of Earthquake

$u_a = 358.5$ Allowable displacement of HDR at time of EQ.

9.5.2 Design of Abutment Bearing

(1) At Normal Condition

a. Maximum Compressive stress

$$\sigma_{max} = \frac{R_{max}}{A_{cn}} \leq \sigma_{maxa}$$

$$Shape\ factor, S1 = \frac{160{,}000}{2(400+400)*12} = 8.33 < 12$$

Therefore: $\sigma_{maxa} = 8.33\ Mpa\ (Table\ 5.7.2)$

$$\sigma_{max} = \frac{R_{max}}{A_{cn}} = \frac{Rll + Rdl}{160000} = \frac{(730.875 + 225.44)kN * 1000N}{160000} = 5.97\ Mpa \leq \sigma_{maxa}(OKAY)$$

b. Buckling Stability

$\sigma_{max} \leq \sigma_{cra}$

$where$:

$\sigma_{cra} = \frac{G_e S_1 S_2}{f_{cr}} = \frac{0.8*8.33*2.23}{2.5} = 5.94\ Mpa > \sigma_{max} = 5.31$ *(OKAY)*

$f_{cr} =$: Coefficient considering the frequency of load occurring in laminated rubber bearing which shall be set to **2.5.**

$$S_2 = \frac{shorter\ length\ of\ a\ or\ b}{\sum t_e} = \frac{400}{179.2} = 2.23$$

c. Tensile Stress of Internal plate

$\sigma_s \leq \sigma_{sa}$ where: $\sigma_{sa} = 140\ Mpa\ for\ SS400, t \leq 40mm$

$\sigma_s = f_c \cdot \sigma_{max} \cdot \frac{t_e}{t_s} = 2 * \frac{5.97*12}{3.2} = 44.77\ Mpa < 140\ Mpa\ \ (OKAY)$

- Verification of Deformation Performance

a. Shear Strain

Shear strain caused by horizontal displacement by temperature change at normal condition, creeping of concrete and dry shrinkage were verified.

$\gamma_s \leq \gamma_{sa}$

$\gamma_{sa} = 70\%\ at\ Normal\ Condition\ (RBSIDG\ 5.7.2)$

$$\gamma_s = \frac{\Delta L_1}{\sum t_e} = \frac{7.5}{179.2} = 4\% < 70\%\ \ (OKAY)$$

$\Delta L_1 = 7mm$ Design deformation of laminated rubber bearing occurring at normal condition.

b. Rotational Displacement

The rotational displacement caused by the girder deflection by living load were verified.

$$u_r \leq \frac{u_c}{f_v} = \frac{7.5}{1.3} = 5.77 > 2.83\ (OKAY)$$

$u_r = \frac{a\ \cdot\ \sin\theta + b\ \cdot\ \cos\theta}{2}\ \cdot \sum \alpha_e$ For Rectangular Section

$\sum \alpha_e$ = 0.00872 rad. (See Deformation results)

$\theta = 0.0872\ rad.$ (5 Degrees Bevel)

$u_r = \frac{a\ \cdot\ \sin\theta + b\ \cdot\ \cos\theta}{2}\ \cdot \sum \alpha_e = \frac{[600\sin(.0872) + 600\cos(.0872)]}{2} * .00872 = 2.83$mm

c. Fatigue Durability

The total of localized shear strain caused by maximum vertical reaction, horizontal traveling amount and rotation were verified.

$\gamma_t \leq \gamma_{ta}$

$\gamma_t = \gamma_c + \gamma_s + \gamma_r = .4 + 1.15 + .039 = 159\% < 433\%\ (OKAY)$

$$\gamma_c = 8.5\ \cdot S_1\ \cdot \frac{R_{max}}{EA_{cn}} = 8.5 * 8.33 * 956 * \frac{1000}{367 * 160000} = 1.15$$

γ_s=7/179.2 =.039

$\gamma_r = 2(1 + a/b)^2\ \cdot S_1^2\ \cdot \alpha_e = 2 * (1+1)^2 * 8.33^2 * .00872/12 = 0.40$

α_e Rotational angle per rubber layer

$$E = (3 + 2/3\ \cdot \pi^2\ \cdot S_1^2)G_e = \left(3 + \frac{2}{3} * \pi^2 * 8.33^2\right)0.8 = 367\ Mpa$$

$$\gamma_{ta} = \frac{\gamma_u}{f_a} = \frac{650}{1.5} = 433.33\%$$

Where:

f_a Safety coefficient of localized shear strain at normal condition, set to which shall be 1.5.

(2) At the Condition of Earthquake

1. Verification of the Vertical Force Support Function

a. Buckling Stability

Buckling stability against downward force during earthquake were verified using Equation below.

$\sigma_{ce} = \frac{R_L}{A_{ce}} \leq \sigma_{cra}$ R_L = Downward Seismic force

Where:

$$\sigma_{cra} = \frac{G_e \cdot S_1 \cdot S_2}{f_{cr}} = \frac{0.8 * 8.33 * 2.23}{1.5} = 9.91\ Mpa$$

$$\sigma_{ce} = \frac{R_L}{A_{ce}} = 1177 * \frac{1000}{160000} = 7.35\ Mpa < 9.91\ Mpa\ \ (OKAY)$$

f_{cr}: Coefficient considering the frequency of the load occurring in laminated rubber bearing, which shall be set to 1.5.

b. Tensile Stress Intensity

Tensile stress intensity caused by the upward force at the condition of earthquake were verified by Equation below.

$\sigma_{te} = \frac{R_U}{A_{te}} = 295 * \frac{1000}{160000} = 1.84\ MPa > \sigma_{ta} = 1.6 Mpa$ NOT Okay,

Bearing Section not adequate. There are two ways to add the capacity 1. By changing rubber type to G10, to be the allowable become 2.0 Mpa and 2. By adding dimension, e.q instead of 400 x 400 mm, Say 450 x 450 mm considering 12.5 mm Cover, the effective dimension becomes 425 x 425 mm. The Effective Area, Ae now becomes 180,625 mm^2.
In this example, the increased of dimension was choose. Since the dimension was change, Re-analysis is required by changing the input bearing parameters to the new one. This is the second iteration, using the same procedure in the previous analysis and design.
$\sigma_{ta} = 1.6 Mpa\ (Table\ 5.7.2)$

- Verification of Deformation Performance in Horizontal Direction

For the horizontal displacement at the condition of earthquake, the shear strain were verified.

$$\gamma_{se} \leq \gamma_{ea} = 200\%$$

$$\gamma_{se} = \frac{u_B}{\sum t_e} = \frac{187.5}{179.2} = 1.05.or\ 105\% < 200\%\ (OKAY)$$

$$\gamma_{ea} = \frac{u_a}{\sum t_e}$$

$u_B = 162\ mm$ Design displacement of HDR at time of Earthquake

$u_a = 358.5$ Allowable displacement of HDR at time of EQ.

9.6 Verification of Bridge Response and Hysteresis of HDR Bearing

9.6.1 Hysteresis Curve of HDR

Following **Figure 9.6-1** shows one of the hysteresis curve of high damp rubber bearing at Abutment in the longitudinal direction due to input earthquake EQx.

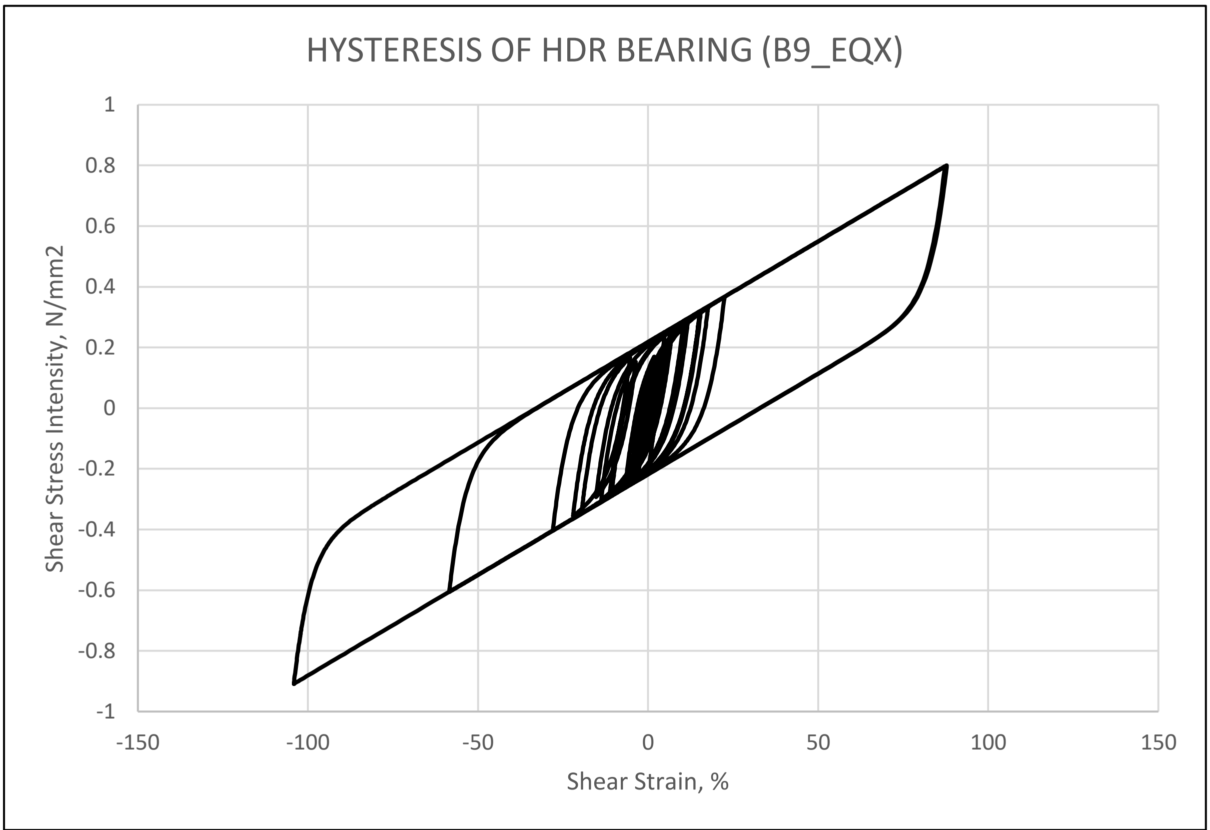


Figure 9.6-1 Hysteresis Curve of HDR Bearing (B9)

In Section 8.1 C8.1 of this guideline also shows the shear stress-shear strain hysteresis of HDR bearing obtained from the actual laboratory testing as shown in Figure C8.1.2. In this section also explained that for HDR bearing, the shear stress intensity increases due to hardening effect when the shear strain exceeds 200%. So, it can be observed that since the shear strain in the example above did not exceed 200%, there is no increase in shear stress due to hardening effect.

