THE REPUBLIC OF THE PHILIPPINES
DEPARTMENT OF PUBLIC WORKS AND HIGHWAYS (DPWH)

THE PROJECT FOR STUDY ON IMPROVEMENT OF BRIDGES THROUGH DISASTER MITIGATING MEASURES FOR LARGE SCALE EARTHQUAKES IN THE REPUBLIC OF THE PHILIPPINES

FINAL REPORT

APPENDIX 1-B

DESIGN EXAMPLE (NEW BRIDGE) USING DPWH-BSDS

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JAPAN INTERNATIONAL COOPERATION AGENCY (JICA)

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APPENDIX 1-B

DESIGN EXAMPLE (NEW BRIDGE)
USING DPWH-BSDS

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PART 1

BASIC KNOWLEDGE OF EARTHQUAKE ENGINEERING AND STRUCTURAL DYNAMICS

1. Basic Knowledge of Earthquake Engineering

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.

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 1.2-1.

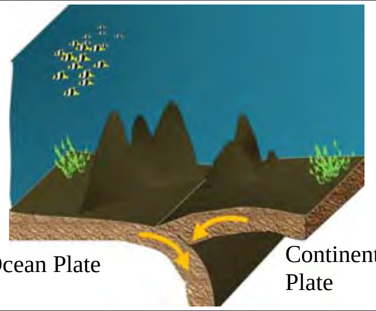
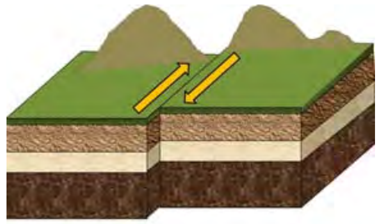
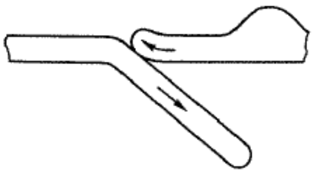
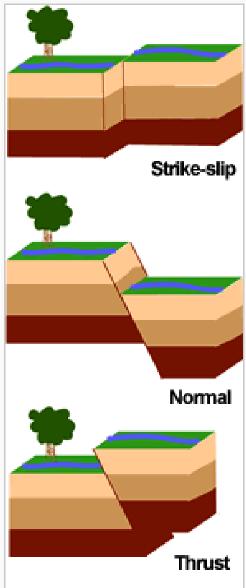
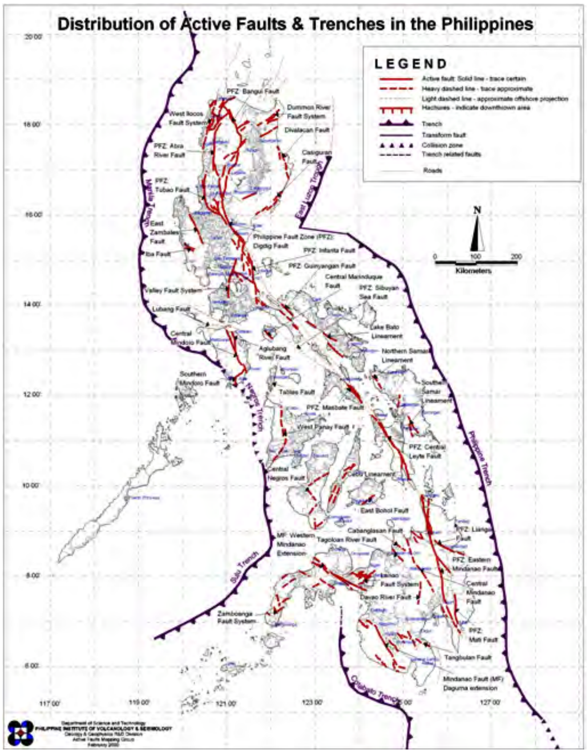
Plate Boundary Type of Earthquake

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 1.2-1 Types of Earthquake

Type	Plate Boundary Earthquake	Active Fault Earthquake
Image	 Ocean Plate Continental Plate	
Mechanism of Occurrence of Earthquake	Continental Plate Ocean Plate  (a) Ocean plate slips in continental plate  (b) Continental plate is slipped in by ocean  (c) The stress of continental plate is released	
Locations of Plate Boundary and Active Faults in the Philippines	 (Source: PHIVOLCS)	

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 1.3-1. 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 1.3-1

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 1.3-2. 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.

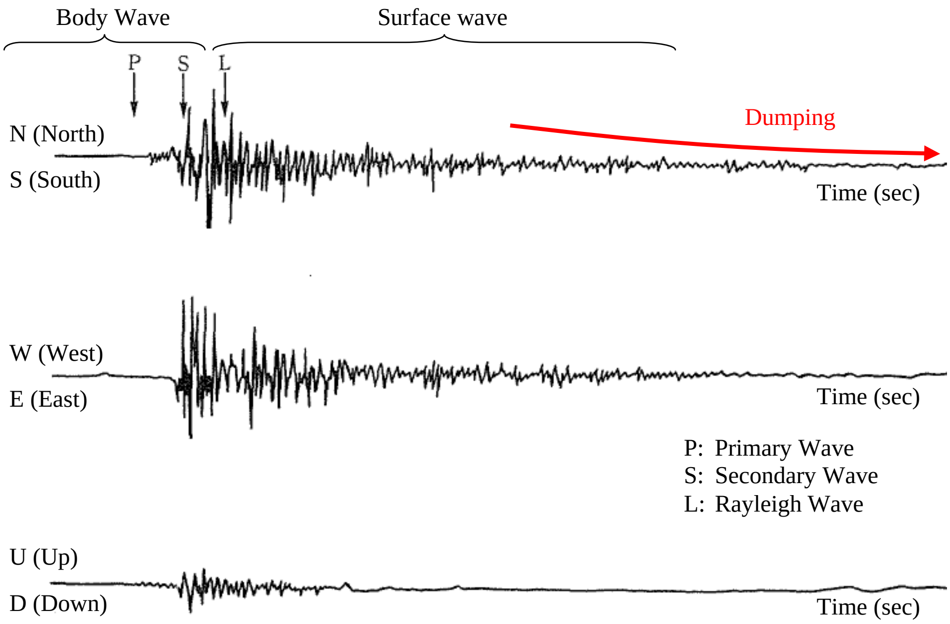
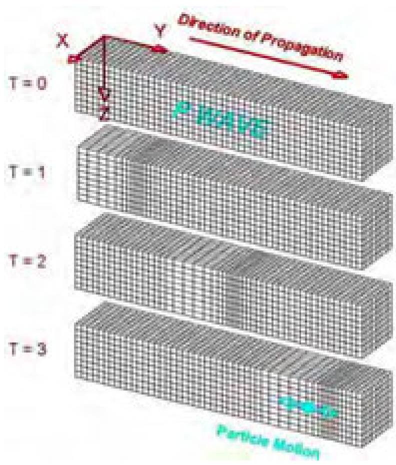
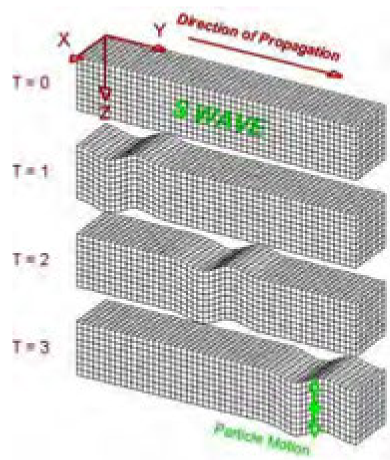
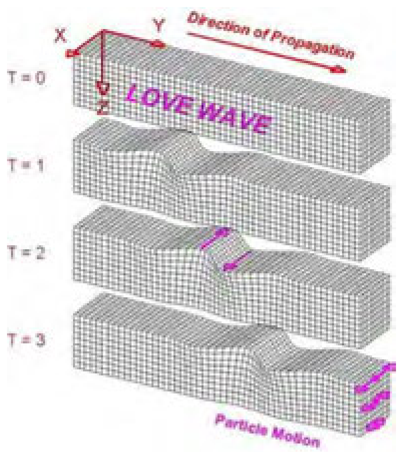
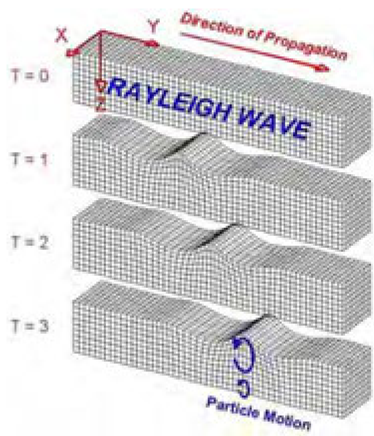
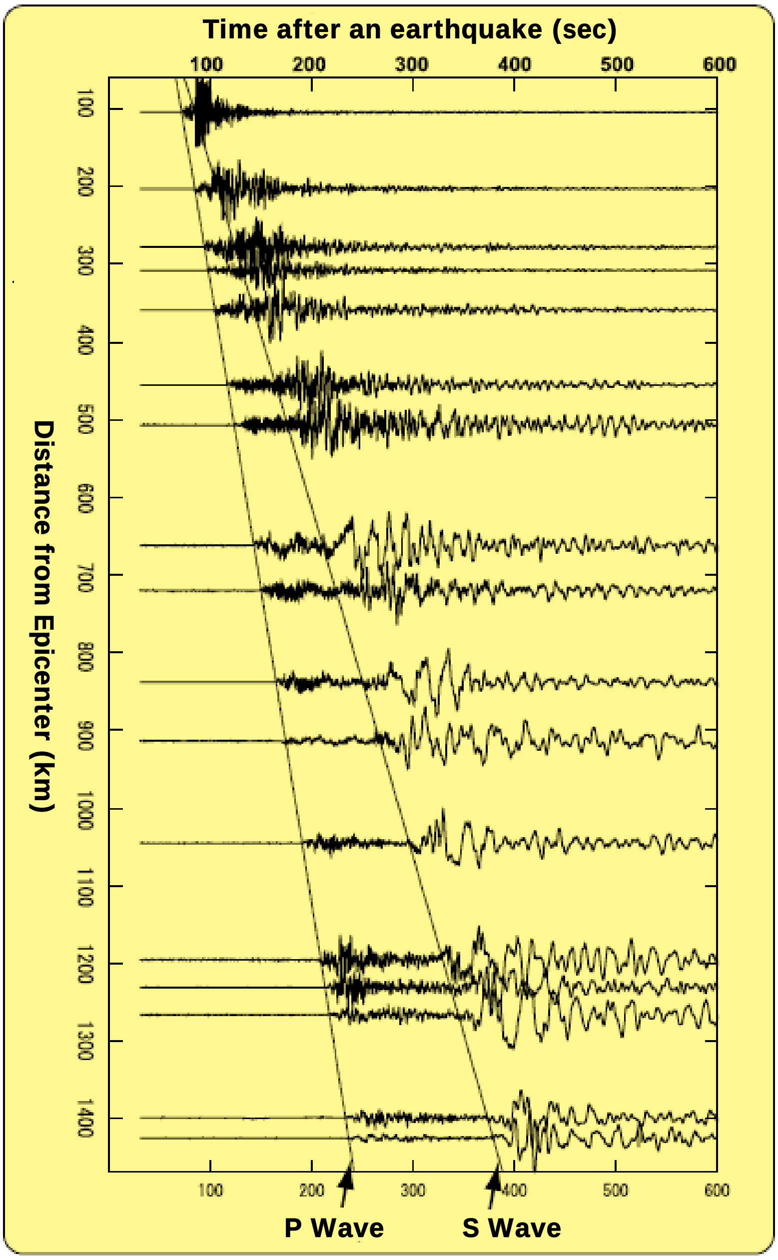