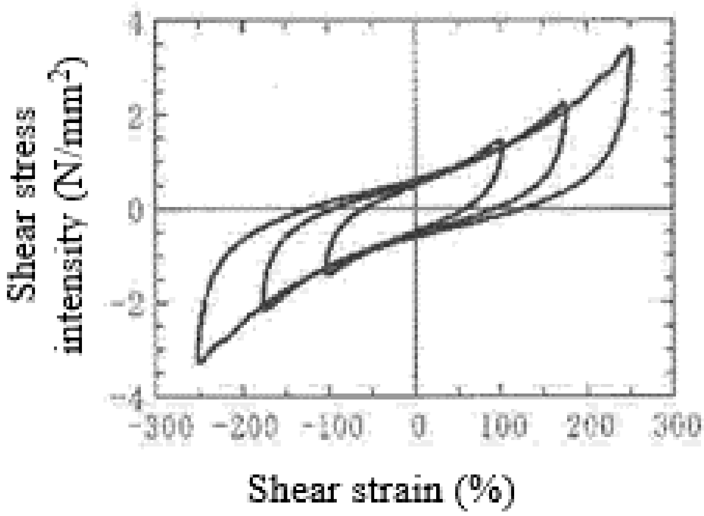


Figure C8.1.2 Hysteresis Curve of High Damping Rubber (HDR-S) Bearing Based on Actual Load Test

9.6.2 Displacement History of superstructure and Pier

Time history response of displacement of both superstructure and pier were also verified as shown in the following Figures.

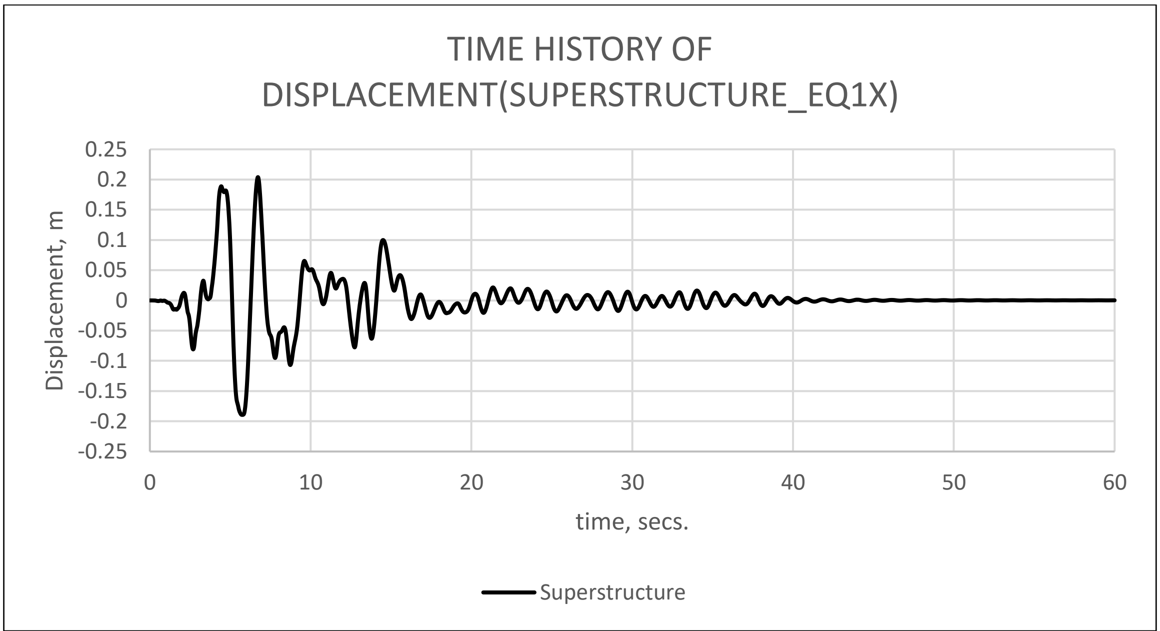


Figure 9.6-2 Time History for Superstructure Displacement

Notice the sudden increase of displacement and also the effect of damping. It is also recommended to provide sufficient gap between the edge of the superstructure and the edge of backwall to allow the movement of isolation during large earthquake. The difference between the Earthquake resisting system and Isolated bridge was discussed in Chapter 10 of this manual.

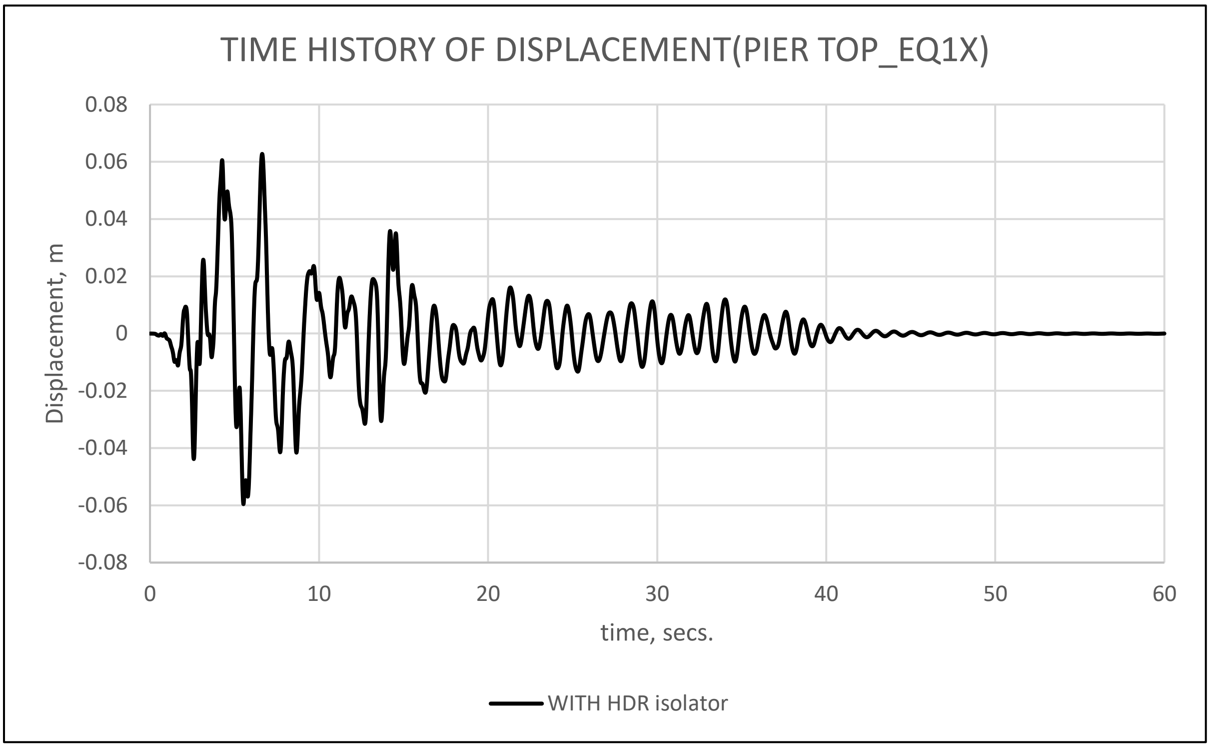


Figure 9.6-3 Sample History of Pier-Top Displacement Due to Eq1x

9.6.3 Verification of Pier Response

The design response of pier was taken as the mean response due to seven (7) set of input ground motions as shown in Table below.

Linear behavior of column during and after earthquake was expected for this isolated bridge example. Secondary plastic behavior which is permitted by this manual at the bottom of the piers is not anticipated in this example. Therefore, moment reduction factor, R set to 1.0 were applied for the elastic response of column. In this case the elastic force due to earthquake were used as a design force as explained in Chapter 9.1.3 of this guideline.

9.6.4 Column Requirements

For the example bridge, following are the requirements for the Pier/ Column. The interaction diagram according the column requirements as shown in **Figure 9.6-5**.

- Required performance Criteria SPL1 (Column is fully elastic)
- Single Round Column
- 2 m (Diameter)
- 76-32 mm Ø Vertical Reinforcement
- 20mm Ø min. pitch Transverse Reinforcement
- Steel Ratio is 2%

Table 9.6-1 Pier Displacement Response

EQX							
PIER NO.:		1	DISPLACEMENT				
Node	Load	DX (m)	DY (m)	DZ (m)	RX (rad)	RY (rad)	RZ (rad)
1522	EQ1X(max)	0.062723	0.049509	0.000063	0.00514	0.006364	0.000214
1522	EQ2X(max)	0.063659	0.047876	0.000044	0.005797	0.006431	0.000114
1522	EQ3X(max)	0.061709	0.04961	0.000045	0.005844	0.006242	0.000228
1522	EQ4X(max)	0.044695	0.060541	0.000034	0.004811	0.004539	0.000148
1522	EQ5X(max)	0.065121	0.041848	0.000058	0.006364	0.006576	0.000162
1522	EQ6X(max)	0.040037	0.056396	0.000037	0.005385	0.004044	0.00016
1522	EQ7X(max)	0.07023	0.05153	0.000061	0.005675	0.007126	0.000248
MEAN:		**0.05831057**	**0.05104429**	**4.8857E-05**	**0.00557371**	**0.00590314**	**0.000182**
EQX							
PIER NO.:		**2**	**DISPLACEMENT**				
Node	**Load**	**DX (m)**	**DY (m)**	**DZ (m)**	**RX (rad)**	**RY (rad)**	**RZ (rad)**
1546	EQ1X(max)	0.062724	0.049514	0.000058	0.00514	0.006364	0.00022
1546	EQ2X(max)	0.063665	0.047882	0.000037	0.005797	0.006432	0.000284
1546	EQ3X(max)	0.061734	0.049618	0.000053	0.005845	0.006245	0.000188
1546	EQ4X(max)	0.044718	0.060545	0.000051	0.004811	0.004541	0.000194
1546	EQ5X(max)	0.065132	0.041853	0.00006	0.006364	0.006577	0.000208
1546	EQ6X(max)	0.040046	0.0564	0.000033	0.005385	0.004045	0.000236
1546	EQ7X(max)	0.070231	0.051534	0.00007	0.005675	0.007126	0.000175
MEAN:		**0.05832143**	**0.05104943**	**5.1714E-05**	**0.00557386**	**0.00590429**	**0.000215**
MAX.		**0.05832143**	**0.05104943**	**5.1714E-05**	**0.00557386**	**0.00590429**	**0.000215**

The effect of P-Delta to the column is not critical due to small displacement.

- Verification of "P-⊿ requirement"					
4*⊿*Pu=	16,278	<	23,412	(=ϕ*Mn)	(OK)
in which:			P-DELTA REQ. SATISFIED		
⊿=	12*Rd*⊿e		⊿: Displacement of the point of contraflexure in the column or pier		
=	0.70	(m)	relative to the point of fixity for the foundation		
Rd=	(1-1/R)*1.25*Ts/T+1/R (if T<1.25*Ts)				
=	1.00				
Rd=	1.0	(if T≧1.25*Ts)			
⊿e=	**0.058**	(m)	⊿e: Displacement calculated from elastic seismic analysis		
T=	**2.10**	>	0.790	(=1.25*Ts) T: Period of fundamental mode of vibration (sec.)	
Ts=	0.632		Ts: Corner period specified in BSDS Article 3.6.2 (=S_{D1}/S_{DS}) (sec.)		
R=	1.0	R: R-factor			
Pu=	5,847	(kN)	Pu: Axial load on column or pier (dead load from the superstructure)		
ϕ=	0.9	ϕ: Flexural resistance factor for column			
Mn=	26,014	(kN*m)	Mn: Nominal flexural strength of column		

Table 9.6-2 Pier Design Forces

EQX							
PIER NO.:		1					
Elem	Load	Part	Shear-y (kN)	Shear-z (kN)	Torsion (kN-m)	Moment-y (kN-m)	Moment-z (kN-m)
3336	EQ1X(max)	J[1558]	458.63	641.86	280.01	22410.31	17902.17
3336	EQ2X(max)	J[1558]	619.22	284.73	150.79	22762.04	17332.95
3336	EQ3X(max)	J[1558]	627.35	603.04	298.28	22256.66	17150.27
3336	EQ4X(max)	J[1558]	226	400.32	195.15	17286.44	20915.49
3336	EQ5X(max)	J[1558]	749.11	685.9	212.64	23209.03	15581.21
3336	EQ6X(max)	J[1558]	312.66	445.16	210.55	16072.82	19041.83
3336	EQ7X(max)	J[1558]	658.23	352.23	324.98	24512.79	18434.06
	MEAN:		521.6	487.605714	238.914286	21215.72714	18051.14
EQX							
PIER NO.:		2					
Elem	Load	Part	Shear-y (kN)	Shear-z (kN)	Torsion (kN-m)	Moment-y (kN-m)	Moment-z (kN-m)
3348	EQ1X(max)	J[1559]	455.99	657.06	283.72	22210.52	17904.9
3348	EQ2X(max)	J[1559]	619.52	300.13	366.66	22563.75	17336.08
3348	EQ3X(max)	J[1559]	627.66	617.28	242.63	22061.14	17154.56
3348	EQ4X(max)	J[1559]	226.49	414.94	250.12	17092.46	20917.7
3348	EQ5X(max)	J[1559]	748.32	701.5	267.81	23010.42	15581.56
3348	EQ6X(max)	J[1559]	312.67	460.82	305.2	15875.92	19046.18
3348	EQ7X(max)	J[1559]	657.14	368.45	225.26	24312.2	18431.96
	MEAN:		521.112857	502.882857	277.342857	21018.05857	18053.27714
MAX FORCES:			**521.6**	**502.882857**	**277.342857**	**21215.72714**	**18053.27714**

9.6.5 Verification of the bearing reaction force to the abutment

In common practice, the abutment is not design to carry the effect seismic lateral force coming from the bridge specially for continuous bridge like in this example. But since the bearing shear force will be transmitted to the abutment during event, the capacity of the abutment to resist such additional lateral force may be checked.

- Max. Bearing Shear forces at the abutment

  Rx = 162 kN (each bearing)

- Total bearing reaction, Rxi = 162*4=648 kN < 7075 kN
- Abutment height = 7.5 m
- Moment Produced at the bottom, M_T = 648*7.5 = 4860 kN-m < 31,386 kN-m

- From Error! Reference source not found. showing that the capacity of the abutment is big enough to carry those additional loads.

- Abutment still adequate.

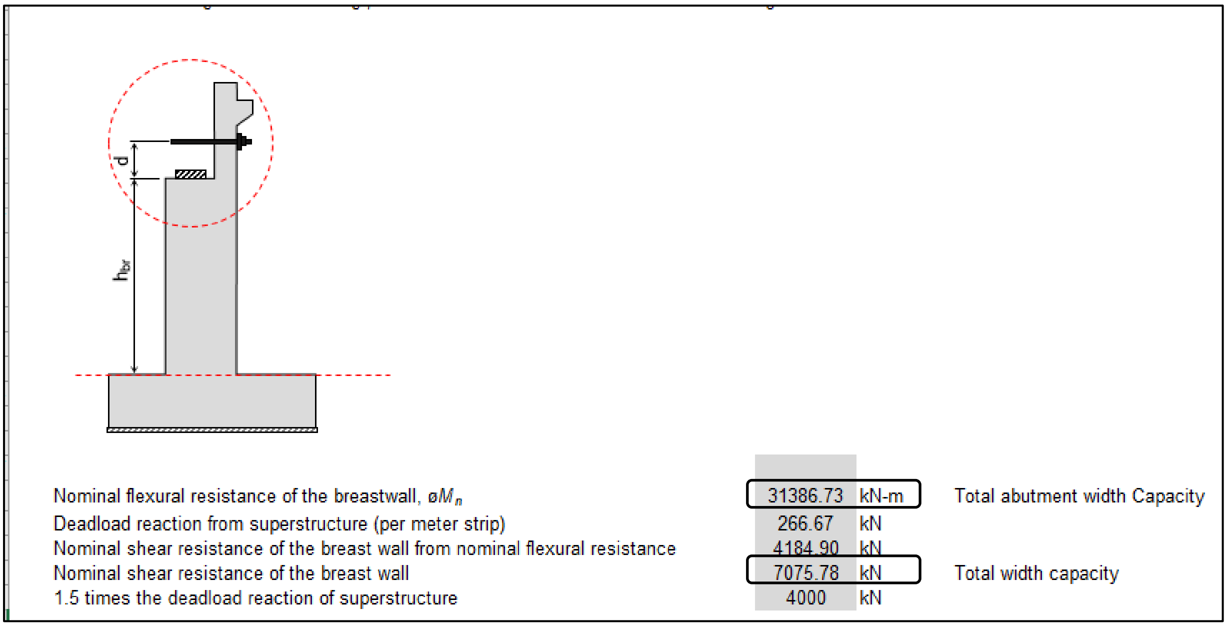


Figure 9.6-4 Abutment Design

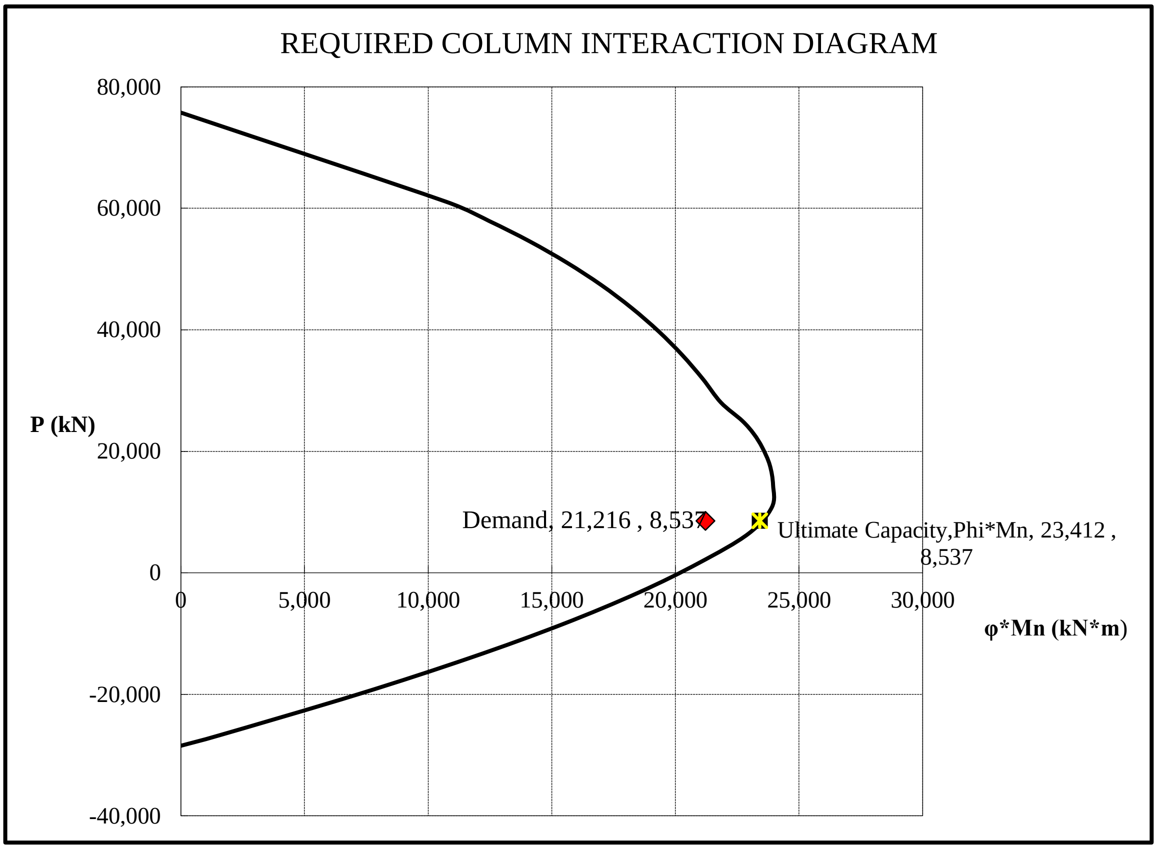


Figure 9.6-5 Interaction Curve of Required Column

9.7 Conclusion

The final design requirements of High Damping Rubber Bearing are summarized below. Checking of all design requirements with the manufacturer is necessary.

Table 9.6-3 Bearing Design Summary

HIGH DAMPING RUBBER BEARING (HDR)	
Bearing Location	**Abutment**
Shape of HDR bearing	Rectangle
Effective Dimension of HDR (a, b)	425 x 425 mm
Overall Dimension (l, w) inc. cover	450 x 450 mm
No. of Rubber Layers	12 pcs.
Thickness of Rubber layers, te	12 mm
Total Rubber Thickness, Σte	179.2
Thickness of internal steel Plate, ts	3.2 mm (SS490)
Shear Modulus of rubber, G	1.2 Mpa
Elongation at Break	550%

Bearing Location	**Pier**
Shape of HDR bearing	Rectangle
Effective Dimension of HDR (a, b)	625 x 625 mm
Overall Dimension (l, w) incl. cover	650 x 650 mm
No. of Rubber Layers	12 pcs.
Thickness of Rubber layers, te	12 mm
Total Rubber Thickness, Σte	179.2
Thickness of internal steel Plate, ts	3.2 mm (SS490)
Shear Modulus of rubber, G	1.2 Mpa
Elongation at Break	550%

Remarks:

The main benefits of Seismic Isolation for bridges, either new or existing are:

1. The addition of flexibility to the system increases the fundamental period, which, for short period structures, will decrease the design forces. However, for long period structures, or ground motions with unusual frequency content, this effect may be negligible, and in extreme cases, design forces may even be higher.
2. Although increase in flexibility can lead to larger displacements, inelastic deformation are confined to the bearing, allowing elastic design of the remaining member of the structure. Bearings are relatively easy to maintain, and if necessary, replace, compared to structural elements.
3. Significant seismic energy may be dissipated in the isolators, by hysteretic damping in its components. This has the effect of further decreasing the shear forces and limiting the maximum displacement demand on the bearing.
4. The shear forces transmitted to the piers are limited by the amount of force that can be transmitted across the bearing, which allows the isolation device to act as a fuse for the structure.

Limitation:

With respect to the 3 benefits of isolation, HDR bearings provide an efficient source of energy dissipation and at moderate displacement levels (like in the example that was presented), satisfy the benefit 1, 2 and 3. However, because the maximum (ultimate) shear force is not well defined (as explained in Clause 8.5 of RBSIDG) the device does not provide an effective fuse across the isolated interface, violating benefit 4. Because of this, at high (ultimate) strain it may not be possible to confine inelastic deformation to the isolator, and piers may experience inelastic demand. The consideration of the interaction between bearing deformation and inelastic pier may be necessary. The more explicit modeling of Pier and more sophisticated analysis may be required in such cases.

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Hwang, J.S, Wu, J.D, Pan, T. C and Yang, G. 2002 "*A mathematical hysteretic model for elastomeric isolation bearings"* Earthquake Engineering and Structural Dynamics, 771-789.