Figure 1.3-1 Example of Seismic Wave in Different Directions

Table 1.3-1 Kinds of Seismic Wave Transmission

Body Wave	
Primary Wave (P wave)	Secondary Wave (S wave)
	
Surface Wave	
Love Wave	Rayleigh Wave
	

(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)

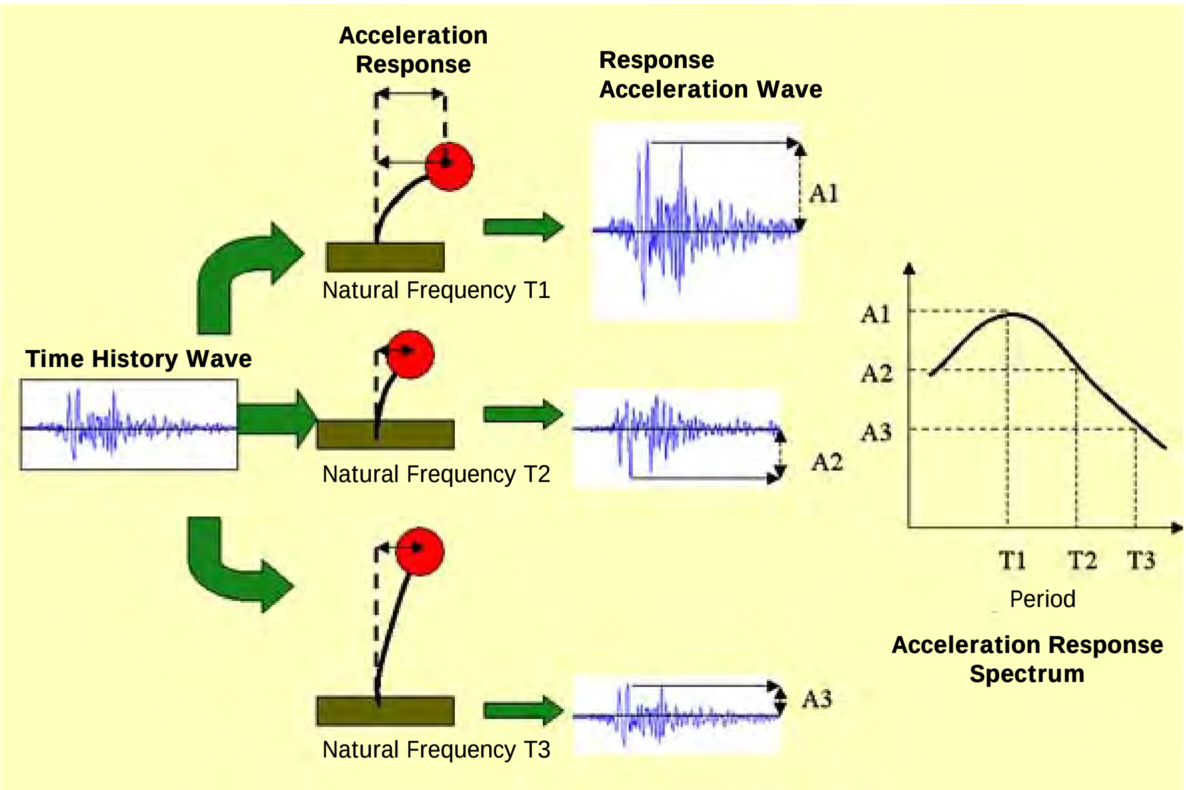
Figure 1.3-2 Example of Seismic Wave Transmission to Different Locations

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.

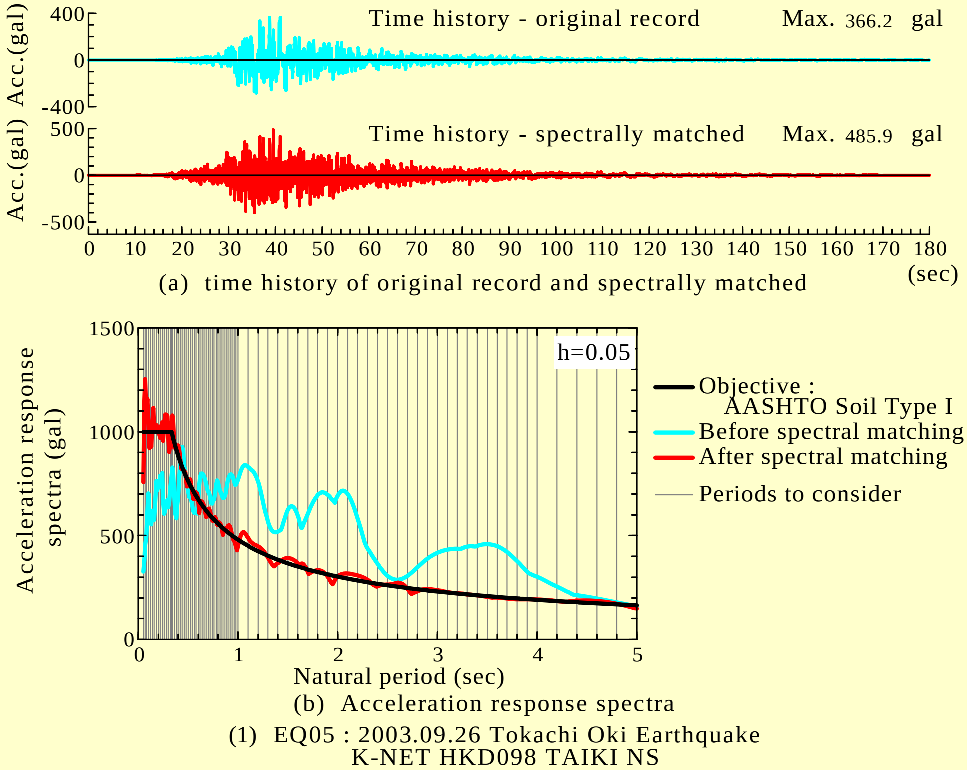
As shown in Figure 1.4-1, a response spectrum is simply a plot of the peak of a series of steady-state response with single-degree-of-freedom system varying natural frequency that are forced into motion by the same base vibration. The resulting plot can then be used to pick off the response of any linear system, given its natural frequency of vibration. In the case of acceleration, the response spectrum is called an acceleration response spectrum.

Figure 1.4-2 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.

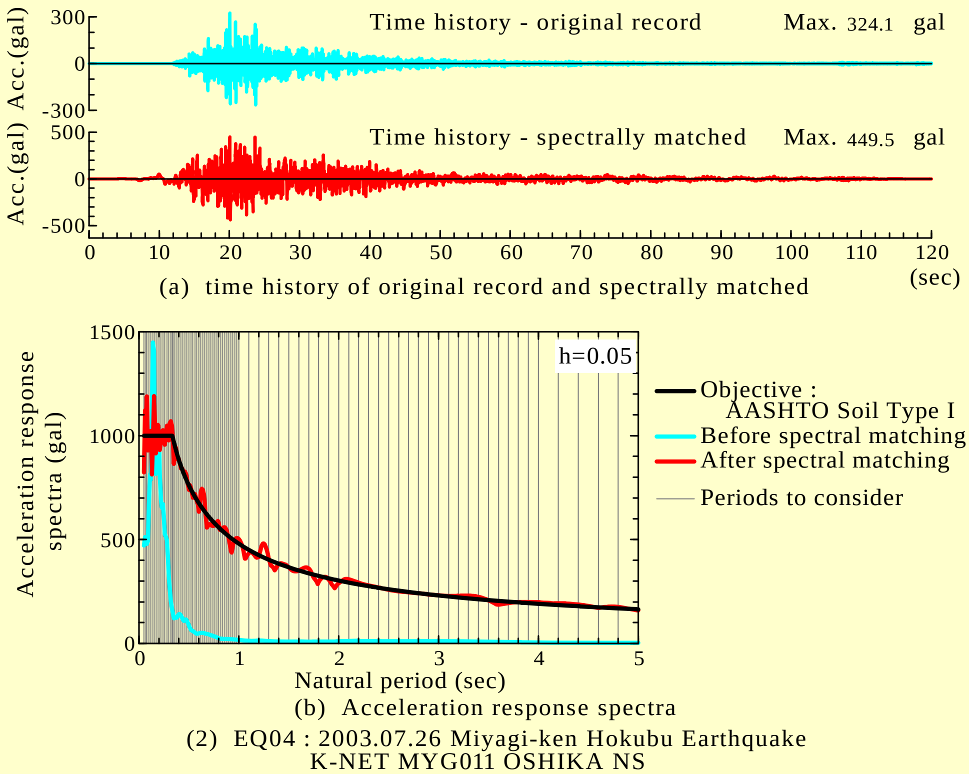


(Source: Japan Meteorological Agency)

Figure 1.4-1 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)

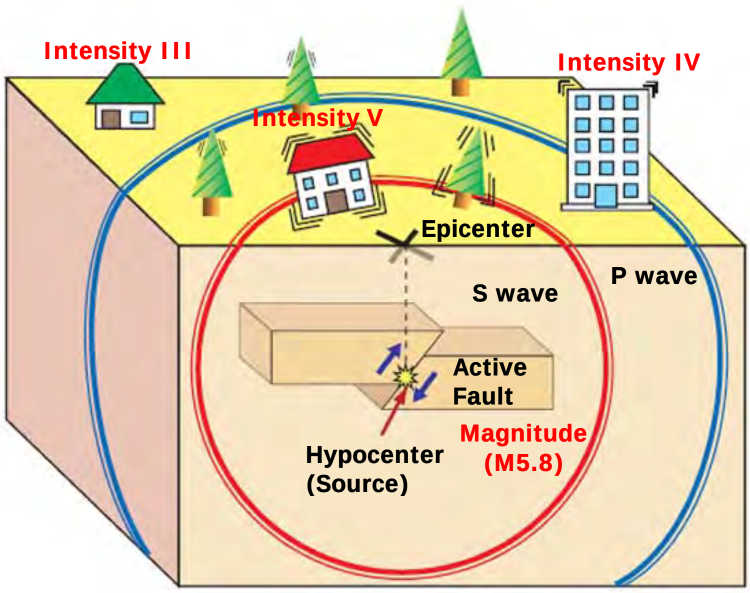
Figure 1.4-2 Example of Time History Earthquake Wave and Acceleration Response Spectrum

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 1.5-1 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 1.5-1.

Table 1.5-1 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 run 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.

Scale		PGA (g values)	Description
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

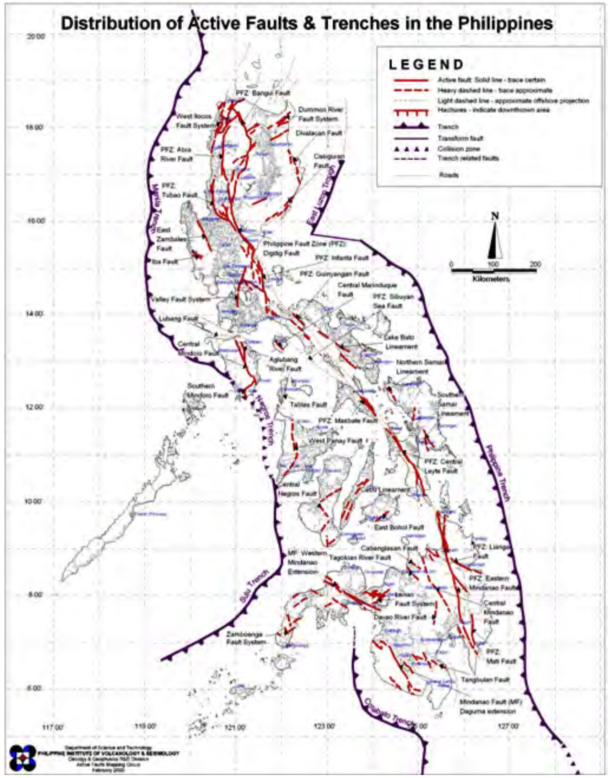
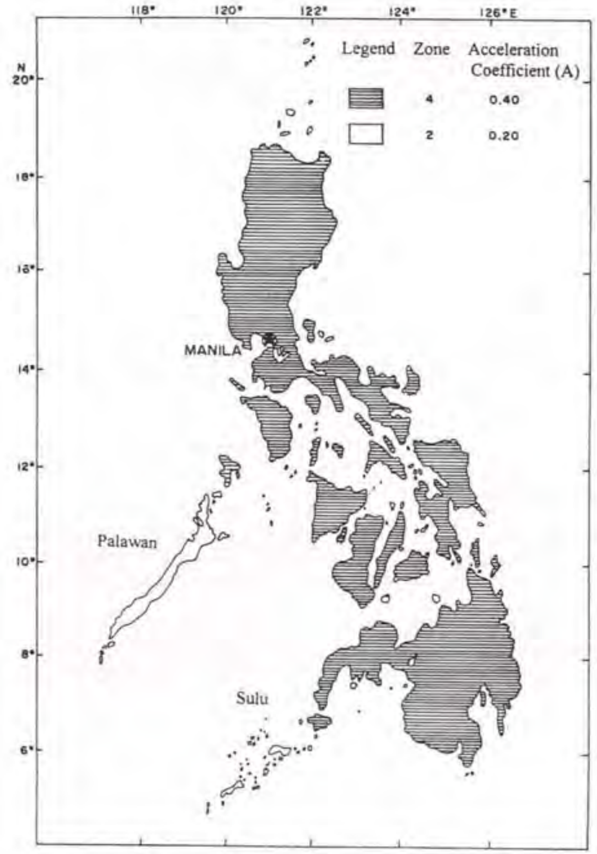
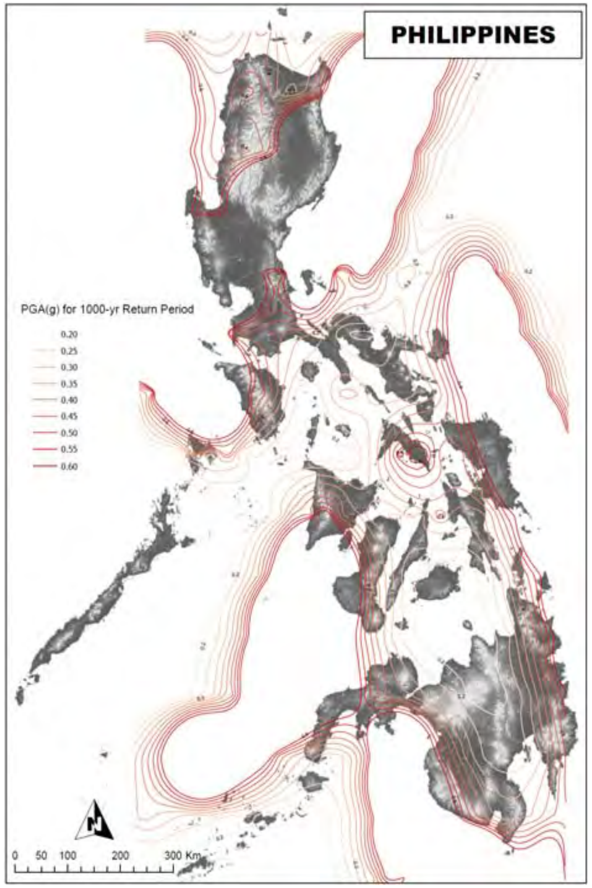
The PHIVOLCS Earthquake Intensity Scale (PEIS) described above show the destructivity impact of earthquakes qualitatively. However, PEIS is not used for bridge seismic design. The bridge seismic design expresses the strength of earthquake by the seismic coefficient or Peak Ground Acceleration (PGA) of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

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

Table 1.5-2 shows a comparison including (1) the location of existing trench and fault, which was shown in Table 1.2-1; (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) he proposed Peak Ground Acceleration (PGA) map provided by the Project.

Table 1.5-2 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

2. Basic Knowledge of Structural Dynamics

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.

2.2 Characteristic Vibration of Structure and Seismic Load

<u>Normal Mode</u>

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 2.2-1, 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 2.2-2 shows the case of cantilever such as bridge pier.