Mata, P, Borosche, R, Barbat, A.H, Oller, S. 2007 "*High Damping Rubber Model for Energy Dissipating devices"* Journal of Earthquake Engineering, London, UK

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CHAPTER 10: COMPARISON BETWEEN SEISMIC RESISTANCE DESIGN AND SEISMIC ISOLATION DESIGN

Chapter 10 Comparison between Seismic Resistance Design and Seismic Isolation Design

10.1 Seismic Resistant Design and Seismic Isolated Design

Over the past few decades, earthquake resistant design of bridge has been largely based on a ductility design concept worldwide. Looking at AASHTO and BSDS specifically, the design philosophy evolved around the intensity of the earthquake: moderate earthquake (Level 1) or Large-scale major earthquake level 2). Seismic performance of bridge according to the design level of earthquake are based on its operational classification.

The acceptable performance of bridges for the traditional force-based seismic design approach is to absorb and dissipate energy by the formulation of plastic hinges in a stable manner to prevent collapse during an earthquake. Specially detailed plastic hinge regions of the supporting ductile columns are capable of absorbing energy through many cycles of the dynamic response of the earthquake. Plastic hinge regions of concentrated damage have been repaired or replaced after earthquakes. The rationale of allowing damage as long as “life safety” is preserved is for economic considerations. The conventional seismic resistant design of structures has been performed under the concept that the structures are designed so that the resistance is greater than the assumed seismic force.

On the other hand, the seismic isolation design is based on the concept of isolating or escaping from seismic force rather than "resisting." Seismic Isolation can be used to avoid having damage to bridge structures and may be achieved at lower initial construction cost. The design of Seismic isolated bridged was explained explicitly in **Chapter 9**.

10.2 Comparison of a Conventional and Seismically Isolated Bridge

The primary objective of applying seismic isolation is to reduce the force that is being generated in the bridge pier and other members by increasing the natural period of the structure and the absorbing energy by means of high damping. In this section, the comparison between the two design techniques was explained.

10.2.1 Bridge Analysis Model

Same configuration of site, also the same analytical and loading model were used in this study. However, the boundary condition for bearings are different. Boundary conditions for the bearing of seismic resistant bridge model are based on the conventional bearing model (free at the abutment and fixed at Pier) as shown in **Figure 10.2-1**

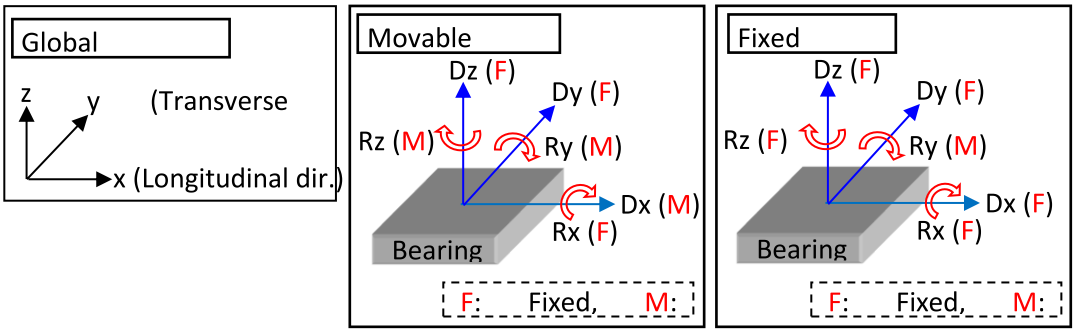


Figure 10.2-1 Degrees of Freedom of Bearing

Detailed modeling procedure for conventional bridge was explained in Chapter 5.0 of the New BSDS Design Guide Manual 1st Edition. Another consideration in the analysis model of earthquake resistance bridge is the consideration of non-linear effects which decrease stiffness. BSDS recommend the use of cracked section for the member which plastic hinging is anticipated equal to one half of the gross moment of inertia.

The dynamic spring for pile foundation are also the same for two modeling. The used of simplified or lumped spring pile foundation model was adopted. Loadings for both static and dynamic are also the same.

10.2.2 Analysis

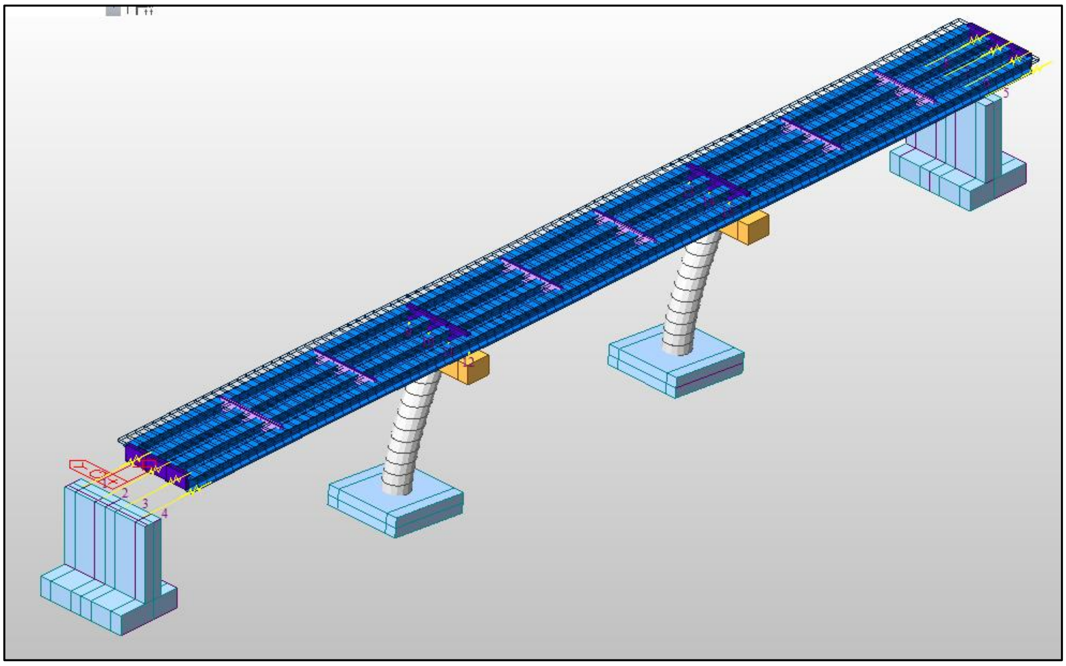
Dynamic analysis using Elastic Time history by modal analysis was performed in the analysis of seismic resistant bridge model as explained also in Chapter 5 of BSDS Design Guide Manual as well as for the analysis of Isolated bridge model in Chapter 9 of this Guideline.

10.3 Comparison of Results

10.3.1 Comparison of Fundamental Periods of Bridge

- Conventional Bridge

1st Mode
Natural Period, Tn = 1.15 secs.



2nd Mode

Natural Period, Tn = 0.80 sec.

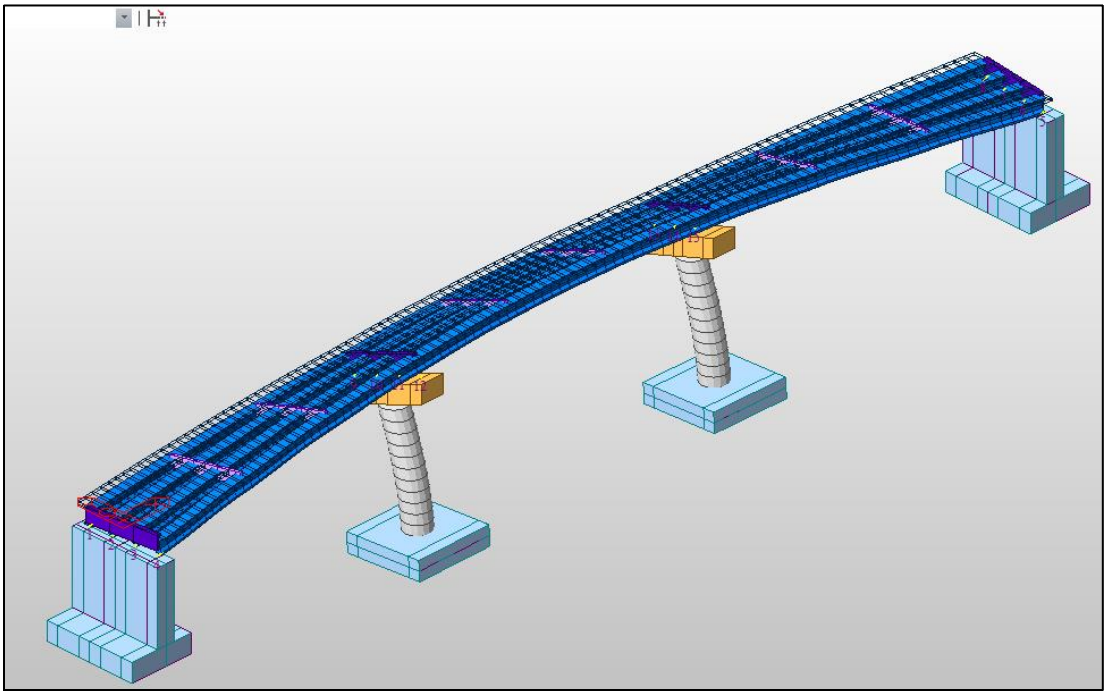


Figure 10.3-1 Fundamental Period of Conventional Model Bridge

The fundamental period of isolated bridge was shown in Chapter 9. Summary of fundamental periods of two different bridge model also shown in **Table 10.3-1**.

Table 10.3-1 Fundamental Period of Bridges Model

Bridge Model	Period, Tn (sec)	Period, Tn (sec)
	1st Mode	2nd Mode
Conventional Bridge	1.15	0.8
Isolated Bridge	2.13	1

Comparing the fundamental period of structure, noticed that the dominant period of isolated bridge was almost double compare to the others. This is due to bearing isolator is flexible enough so that the cycle of bridge excitation was lengthen, that is why the applicability of isolator also limited to stiff structures and for the bridge that was located where the soil is hard enough as explained in BSDS Section 8.1.

10.3.2 Comparison of Superstructure Displacement

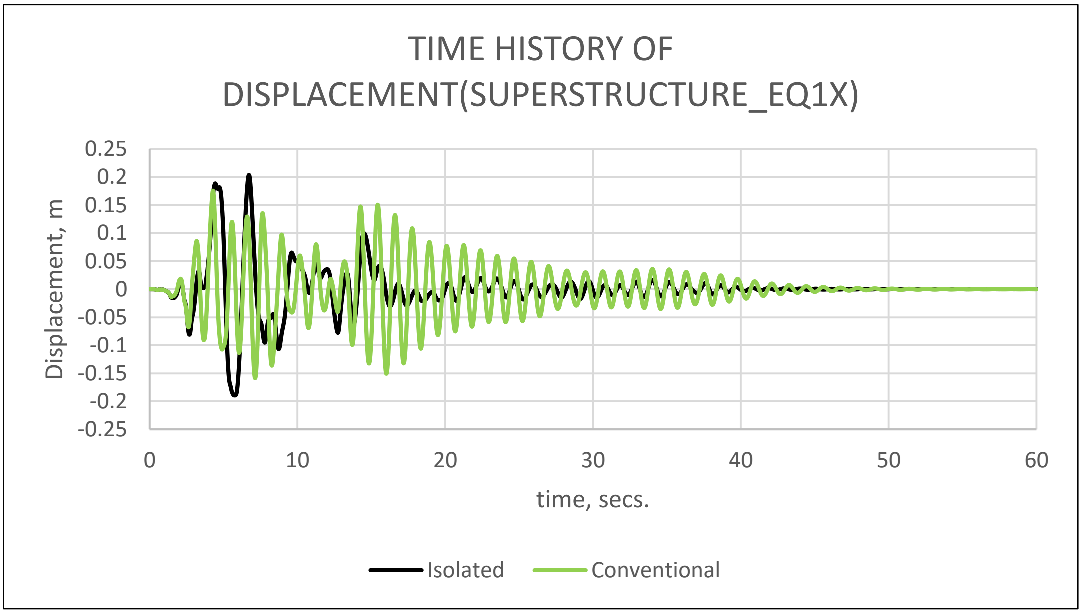


Figure 10.3-2 Displacement History of Superstructure of Two Different Model

The difference of maximum displacement response of superstructure taken as the mean of the maximum response due to seven (7) pairs of input spectrally matched ground motion as shown in **Figure 9.2-2** is small at this case, however, the damping effect due to damping properties of rubber was obvious in black line. Also, the increase in period due to isolator was definitely obvious. The design displacement for superstructure of both models as shown in **Table 10.3-2**.

Table 10.3-2 Design Displacement of Superstructure

Bridge Model	Superstructure	
	Displacement, Dx (mm)	Displacement, Dx (mm)
Conventional Bridge	176	176
Isolated Bridge	188	188

10.3.3 Comparison of Top of Pier Displacement

Unlike the conventional design bridge that the bearing is fixed, the displacement of the top of pier is equal to the displacement of superstructure due to fixity of bearing, in the case of isolated bridge is different as shown in Error! Reference source not found.. The displacement of pier top is small in case of isolated bridge, this is due to the effect of HDR isolator. The isolation physically uncoupled a bridge superstructure from the horizontal components of earthquake ground motion, leading to a substantial reduction in the forces generated by an earthquake. Also, the energy produced by an earthquake was dissipated by the hysteretic property of high damping rubber bearing.

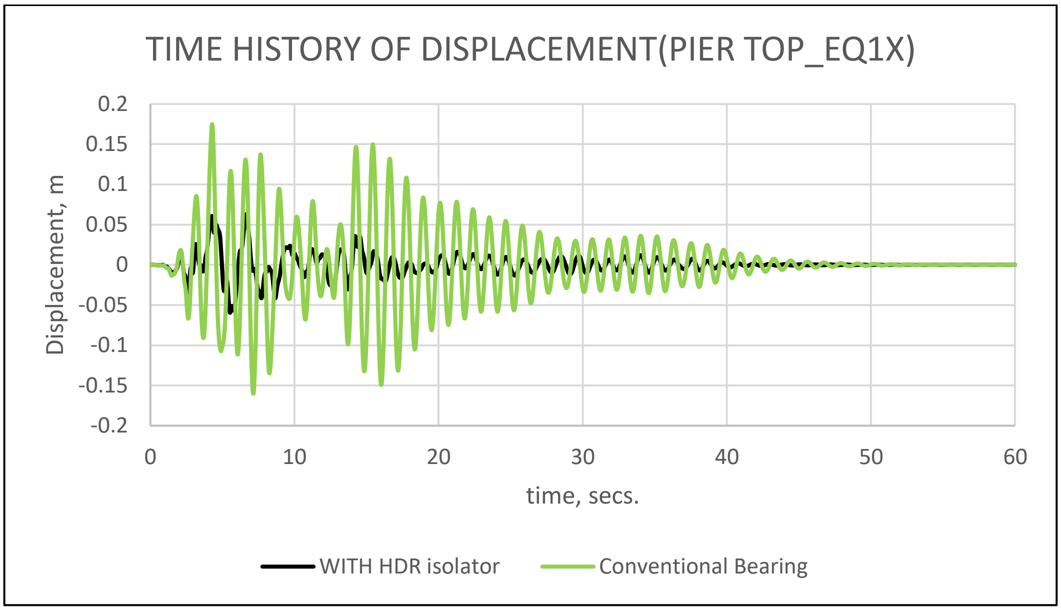


Figure 10.3-3 Displacement History of Pier Top of Two Different Model

The comparison of the design displacement at the top of pier as shown in **Table 10.3-3**. Those value were taken from the mean of maximum response due to seven input ground motion.

Table 10.3-3 Design Displacement at Top of Pier

Top of	Pier 1	Pier 2
Bridge Model	**Displacement, Dx (mm)**	**Displacement, Dx (mm)**
Conventional Bridge	162	162
Isolated Bridge	58	58

10.3.4 Comparison of Forces at Pier

The response of the pier for isolated bridge particularly the forces extracted by the pier at the bottom was illustrated in Chapter 9 of this guideline. Comparison of the difference in the design forces between these two different models that were used for the design of section of column are shown in **Table 10.3-4**.

Notice the difference of the design forces, this is mainly because of isolation as explained in Chapter 9. However, for the design of conventional bridge, ductility factor which may reduce the design elastic force at the base of column where plastic hinging was anticipated was required. The elastic force in **Table 10.3-4** for conventional bridge was reduced by response modification factor or a certain ductility factor according to BSDS Section 3.8 Table 3.8.1-1, and for essential bridge (OC2) the ductility factor to be used is 2.0. Therefore, for the design of column of conventional bridges the forces to be used in this example was divided by this factor and the final results as shown in **Table 10.3-5**.

Table 10.3-4 Comparison of Design Forces at Column Base

PIER NO.:	1	Mean of max. Response due to 7 Pairs of EQ				
Bridge Model		Shear-y (kN)	Shear-z (kN)	Torsion (kN-m)	Moment-y (kN-m)	Moment-z (kN-m)
Conventional		5051.97	6988.17	3932.19	93490.43	62638.72
Seismic Isolated		521.6	487.6	238.91	21215.72	18051.14
PIER NO.:	2	Mean of max. Response due to 7 Pairs of EQ				
Bridge Model		Shear-y (kN)	Shear-z (kN)	Torsion (kN-m)	Moment-y (kN-m)	Moment-z (kN-m)
Conventional		5101.06	6952.14	4268.75	92994.51	63186.79
Seismic Isolated		521.11	502.88	277.34	21018.06	18053.28

Table 10.3-5 Modified Design Forces at Column Base

	Mean of max. Response due to 7 Pairs of EQ				
Bridge Model	Shear-y (kN)	Shear-z (kN)	Torsion (kN-m)	Moment-y (kN-m)	Moment-z (kN-m)
Conventional Pier 1	2525.99	3494.09	1966.09	46745.22	31319.36
Conventional Pier 2	2550.53	3476.07	2134.38	46497.25	31593.39

This modification factor or ductility factor is not necessary for isolated bridge, unless the ductility at pier was considered in the design as explained in Chapter 9 of this guideline.

Using the above design forces for conventional bridge, the required section capacity was illustrated in **Figure 10.3-4**. Followed the column design procedure, then, the required Design for the column section for conventional bridge should be as follows:

- Required Diameter of Column, D = 2700 mm
- Required No. of Vertical Rebars, N = 120 pcs.
- Required Steel ratio, ρ = 1.7%
- Diameter of ties, dt = 20 mmØ (Bundled in two)
- Minimum Pitch, s = 120 mm
- Verification of "P-⊿ requirement"

4*⊿*Pu = 45,466 < 51,727 (=ϕ*Mn) **(OK)**

in which: P-DELTA REQ. SATISFIED

⊿ = 12*Rd*⊿e ⊿: Displacement of the point of contra-flexure in the column or pier.
= 1.94 (m) relative to the point of fixity for the foundation
Rd = (1-1/R) *1.25*Ts/T+1/R (if T<1.25*Ts)
= 1.00

Rd	=	1.0 (if T≧1.25*Ts)	
⊿e	=	0.162 (m)	⊿e: Displacement calculated from elastic seismic analysis
T	=	1.15 > 0.790 (=1.25*Ts)	T: Period of fundamental mode of vibration (sec.)
Ts	=	0.632	Ts: Corner period specified in BSDS Article 3.6.2 (=SD1/SDS) (sec.)
R	=	2.0	R: R-factor
Pd	=	5,847 (kN)	PD: Axial load on column or pier (dead load from the superstructure)
Φ	=	0.9	ϕ: Flexural resistance factor for column
Mn	=	57,475 (kN*m)	Mn: Nominal flexural strength of column

By checking the P-Delta effect it is clear that the design section has satisfied.
The design of section for isolated bridge was already explained in Chapter 9.

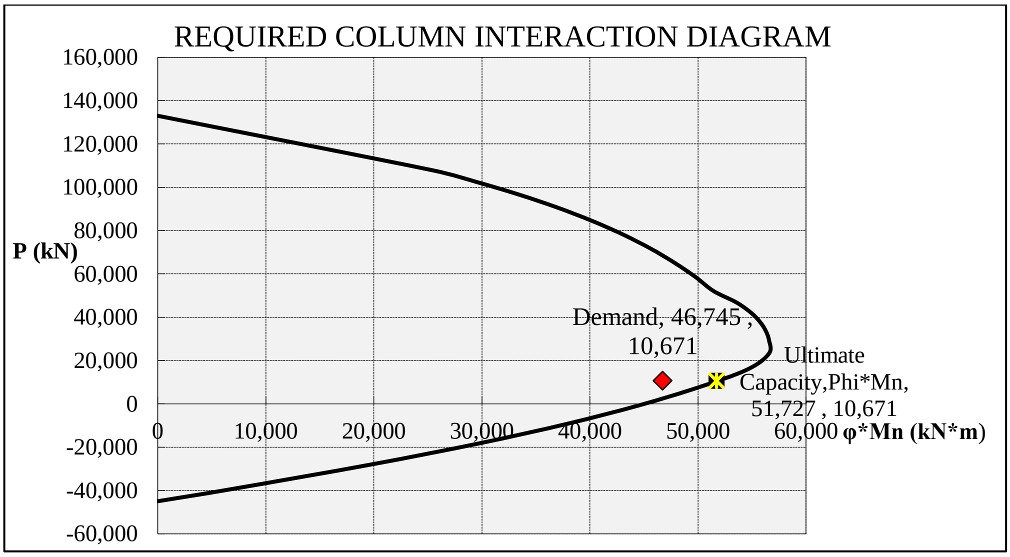


Figure 10.3-4 Interaction Diagram of Required Column Section for Conventional Bridge

10.4 Conclusion

Summary of the results comparing the two bridges model was illustrated in **Table 10.4-1**.

Table 10.4-1 Comparison Table of Conventional and Seismic Isolated Bridge

	PIER BEARING TYPE	
	CONVENTIONAL	HIGH DAMP RUBBER
Shape of Bearing	N/A	RECTANGLE
Dimension of Bearing for Abutment (mm)	N/A	450 X 450
Dimension of Bearing for Piers (mm)	N/A	650 X 650
Thickness of Bearing (mm)	N/A	179.2
Max. Displacement of Superstructure (mm)	176	204
Max. Displacement of Pier top (m)	162	58
Natural Period of Structure, Tn (secs)	1.15	2.13
Moment at Pier Bottom (kN-m)	93,490.00	21,215.00
Shear Force at Pier Bottom (kN)	3495	502
Size of Bridge Pier Required (m)	2.7	2.0
Percentage of Rebar Required (%)	1.7	2

In this table, the difference in response between these two different bridges model was obvious. Another difference was not illustrated but already explained in Chapter 9.

With the same earthquake loading the performance level of these two models are also different. For conventional design bridge, the ductility of bridge pier is expected at a certain location meaning the plastic hinging is anticipated and repairable damages (no collapse) are allowed during earthquake.