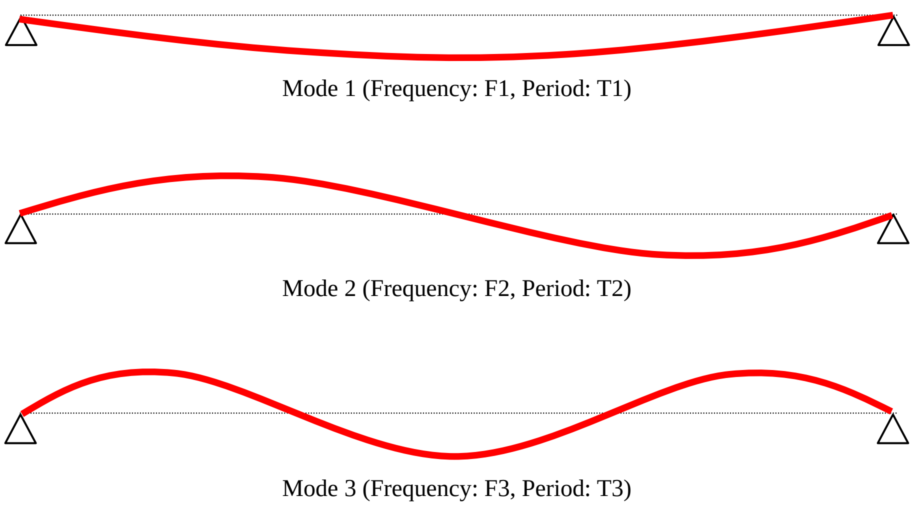


Figure 2.2-1 Example of Normal Modes of Beam

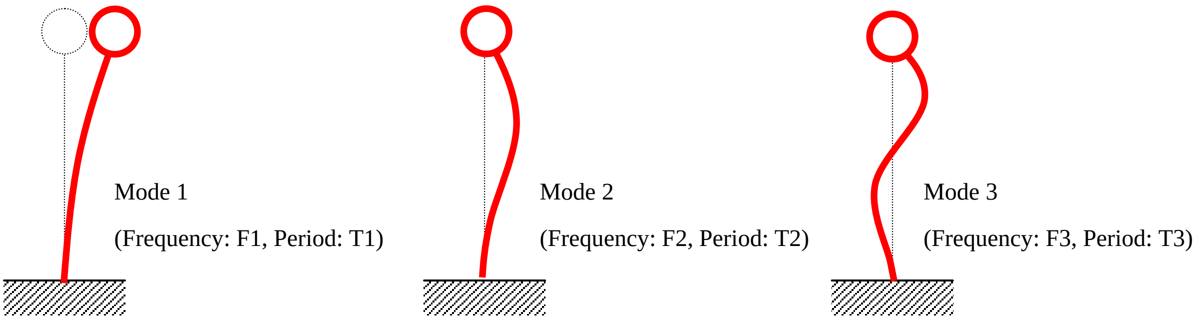


Figure 2.2-2 Example of Normal Modes of Cantilever

<u>Resonance and Forced Vibration</u>

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.

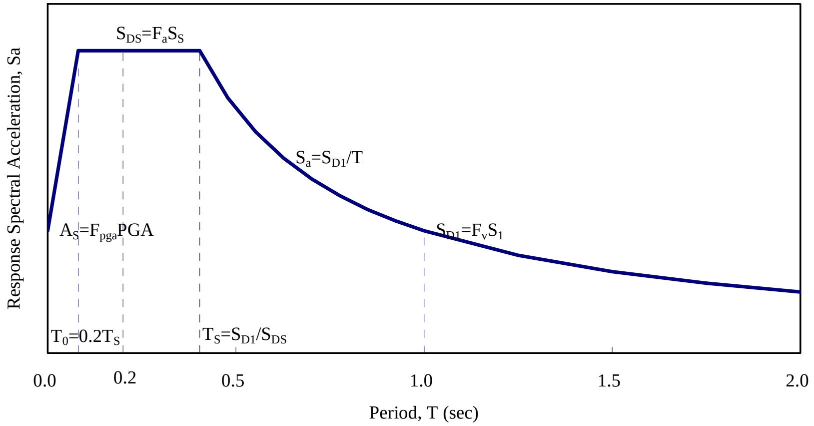
Forced vibration is a vibration caused forcibly by receiving the external force to fluctuate such as earthquakes. When the periods of forced vibration is the same or close to the natural frequency of the bridge, the vibration occurs remarkably. It is also called as resonance.

<u>Acceleration Response Spectrum and Vibration Mode</u>

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 2.2-1 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 2.2-3. 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 bridges.

Table 2.2-1 Design Response Spectrum, Bridge Seismic Design Specification (BSDS)



$A_s = F_{pga}PGA$

$S_{DS} = F_aS_s$

$S_{D1} = F_vS_1$

$S_a = (S_{DS}-A_S)T/T_0+A_S$ for periods less than or equal to T_0

$= S_{DS}$ for periods greater than or equal to T_0 and less than or equal to T_S

$= S_{D1}/T$ for periods greater than T_S

$T_0 = 0.2T_S$

$T_S = S_{D1}/S_{DS}$

As : acceleration coefficient

S_{DS} : design spectral acceleration coefficient at 0.2-sec period

S_{D1} : design spectral acceleration coefficient at 1.0-sec period

PGA : peak horizontal ground acceleration coefficient on Class B rock

S_s : 0.2-sec period spectral acceleration coefficient on Class B rock

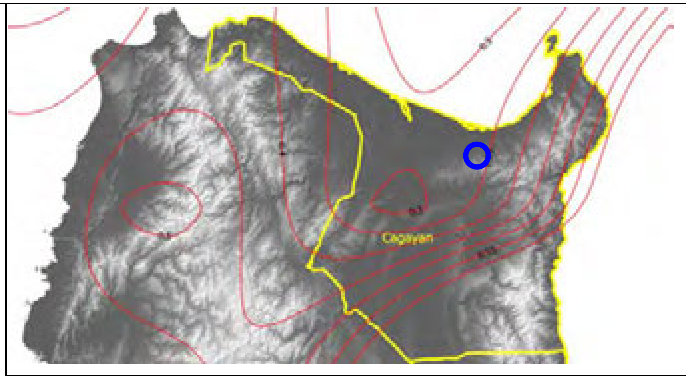
S_1 : 1.0-sec period spectral acceleration coefficient on Class B rock

F_{pga} : site coefficient for peak ground acceleration deified in Article 3.4.2.3

F_a : site coefficient for 0.2-sec period spectral acceleration specified in Article 3.4.2.3

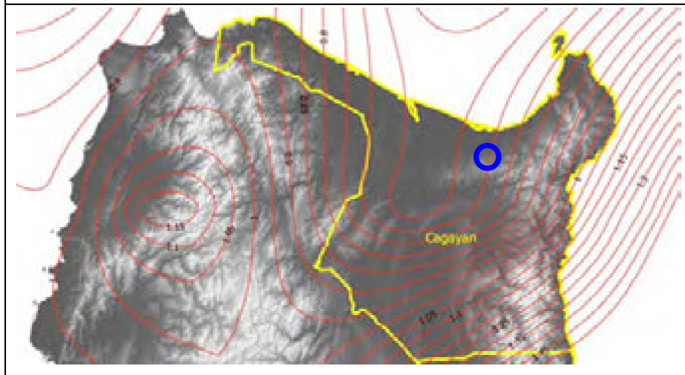
F_v : site coefficient for 1.0-sec period spectral acceleration specified in Article 3.4.2.3

T : period of vibration (sec)