In contrary, for the Seismic Isolated Design bridge, because a seismic isolation bearing can absorb the energy, this reduces the forces transmitted to the substructure columns, piers, and foundations. Also, the earthquake energy is absorbed by heat in the isolation bearing that provides protection for the substructure, therefore, by protecting the structure it also assures the elastic response.

The effectivity of seismic isolation for bridges is not only for improving the structural performance but of course another factor is the cost effectivity. By reducing the amount of forces attracted to Pier, the section also reduced including the foundation and the cost also reduced. Another factor is the performance of bridge during and after earthquake. The isolated bridge requires only minimal or no damage after earthquake due to its seismic performance level, however, the conventional bridge allowed structural damage but no collapsed. The higher cost of repair must be expected on that performance level. Cost implication is another consideration of choosing or specifying the performance level of bridge.

References:

AASHTO. 2014, "*Guide Specifications for Seismic Isolation Design".* American Association of State Highway and Transportation Officials: Washington, DC, 2014.
DPWH. 2013 "*Bridge Seismic Design Specification, BSDS"* (1st Edition), Department of Public Works and Highways, Manila.
DPWH. 2018 "*Highway Bridge Seismic Isolation Design Specification"* (1st Edition), Department of Public Works and Highways

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CHAPTER 11: GAP BEARING ADJACENT GIRDERS AND SUBSTRUCTURES

Chapter 11 Gap Bearing Adjacent Girders and Substructure

11.1 Gap between adjacent girder and Substructures

$S_B =$ $u_s + L_A$ (between a superstructure and an abutment , or a superstructure and a truncated section of a pier head)

$c_B\ u_s + L_A$ (between two adjacent girder)

L_A: marginal value. 15mm in General.
u_s: Maximum relative displacement due to Level 1 or Level 2 Earthquake.
C_B: Modification factor depending on the difference of natural periods of adjacent superstructure as shown in **Table 11.1-1**

Table 11.1-1 Joint Gap Width Modification Factor for Natural Period Difference between Adjacent Girders c_B

Ratio of Natural period Difference of adjacent girders $\Delta T/T_1$	c_B
$0 \leqq \Delta T / T_1 < 0.1$	1
$0.1 \leqq \Delta T / T_1 < 0.8$	$\sqrt{2}$
$0.8 \leqq \Delta T / T_1 \leqq 1.0$	1

Notes: $\Delta T = T_1 - T_2$, and T_1 and T_2 represent the natural periods of two adjacent girders, respectively. However, T_1 is assumed equal to or greater than T_2.

The amount of gap between adjacent girder and substructure is classified into two cases.

(1) Ordinary Bridge

This amount is determined as that collision will not occur for level 1 Earthquake Ground Motion, if verification confirms that the collision will not affect the seismic performance of the bridge when subjected to Level 2 Earthquake Ground Motion.

(2) Seismically –Isolated Bridge

The amount of gap is determined to ensure the expected behavior of seismic isolation.
As for this case, calculation example is shown in this Design Guideline 8.2.

CHAPTER 12: EXAMINATION OF LIQUEFACTION

Chapter 12 Examination of Liquefaction

12.1 Liquefaction

12.1.1 Assessment of seismically unstable soil layer

Assessment of seismically unstable soil layer is defined as **Table 12.1-1.**

Table 12.1-1 Assessment of soil layer

Assesment of Soil Layer	Condition of Decision	Calculation Treatment
Extremely Soft layer	For a clayey or silt soil within 3 m from ground surface, having a compressive strength of 20kPa(kN/m2)(0.02N/mm2) or less obtained by unconfined compression test or an in-situ test.	Geotechnical parameters shall be zero. Acts as overburden.
Liquefiable Layer Liquefaction Assessment	For the alluvial sandy layer having all of the following three conditions, liquefaction assessment shall be conducted. 1) Saturated soil layer with depth less than 20m below the ground surface and having ground water level higher than 10m below the ground surface. 2) Soil layer containing a fine content(FC) of 35% or less, or soil layer having plasticity index, PI, less than 15,even if FC is larger than 35%. 3) Soil layer having a mean particle size(D_{50})of less than 10mm and a particle size at 10% passing (D_{10})(on the grading curve is less than 1mm.	Reduce geotechnical parameter for seismic design If Reduction factor D_E =0 and geotechnical parameter is zero, it acts as overburden.

12.1.2 Assessment of Liquefaction

In BSDS, following provisions are described.

For the soil layer requiring liquefaction assessment according to the provisions specified in Item (1) above, the liquefaction resistance factor F_L, shall be calculated by Equation 6.2.3-1. When the result turns out to be less than 1.0, the layer shall be regarded as a soil layer having liquefaction potential.

$$F_L = R / L \quad \text{..........(6.2.3-1)}$$

$$R = c_w R_L \quad \text{..........(6.2.3-2)}$$

$$L = r_d\, k_{hgL}\ \sigma_v / \sigma'_v \quad \text{..........(6.2.3-3)}$$

$$r_d = 1.0 - 0.015x \quad \text{..........(6.2.3-4)}$$

$$k_{hgL} = F_{pga}\, PGA \quad \text{..........(6.2.3-5)}$$

$$\sigma_v = \gamma_{t1} h_w + \gamma_{t2}(x-h_w) \quad \text{..........(6.2.3-6)}$$

$$\sigma'_v = \gamma_{t1} h_w + \gamma'_{t2}(x-h_w) \quad \text{..........(6.2.3-7)}$$

$$c_w = \begin{cases} 1.0 & (R_L \leq 0.1) \\ 3.3R_L + 0.67 & (0.1 < R_L \leq 0.4) \\ 2.0 & (0.4 < R_L) \end{cases} \quad \text{.......(6.2.3-8)}$$

where:

- F_L : Liquefaction resistance factor.
- R : Dynamic shear strength ratio.
- L : Seismic shear stress ratio.
- c_w : Modification factor on earthquake ground motion.
- R_L : Cyclic triaxial shear stress ratio to be obtained from Equation 6.2.3-9 in Item (3) below.
- r_d : Reduction factor of seismic shear stress ratio, in terms of depth.
- k_{hgL} : Design horizontal seismic coefficient at the ground surface for Level 2 EGM.
- F_{pga} : Site coefficient for peak ground acceleration specified in Article 3.5.3.
- PGA : Peak ground acceleration coefficient on rock, as given in Article 3.6.
- σ_v : Total overburden pressure, (kN/m^2).
- σ'_v : Effective overburden pressure, (kN/m^2).
- x : Depth from the ground surface, (m).
- γ_{t1} : Unit weight of soil above the ground water level, (kN/m^3).
- γ_{t2} : Unit weight of soil below the ground water level, (kN/m^3).
- γ'_{t2} : Effective unit weight of soil below the ground water level, (kN/m^3).
- h_w : Depth of the ground water level, (m).

Cyclic triaxial shear stress ratio

Cyclic triaxial shear stress ratio R_L shall be calculated by Equation 6.2.3-9.

$$R_L = \begin{cases} 0.0882\sqrt{N_a/1.7} \\ 0.0882\sqrt{N_a/1.7} + 1.6x10^{-6} \bullet (N_a - 14)^{4.5} \end{cases} \quad (N_a < 14) \quad \text{........................ (6.2.3-9)}$$

where:

(For Sandy Soil)

$$N_a = c_1 N_1 + c_2 \quad \cdots\cdots \quad (6.2.3\text{-}10)$$

$$N_1 = 170N / (\sigma'_v + 70) \quad \cdots\cdots \quad (6.2.3\text{-}11)$$

$$c_1 = \begin{cases} 1.0 & (0\% \le FC < 10\%) \\ (FC + 40) / 50 & (10\% \le FC < 60\%) \\ FC / 20 - 1 & (60\% \le FC) \end{cases} \quad \cdots\cdots \quad (6.2.3\text{-}12)$$

$$c_2 = \begin{cases} 0 & (0\% \le FC < 10\%) \\ (FC - 10) / 18 & (10\% \le FC) \end{cases} \quad \cdots\cdots \quad (6.2.3\text{-}13)$$

(For Gravelly Soil)

$$N_a = \{1 - 0.36 \log_{10}(D_{50}/2)\}\ N_1 \quad \cdots\cdots \quad (6.2.3\text{-}14)$$

R_L : Cyclic triaxial shear stress ratio.

N : N-value obtained from the standard penetration test.

N_1 : Equivalent N value corresponding to effective overburden pressure of 100 kN/m^2.

N_a : Modified N value taking into account the effects of grain size.

c_1, c_2 : Modification factors of N value on fine content.

FC : Fine content, (%) (percentage by mass of fine soil passing through the 75μm mesh).

D_{50} : Mean grain diameter, (mm).

12.1.3 Calculation Example

(1) Examination of liquefaction potential

Next Tables are examples of Lambingan Bridge case.

Table 12.1-2 Assessment of Liquefaction Potential

BH-4 FOR ABUTMENT A1								
Summary Assessment of Liquefaction Potential								
GL	Soil Layers	N-Value	Ground Water Level	FC	PI	D50	D10	Assessment
m	-	by SPT	m	%	-	mm	mm	-
-1.4	Sandy	10	1.03	0	0	0	0.00	o
-2.4	Sandy	13	1.03	0	0	0	0.00	o
-3.4	Sandy	20	1.03	10	0	0.55	0.08	o
-4.9	Sandy	4	1.03	0	0	0	0.00	o
-6.4	Sandy	17	1.03	7	0	0.52	0.13	o
-7.9	Sandy	2	1.03	52	0	0.055	0.00	o
-9.4	Clayey	4	1.03	90	44	0.1	0.00	
-10.9	Clayey	5	1.03	62	41	0.1	0.00	
-12.4	Clayey	3	1.03	86	45	0.1	0.00	
-13.9	Clayey	3	1.03	85	39	0.1	0.00	
-15.4	Clayey	5	1.03	88	47	0.1	0.00	
-16.9	Clayey	13	1.03	77	74	0.1	0.00	
-18.4	Sandy	25	1.03	16	0	1	0.10	o
-19.9	Sandy	50	1.03	100	50	100	10.00	
-21.4	Sandy	50	1.03	100	50	100	10.00	
-22.9	Sandy	50	1.03	100	50	100	10.00	
-24.4	Sandy	50	1.03	100	50	100	10.00	
-25.9	Sandy	50	1.03	100	50	100	10.00	
-27.4	Sandy	50	1.03	100	50	100	10.00	

BH -1 FOR ABUTMENT A2								
Summary Assessment of Liquefaction Potential								
GL	Soil Layers	N-Value	Ground Water Level	FC	PI	D50	D10	Assessment
m	-	by SPT	m	%	-	mm	mm	-
1.41	Sandy	2	1.03	25	0	0.52	0.00	o
1.03	Sandy	2	1.03	25	0	0.52	0.00	o
0.41	Sandy	2	1.03	0	0	0	0.00	o

-0.59	Clayey	2	1.03	62	10	0.1	0.00	o
-3.59	Clayey	5	1.03	12	0	0.8	0.05	o
-5.09	Clayey	13	1.03	52	0	0.05	0.00	o
-6.59	Clayey	24	1.03	75	0	0.1	0.00	o
-8.09	Clayey	3	1.03	70	20	0.1	0.00	
-11.09	Clayey	4	1.03	83	29	0.1	0.00	
-12.59	Clayey	3	1.03	78	39	0.1	0.00	
-15.59	Sandy	50	1.03	16	0	0.8	0.04	o
-15.74	Sandy	50	1.03	100	50	100	10.00	
-17.09	Sandy	50	1.03	100	50	100	10.00	
-18.59	Sandy	50	1.03	100	50	100	10.00	
-20.09	Sandy	50	1.03	100	50	100	10.00	
-21.59	Sandy	50	1.03	100	50	100	10.00	

Painted layers have a potential of liquefaction.

(2) Calculation of Liquefaction Resistance Factor F_L

Table 12.1-3 Calculation of FL

Calculation for FL (A1)								**Reduction Factor DE**		
Depth	N1	c1	c2	Na	R	L	FL	R Ave.	FL Ave.	DE
1.00	22.911	1.000	0.000	22.911	0.650	1.740	0.374			
2.00	28.189	1.000	0.000	28.189	1.207	1.714	0.704	3.934	2.324	1.00
3.00	41.162	1.000	0.000	41.162	9.946	1.687	5.895			
4.50	7.281	1.000	0.000	7.281	0.232	1.426	0.163			
6.00	27.735	1.000	0.000	27.735	1.135	1.312	0.865	**0.537**	**0.408**	**0.67**
7.50	2.957	1.840	2.333	7.773	0.244	1.239	0.197			
9.00	5.613	3.500	4.444	24.090						
10.50	6.677	2.100	2.889	16.911						
12.00	3.822	3.300	4.222	16.834						
13.50	3.653	3.250	4.167	16.040						
15.00	5.832	3.400	4.333	24.162						
16.50	14.549	2.850	3.722	45.187						
18.00	25.883	1.120	0.333	29.322	1.423	1.111	1.280	1.423	1.280	1.00
19.50	48.159	4.000	5.000	18.703						

21.00	44.808	4.000	5.000	17.402						
22.50	41.893	4.000	5.000	16.270						
24.00	39.334	4.000	5.000	15.276						
25.50	37.069	4.000	5.000	14.397						
27.00	35.052	4.000	5.000	13.613						

Calculation for FL(A2)								**Reduction Factor DE**		
Depth	N1	c1	c2	Na	R	L	FL	R Ave.	FL Ave.	DE
1.00	4.048	1.300	0.833	6.095	0.204	0.522	0.391	**0.181**	**0.329**	**0.00**
1.38	3.807	1.300	0.833	5.782	0.196	0.519	0.378			
2.00	3.701	1.000	0.000	3.701	0.143	0.657	0.218			
3.00	3.543	2.100	2.889	10.329	0.302	0.816	0.370	**0.302**	**0.370**	**0.67**
6.00	7.851	1.040	0.111	8.276	0.255	1.053	0.243	**0.255**	**0.243**	**0.00**
7.50	19.316	1.840	2.333	37.875	5.913	1.106	5.348	683.310	601.818	1.00
9.00	33.842	2.750	3.611	96.675	1360.708	1.136	1198.287			
10.50	4.025	2.500	3.333	13.396						
13.50	4.892	3.150	4.056	19.464						
15.00	3.513	2.900	3.778	13.966						
18.00	50.971	1.120	0.333	58.273	82.836	1.038	79.791	41.418	79.791	1.00
18.15	50.625	4.000	5.000	19.661						
19.50	47.709	4.000	5.000	18.529						
21.00	44.840	4.000	5.000	17.415						
22.50	42.297	4.000	5.000	16.427						
24.00	40.026	4.000	5.000	15.545						

(3) Reduction of Geotechnical Parameters

Reduction factor is determined by shear strength ratio R, Resistance Factor F_L and depth x.

Table 12.1-4 Reduction Factor D_E for Soil Parameters

Range of F_L	Depth from Ground Surface x (m)	Dynamic shear strength ratio R	
		$R \leqq 0.3$	$0.3 < R$
$F_L \leqq 1/3$	$0 \leqq x \leqq 10$	0	1/6
	$10 < x \leqq 20$	1/3	1/3
$1/3 < F_L \leqq 2/3$	$0 \leqq x \leqq 10$	1/3	2/3
	$10 < x \leqq 20$	2/3	2/3
$2/3 < F_L \leqq 1$	$0 \leqq x \leqq 10$	2/3	1
	$10 < x \leqq 20$	1	1

D_E is applied for all Geo Technical constants.

12.2 Lateral spreading

Example bridge and target pier are shown in Error! Reference source not found.

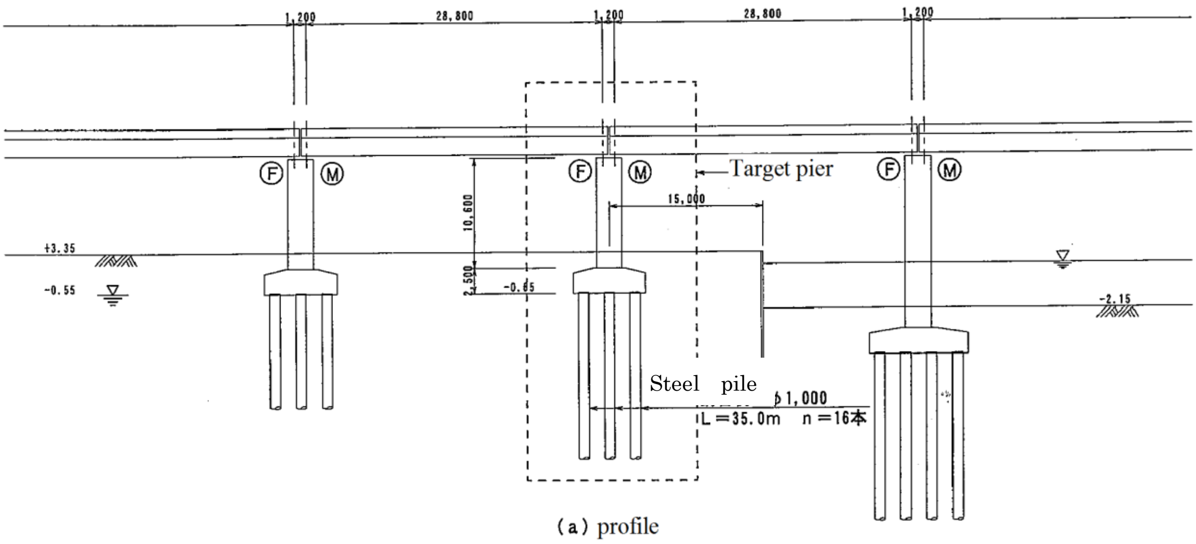


Figure 12.2-1 Example Bridge

Borehole Log

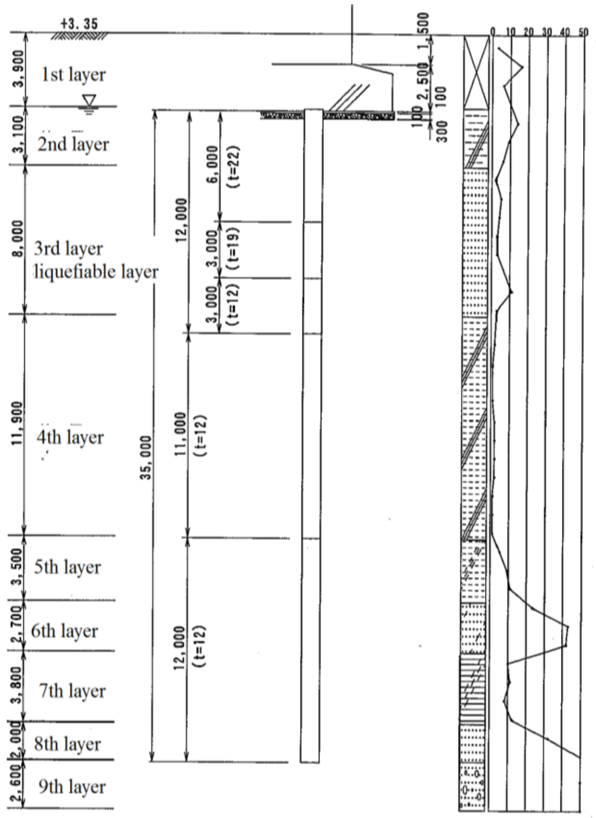


Figure 12.2-2 Borehole Log

Table 12.2-1 Geological Constants

Soil layer number	Thickness (m)	N value	γ t (KN/m3)	Cohesion c (KN/m²)	Internal friction angle(φ°)	Modulus of deformation (KN/m²)
1st layer	3.90	7	16.2	0.0	25	19600
2nd layer	3.10	5	16.2	0.0	25	14000
3rd layer	8.00	4	17.2	0.0	20	12000
4th layer	11.90	1	16.7	49	0	20000
5th layer	3.50	4	18.6	6.0	0	40000
6th layer	2.70	40	17.2	0.0	40	112000
7th layer	3.80	10	19.1	98	0	28000
8th layer	2.00	40	19.1	0.0	40	112000
9th layer	2.60	50	19.6	0.0	40	140000

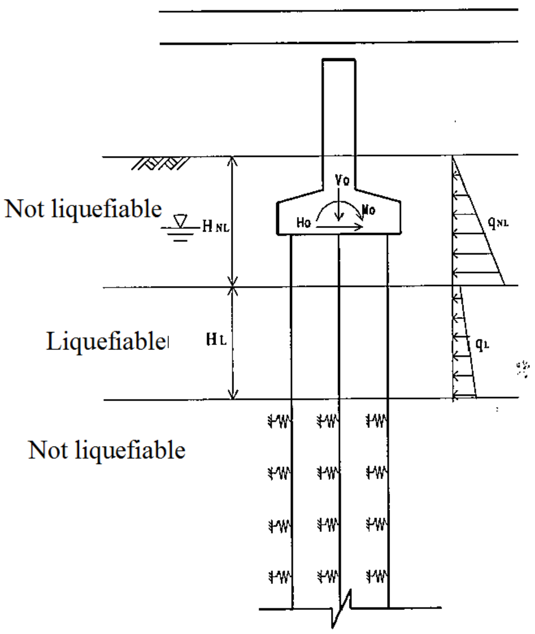


Figure 12.2-3 Analysis Model

(1) Calculation of Lateral Spreading Force

Lateral Spreading Force is calculated with 8.3.2 JRA Vol 5, 2012.

$$q_{NL} = C_s C_{NL} K_p \gamma_{NL} x \qquad (0 \leqq x \leqq H_{NL})$$

$$q_L = C_s C_L(\gamma_{NL} H_{NL} + \gamma_L(x - H_{NL})) \qquad (H_{NL} \leqq x \leqq H_{NL} + H_L)$$

$$q_{NL} = C_s C_{NL} K_p \gamma_{NL} x$$

$$= 1.0 \times 1.0 \times 2.0 \times 16.2 \times 7.0$$

$$= 226.8 \text{kN/m}^2$$

$$q_{L1} = C_s C_L(\gamma_{NL} H_{NL} + \gamma_L(x - H_{NL}))$$

$$= 1.0 \times 0.3 \times 16.2 \times 7.0 = 34.0 \text{kN/m}^2 \qquad (x = H_{NL})$$

$$q_{L2} = C_s C_L(\gamma_{NL} H_{NL} + \gamma_L(x - H_{NL})) \qquad (x = H_{NL} + H_L)$$

$$= 1.0 \times 0.3 \times [16.2 \times +17.2 + 17.2 \times (15.0 - 7.0)$$

$$= 75.3 \text{ kN/m}^2$$

Loading width

Pier 14.0m

Footing 14.5m

Pile foundation 13.5m

Table 12.2-2 Calculation of Lateral Spreading Force Applying to Pier

Layer name	Thickness (m)	Unit load(KN/m^2)	Resultant force(KN/m)	Structure name	Load width	Lateral spreading force(KN)
1st layer	1.5	0.00	37	Column	14.0	518
		49.5				
	2.5	49.5	227	Footing	14.5	3292
		132.0				
2nd layer	3.1	132.0	564	Pile	13.5	7614
		232.1				
3rd layer	8.0	35.0	448			6048
		77.0				