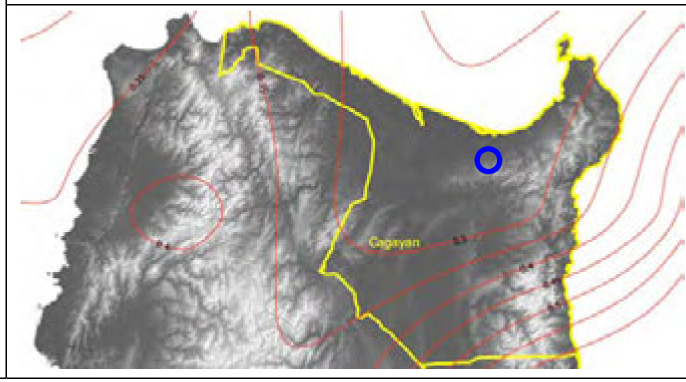
PGA (g)
for 1,000-year Return Period

Example: PGA (g) =0.34



SS (g) at 0.2 sec
for 1,000-year Return Period

Example: PGA (g) =0.70



S1 (g) at 1.0 sec
for 1,000-year Return Period

Example: PGA (g) =0.28

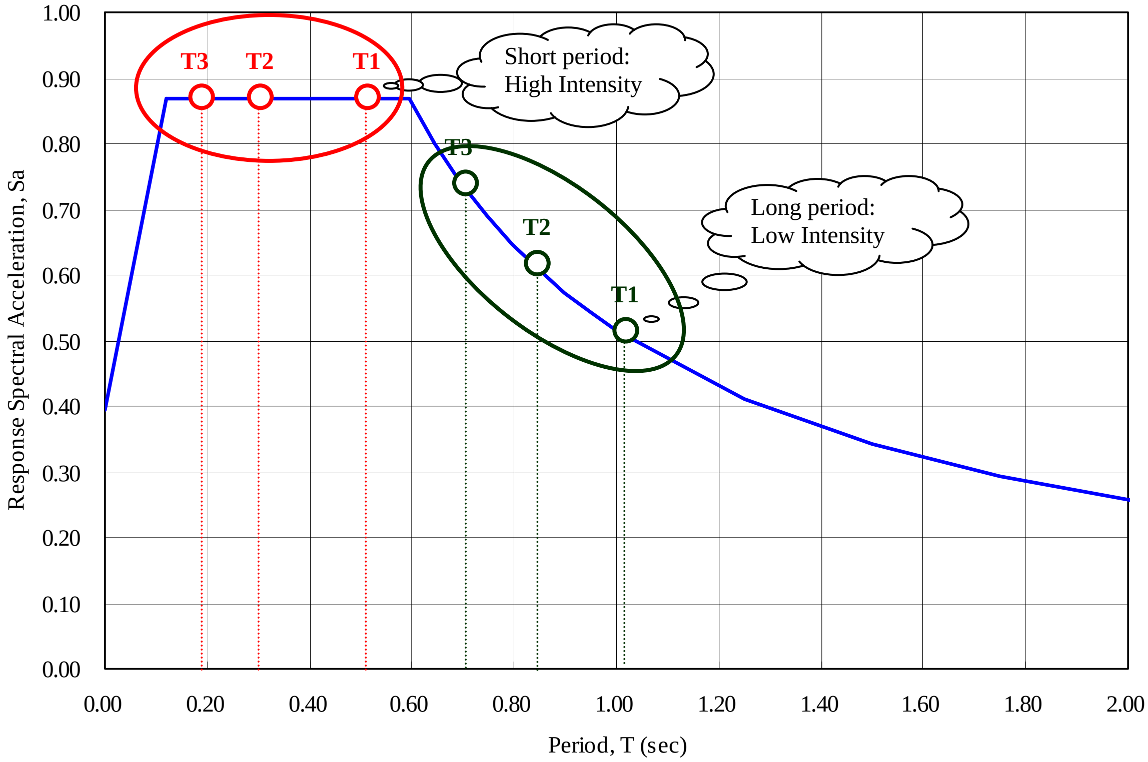


Figure 2.2-3 Example of Design Acceleration Spectrum (Soil Type I, 5% dumping)

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 1.3-1, 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.

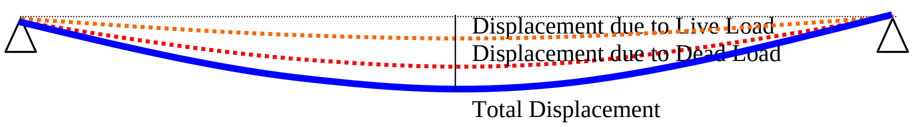
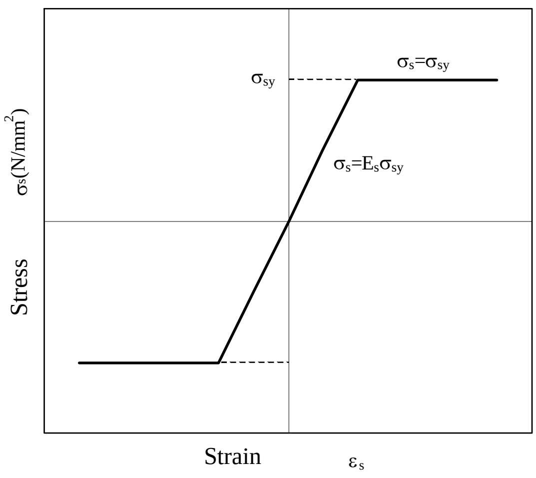


Figure 2.3-1 Example of Superposition Relation in Linear Property

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 2.3-2) 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 Figure 2.3-3.



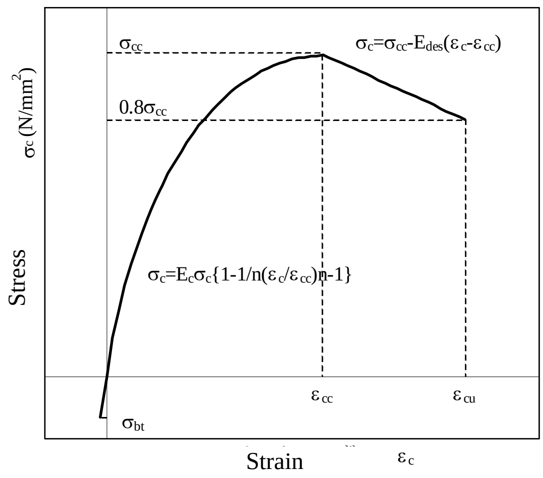


Figure 2.3-2 Ideal Stress-Strain Relation of Reinforcing Bar **Figure 2.3-3 Ideal Stress-Strain Relation of Concrete**

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 2.3-4. 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 2.3-5 shows an example of historical curve of the bending moment-curvature relation at pier bottom obtained by Non-linear time history response analysis.

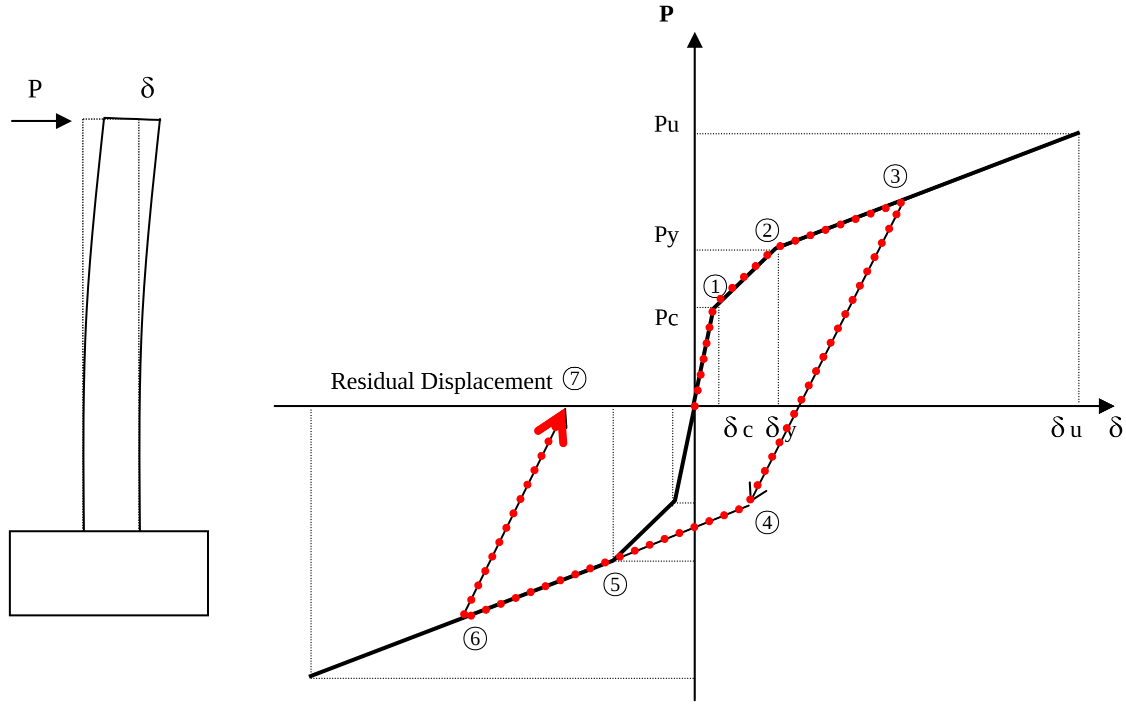


Figure 2.3-4 Non-linear Behavior of Reinforced Concrete Pier

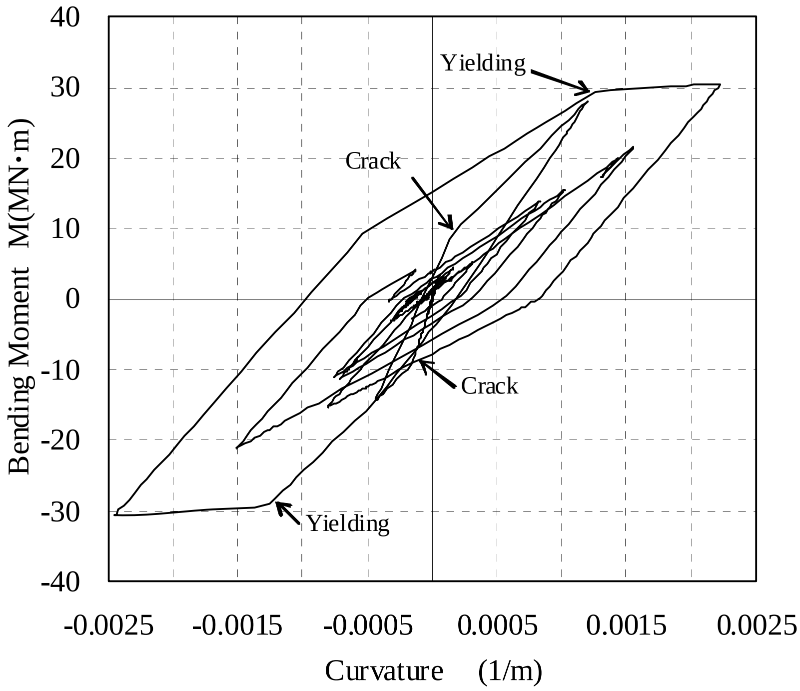


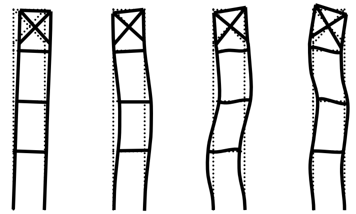
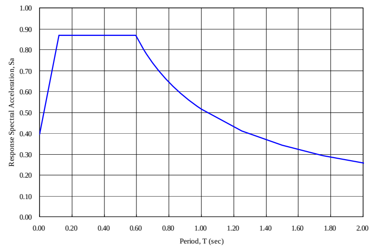
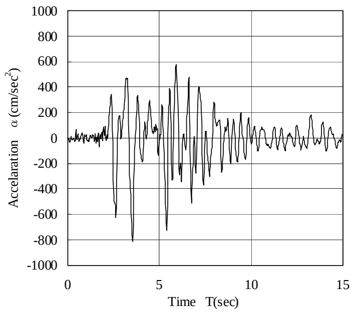
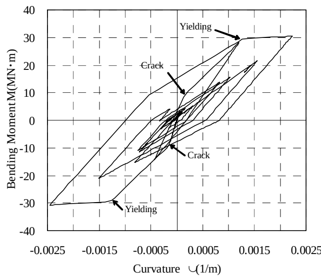
Figure 2.3-5 Example of Historical Curve of the Bending Moment-Curvature at Pier Bottom

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 2.4-1. These analysis methods have their own characteristics and the method shall be selected according to the type of bridge.