Total 17472(KN)

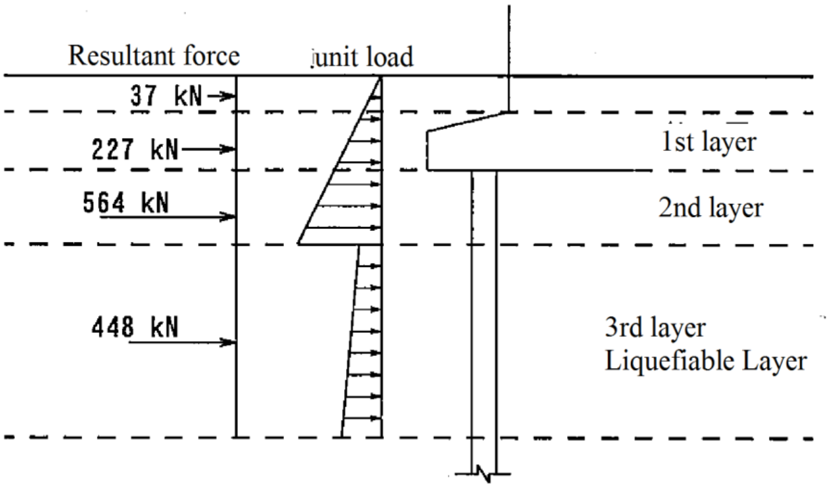


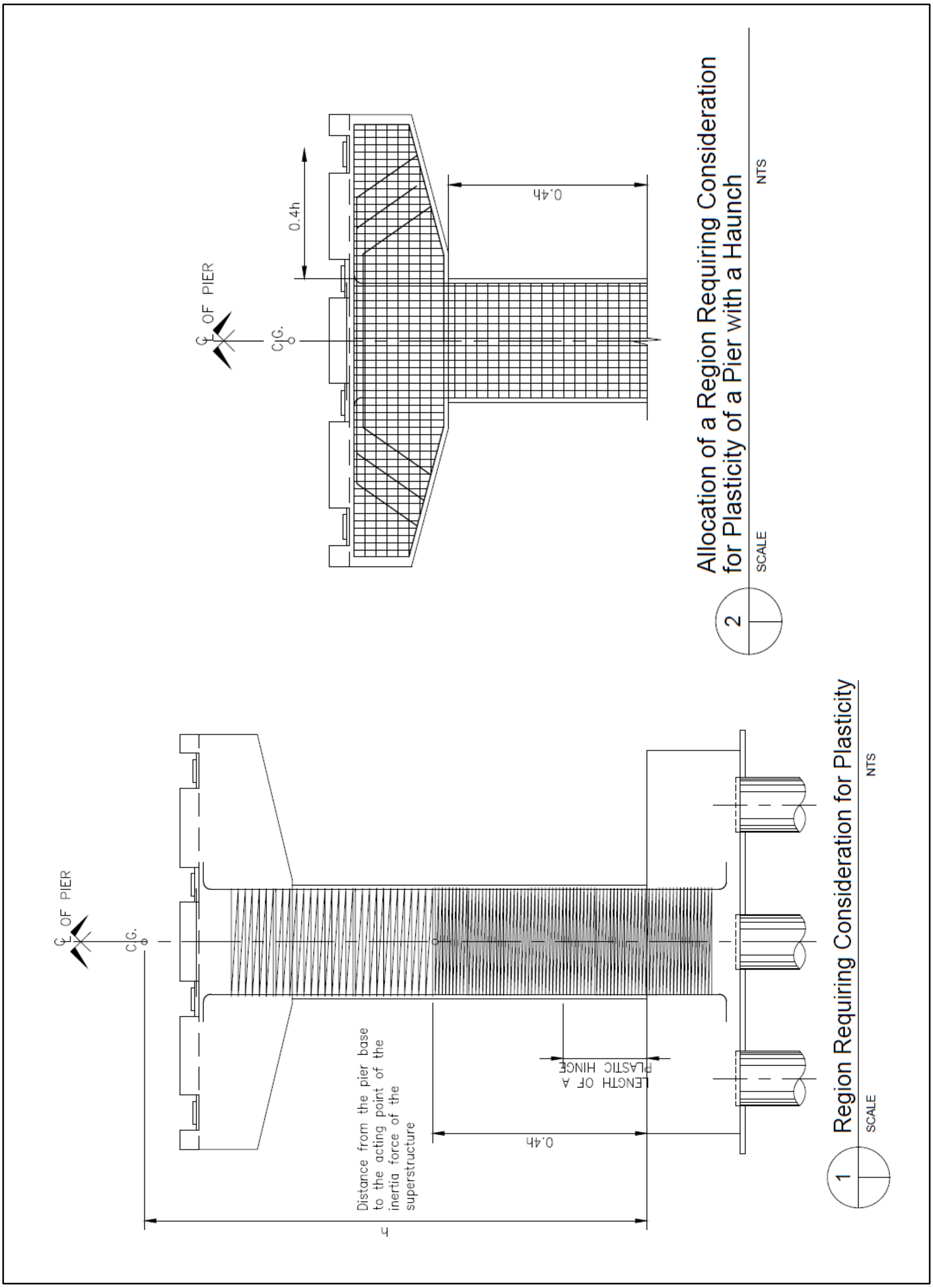
Figure 12.2-4 Lateral Spreading Force applying to Pier

Intentionally Blank

APPENDIX: STRUCTURAL DETAILS

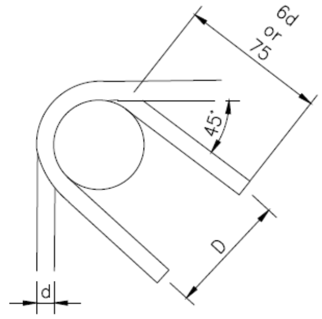
APPENDIX: Structural Details

A1. Plastic Hinge Part



Note: The following are the JRA (Japan Road Associations) recommended structural details:

A2. Anchorage of Hoops



1 TYP. SEISMIC HOOK DETAIL

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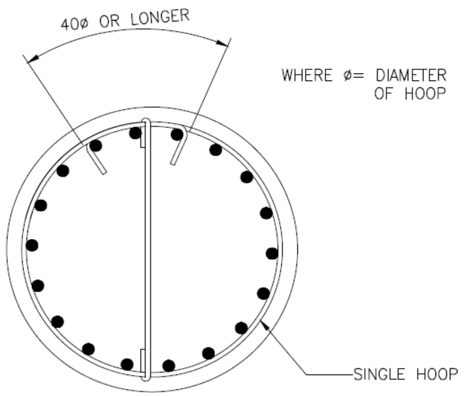

2 FOR CIRCULAR COLUMN

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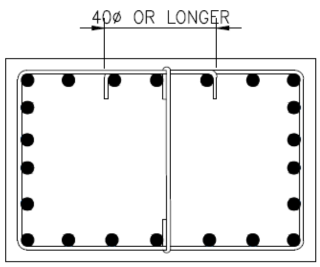
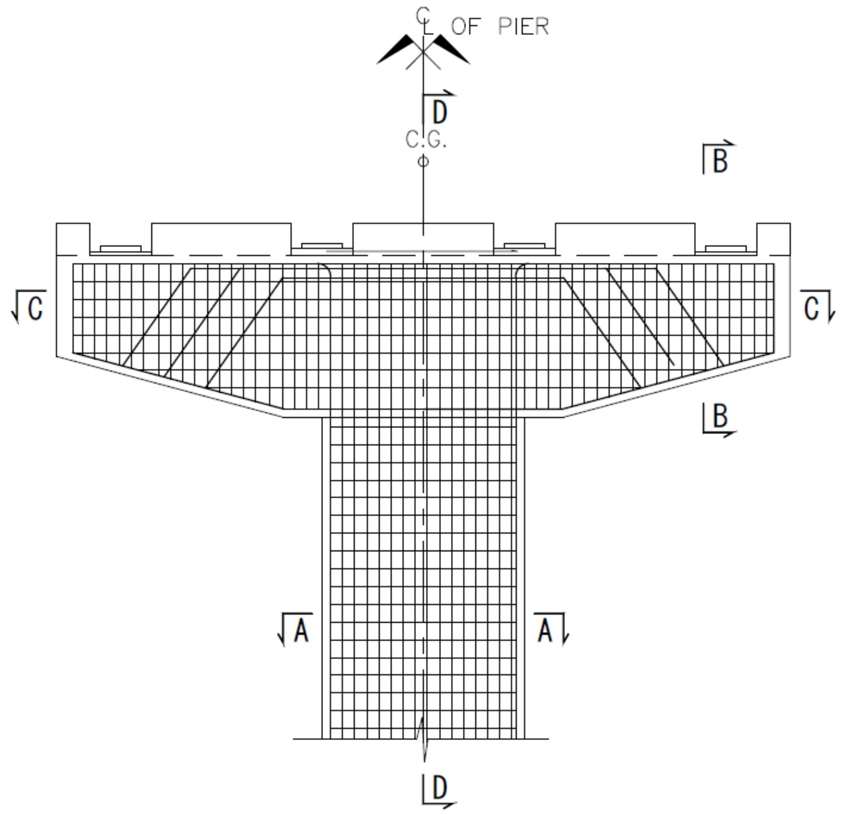


3 FOR RECTANGULAR COLUMN

SCALE NTS

A3. Standard Reinforcement at the Joint of Column and Beam



1 Standard Reinforcement at the Joint Region of a Column and a Beam

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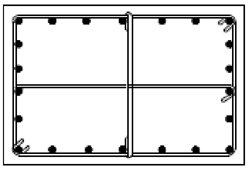
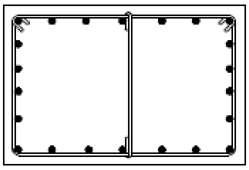
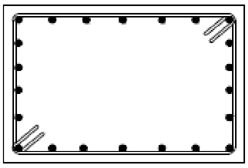


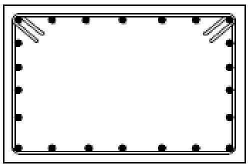
(a.) Example of column reinforcement in the region requiring consideration for plasticity



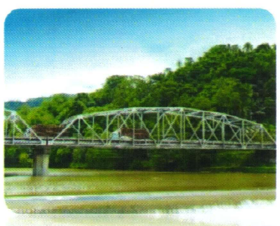
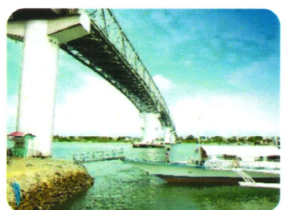
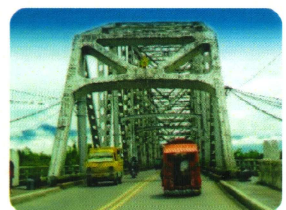
(b.) Example of beam reinforcement in the region requiring consideration for plasticity



(c.) Example of column reinforcement the at the joint



(d.) Example of beam reinforcement the at the joint

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