Table 2.4-1 Major Dynamic Analysis Methods for Bridge Seismic Design

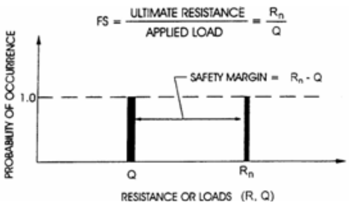
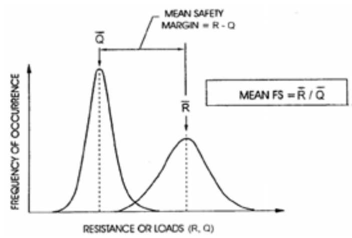
Analysis Method	Description	Analytic Model	Input Force for Seismic Design	Major Output	Remarks
Eigenvalue Analysis	Eigenvalue analysis is an analysis to obtain the vibration characteristics of the structure itself (natural period and mode shape). When the natural period of the structure is close to the 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 β Wilson θ Runge-Kutta

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 2.5-1.

Table 2.5-1 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 : η Factor 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

PART 2

DESIGN EXAMPLE-1:
DESIGN EXAMPLE OF SIMPLY-SUPPORTED BRIDGE

1. Fundamental Design Conditions for Bridge Seismic Design Example

1.1 Outline of Seismic Design Example of Pier with Pile Foundation

The design example is explained along with the following flowchart, which covers the basic process of seismic design in accordance with Bridge Seismic Design Specifications (BSDS).

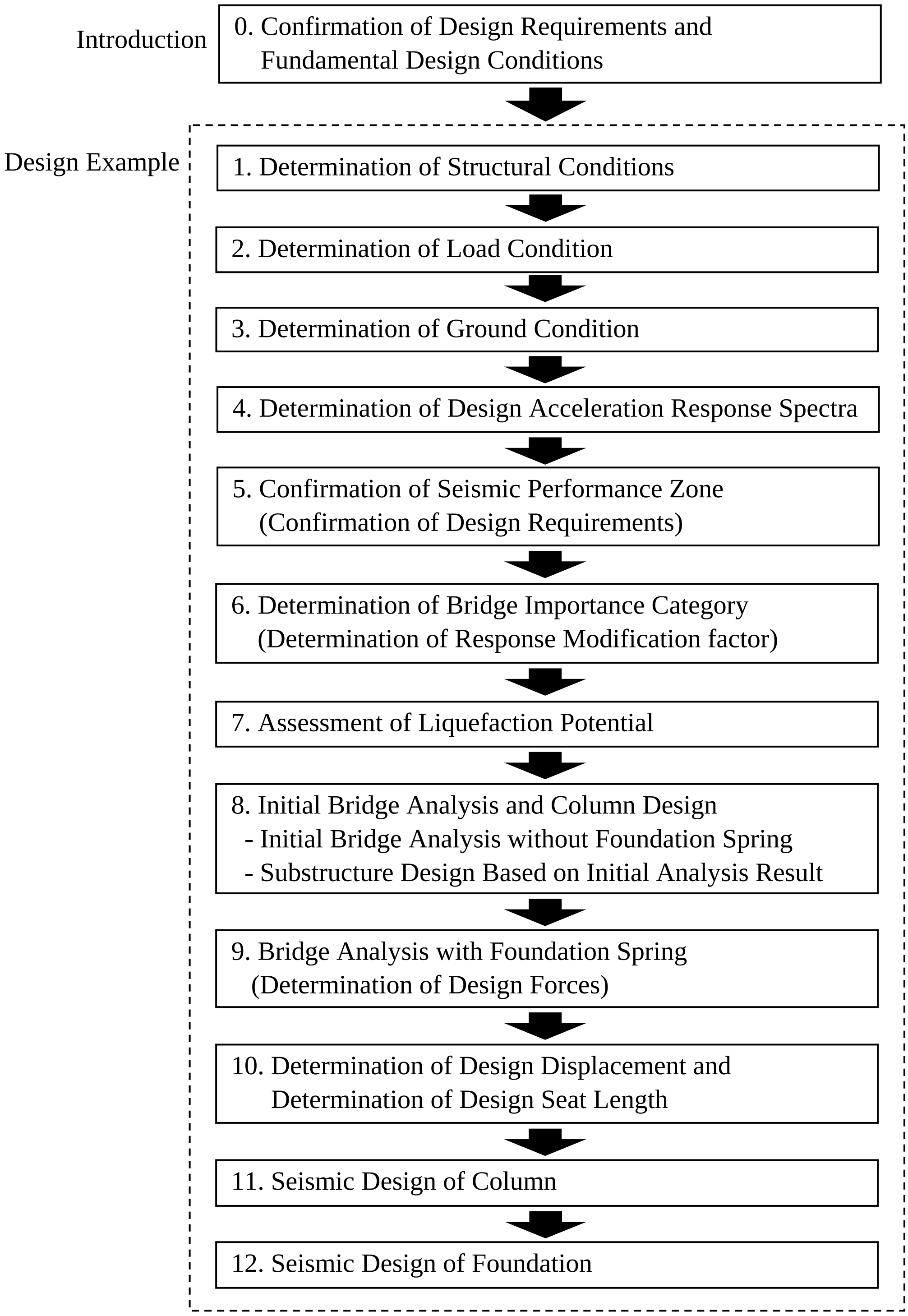


Figure 1.1-1 Outline of Seismic Design Example

1.2 Confirmation of Design Requirements and Fundamental Design Conditions

1.2.1 Bridge Importance (Bridge Operational Classification) BSDS (Article 3.2)

For the purpose of seismic design, bridges shall be classified into one of the following three operational categories, as shown in the following table. DPWH or those having jurisdiction shall classify the bridges into one of the above three operational categories. 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.

Table 1.2.1-1 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)** (Selected)	- 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)**	- All other bridges not required to satisfy OC-I or OC-II performance

In the design example, "OC-II (Essential Bridge)" is applied for the operational classification. Accordingly, the following two conditions are given to designers;

1) Design seismic force

"An earthquake with 1,000-year return period" shall be used for the seismic design force.

2) Response Modification Factors for Substructures

Response modification factors for Substructures (hereafter, called as R-factor) for "Essential Bridge", shall be applied to design of piers/columns. The relationship between "R-factor" and "Operational Classification" is shown in the following table. In the design example, "R=2.0" is selected under single column condition.

Table 1.2.1-2 Response Modification Factors

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 (Selected)	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

1.2.2 Seismic Performance Requirements BSDS (Article 3.3)

In BSDS, the following three requirements are given as seismic performance requirements. However, in the design example, only performance requirement against Level-2 EQ (Large earthquakes with a 1000-year return period) is verified for simplification. Also, as a part of design example for unseating prevention system, only design of seat length is explained in the design for simplification.

[Seismic performance requirements]

1) Bridges shall have the following three levels of seismic performance.

- Seismic Performance Level 1 (SPL-1)
  Performance level of a bridge to ensure its normal sound functions during an earthquake
- Seismic Performance Level 2 (SPL-2)
  Performance level of a bridge to sustain limited damages during an earthquake and capable of recovery immediately for critical bridges and within a short period for essential bridges
- Seismic Performance Level 3 (SPL-3)
  Performance level of a bridge to ensure safety against collapse during an earthquake

2) Bridges classified under "operational classification" shall conform to the performance requirements given in the following table and the design earthquake ground motion.

Table 1.2.2-1 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)	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)* (Selected)	SPL-3 *(May suffer damage but should not cause collapse of bridge or any of its structural elements)*

3) Bridges shall be designed to ensure that unseating of superstructures can be prevented, even if structural failures may occur due to structural behavior or ground failures which were not expected in the seismic design.

1.2.3 Load Condition for Seismic Design

1.2.3.1 Load Combination and Load Factors BSDS (Article 1.5)

The combination of factored extreme force effects for "Extreme Event I load combination", as stated in "Article-3.4 of the AASHTO LRFD Bridge Design Specifications (2012)", shall be used for seismic design. With this regards, the load factor for live load γ_{EQ} shall be 0.50.

The detail of the applied loads and load combination for seismic design is shown in the following table.

Table 1.2.3-1 Load Combination and Load Factors for "Extreme Event I"

Load Combination / Limit State	Permanent Loads: DC DW DD EL PS EH ES EV CR SH	Transient Loads: LL	Transient Loads: WA	Transient Loads: FR	Transient Loads: EQ
Extreme Event I	γ_p	0.5	1.0	1.0	1.0

γ_p: Load factors for permanent loads (refer to AASHTO LRFD for the detail)

[Permanent Loads]
- DC: Dead load of structural components and nonstructural attachments
- DW: Dead load of wearing surface and utilities
- DD: Downdrag force
- EL: Miscellaneous locked-in force effects resulting from the construction process, including jacking apart of cantilevers in segmental construction
- PS: Secondary forces from post-tensioning
- EH: Horizontal earth pressure load
- ES: Earth surcharge load
- EV: Vertical pressure from dead load of earth fill
- CR: Force effects due to creep
- SH: Force effect due to shrinkage

[Transient Loads]
- LL: Vehicular live load
- WA: Water load and stream pressure
- FR: Friction load
- EQ: Earthquake load

In the design example, the following load combination is applied.

Load combination: [1.0*(DC+DW+EH+EV)] + [0.5LL] + [1.0EQ]

1.2.3.2 Unit Weight

The following unit weights are applied in the design example.

- Reinforced concrete: γc= 24.0 (kN/m3); rounded up for modification

  Note: - (Unit weight of concrete)= 2320 (kg/m3); normal density concrete

  - (Unit weight of re-bars in 1m^3 of concrete)= 200 (kg/m3)

- Wearing surface: γ_{ws}= 22.5 (kN/m3)
- Water: γw= 10.0 (kN/m3)
- Soil: γt= (result of soil tests)

1.2.4 Material Properties

The following material properties are applied in this design example.

Table 1.2.4-1 Material Properties

Material	Strength	Remarks
Concrete	fc'= 28.0 (MPa); Compressive Strength at 28 days	-To be applied to all the substructure members
Re-bars	Fy= 415 (N/mm2); Grade60	-To be applied to all the substructure members - Applicable diameter: D16, D20, D25, D28, D32, D36

- Young's modulus
  - Concrete: Ec= 4800$\sqrt{\text{fc'}}$ = 25,000 (MPa); rounded down for modification
  - Steel: Es= 20,000 (MPa)

1.2.5 Ground Condition for Seismic Design BSDS (Article 3.5.1)

Ground types for seismic design shall be classified, in principle, in accordance with the types defined in following table, in accordance with the ground characteristic value T_G defined by the following equation. When the ground surface lies on the same level as the surface of a base ground surface for seismic design, the ground type shall be Type-I.

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

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 layers from the ground surface to the surface of the base ground surface for seismic design

Table 1.2.5-1 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 ≤T_G<0.6
Type-III	Soft ground and alluvial ground	0.6≤T_G

In the design example, average shear elastic wave velocity, "V_{si}" is estimated with the following equations.

- For cohesive soil layer,

$$V_{si} = 100 N_i^{1/3} \ (1 \le N_i \le 25)$$

- For sandy/cohesionless soil layer,

$$V_{si} = 80 N_i^{1/3} (1 \le N_i \le 50)$$

Where,

Ni: Average N-value of the i-th soil layer obtained from SPT

Note: When the N-value is 0, the value of V_{si} can be taken as 50 (m/s).

In the design example, the following bridge site is chosen for the ground conditions.

- Lambingan Bridge; consideration of liquefiable ground condition (pile foundation)

1.2.6 Design Acceleration Response Spectrum Methodology BSDS (Article-3.6)

The five-percent-damped-design response spectrum shall be taken as specified in the following figure. The spectrum shall be calculated using the mapped peak ground acceleration coefficients and the spectral acceleration coefficients from three acceleration-contour maps, scaled by the zero-, short-, and long-period site factors, Fpga, Fa, and Fv, respectively.

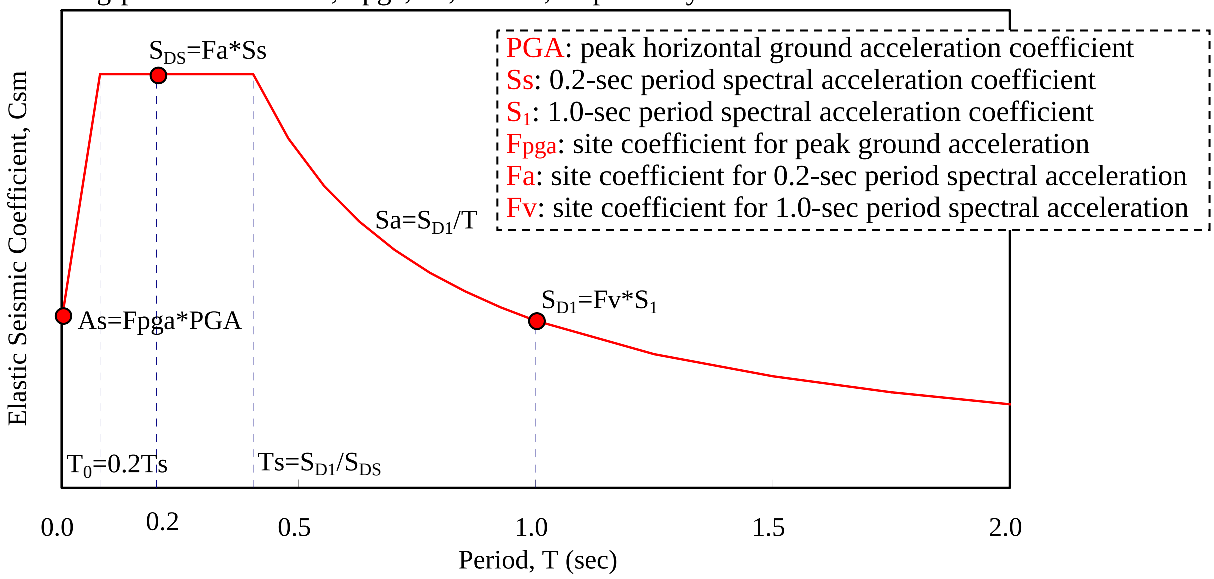


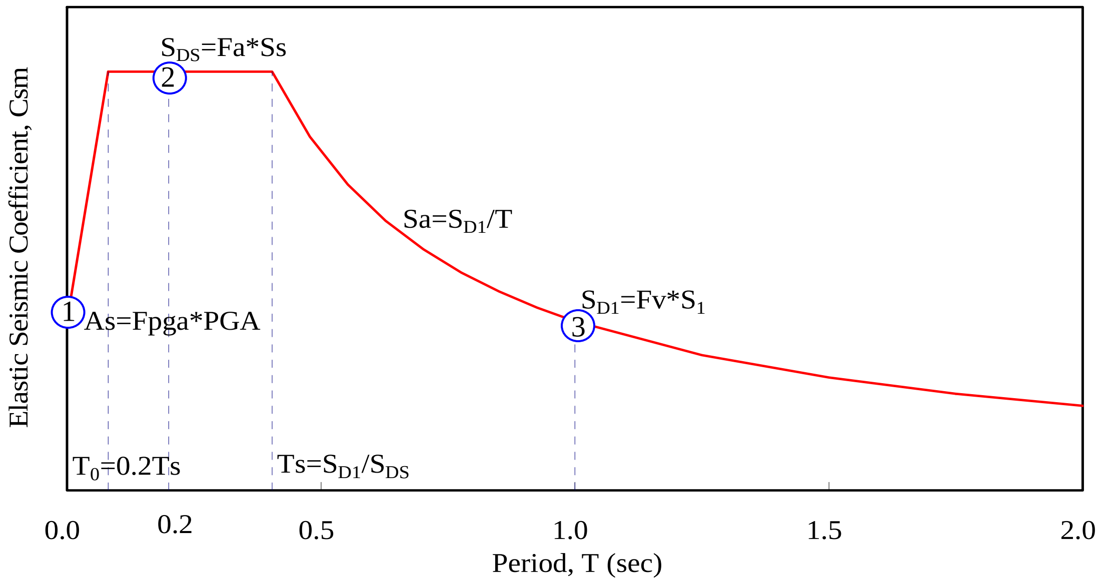
Figure 1.2.6-1 Design Response Spectrum

Design response spectrum can be formed along with the following four steps.

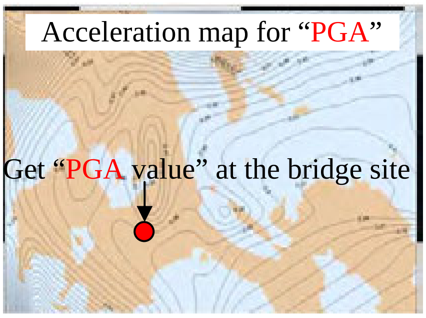
- Step-1: Identify specific values of PGA, Ss, and S_1 for bridge sites on the acceleration-contour maps.
- Step-2: Identify specific values of Fpga, Fa, and Fv in the each "site factor table".
- Step-3: Calculate and plot the coordinates of the following points in the graph.

  (0, Fpga*PGA), (0.2*Ts, Fa*Ss), (0.2, Fa*Ss), (S_{D1}/S_{DS}, Fa*Ss), (1.0, Fv*S_1)
- Step-4: Form spectrum by connecting the plotted points (Csm=S_{D1}/T, if "Ts<T")

(See the following figure for the reference.)



1 Acceleration Coefficient at T=0 (s), As=Fpga*PGA



Get "linearly-interpolated value of Fpga" with the table.

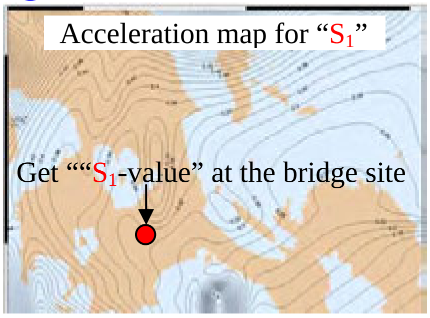
PGA (T=0)		Fpga (site factor for PGA)					
		PGA< 0.10	PGA= 0.20	PGA= 0.30	PGA= 0.40	PGA= 0.50	PGA≥ 0.80
Soil type	I	1.2	1.2	1.1	1.0	1.0	1.0
	II	1.6	1.4	1.2	1.0	0.9	0.85
	III	2.5	1.7	1.2	0.9	0.8	0.75

2 Design spectral acceleration coefficient at T=0.2 (s), S_{DS}=Fa*Ss

Get "linearly-interpolated value of Fa" with the table.

Ss (T=0.2)		Fa (site factor for Ss)					
		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 Design spectral acceleration coefficient at T=1.0, S_{D1}=Fv*S_1



Get "linearly-interpolated value of Fv" with the table.

S_1 (T=1.0)		Fv (site factor for S_1)					
		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 1.2.6-2 Formation of Design Response Spectrum

In the design example, the following bridge site is chosen for design response spectrum formation, corresponding to the ground condition of specific bridge site explained before.

- Lambingan Bridge; consideration of response spectra at the site with liquefiable ground condition

1.2.7 Analysis Requirements and Applied Methodology

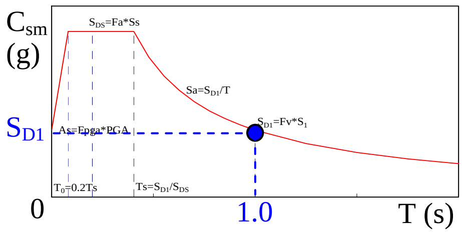
1.2.7.1 Seismic Performance Zones BSDS (Article-3.7)

Each bridge shall be assigned to one of the four seismic zones in accordance with the following table using the value of S_{D1} (based on the 1.0-sec period design spectral acceleration for the design earthquake).

If liquefaction-induced lateral spreading or slope failure that may impact the stability of the bridge could occur, the bridge should be designed in accordance with Seismic Zone 4 (SZ-4), regardless of the magnitude of S_{D1}.

Table 1.2.7-1 Seismic Zones

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
$0.50 \leq S_{D1}$	SZ-4



The seismic zones (SZ) reflect the variation in seismic risk across the country and are used to permit different requirements for methods of analysis, minimum support lengths, column design details, and foundation and abutment design procedures.

1.2.7.2 Analysis Requirements (for Single-span Bridges) BSDS (Article-4.1.2)

The following three analysis requirements are given for single-span bridges although multispan bridge condition is selected for the design example.

1) Seismic analysis is not required for single-span bridges, regardless of seismic zone.
2) Connections between the bridge superstructure and the abutments shall be designed for the minimum force requirements as specified in Article 4.2.1 (3.10.9 AASHTO).
3) Minimum support length requirements shall be satisfied at each abutment as specified in Article 4.1.4 (4.7.4.4 AASHTO).

1.2.7.3 Analysis Requirements (for Multispan Bridges) BSDS (Article-4.1.3)

The following four analysis requirements are given for multispan bridges. As explained below, either "multimode elastic method (MM)" shall be applied for multispan bridges whose importance is categorized as "Essential". However, in the design example, "uniform load elastic method (UL)" is applied for simplification of analysis.

[Analysis requirements]

1) For multi-span structures, the minimum analysis requirements shall be as specified in the following table.

Table 1.2.7-2 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

2) Except as specified below, bridges satisfying the requirements of the following table may be taken as "regular" bridges. Bridges not satisfying the requirements of the table shall be taken as "irregular" bridges.

Table 1.2.7-3 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

3) Curved bridges comprised of multiple simple-spans shall be considered to be "irregular" if the subtended angle in plan is greater than 20 degrees. Such bridges shall be analyzed by either the multimode elastic method or the time-history method.

4) A curved continuous-girder bridge may be analyzed as if it were straight, provided all of the following requirements are satisfied:

- The bridge is "regular" as defined in the above table, except that for a two-span bridge the maximum span length ratio from span to span must not exceed 2;
- The subtended angle in plan is not greater than 90 degrees; and
- The span lengths of the equivalent straight bridge are equal to the arc lengths of the curved bridge.

If these requirements are not satisfied, then curved continuous-girder bridges must be analyzed using the actual curved geometry.

1.2.7.4 Bridge Seismic Analysis Conditions in the Design Example

(1) Application of Single Degree of Freedom Method

As one of uniform load elastic methods, "Single Degree of Freedom Method" is applied in the design example. Vibration characteristics of the simply-supported bridge can be acquired with the method for both longitudinal and transverse directions. In order to acquire the realistic vibration characteristics of structures, foundations are modeled as spring in the design. The detail of the analysis model is illustrated below.

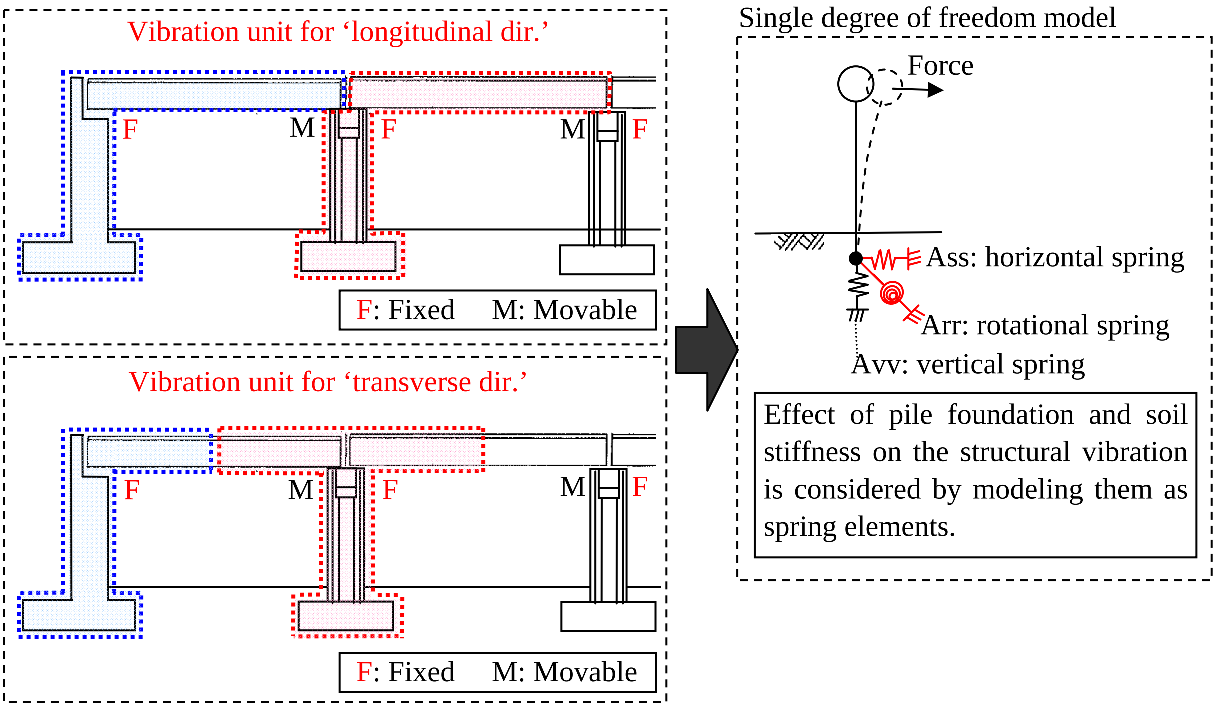


Figure 1.2.7-1 Design Model for Seismic Design of Pier

If the vibration unit is assumed with the above model, the natural period of the structure can be determined with the following equation.

$$T = 2 * \pi \sqrt{\frac{W}{g * K}} = 2 * \pi \sqrt{\frac{\delta}{g}} = \boxed{2.01\sqrt{\delta}}$$: natural period of the bridge structure (s)

δ=W/K

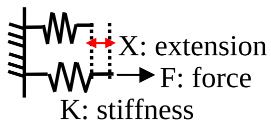
Input

$\delta = \boxed{\delta p + \delta o + \theta o * ho}$

: lateral displacement of the superstructure at application point of seismic inertial force from the superstructure (m)

[Hook's Law]

F=K*X ↔ W=K*δ



Where,

W: 80% of the substructure weight and the entire weight of the superstructure (kN)

g: gravity acceleration: g=9.8 (m/s^2)

K: stiffness of the structure (kN/m)

δ: lateral displacement of the superstructure at application point of seismic inertial force from the superstructure when 80% of the substructure weight and the entire weight of the superstructure act jointly in the direction of the seismic force (m); δ= δp+δo+θo*ho

δp: lateral displacement of the substructure body at application point of seismic inertial force from the superstructure (m);

$$\delta_p = \frac{Wu * h^3}{3 * E * I} + \frac{0.8 * W_p * h_p^3}{8 * E * I}$$

Wu: weight of superstructure under consideration (kN)

h:

Wp: weight of substructure (kN)

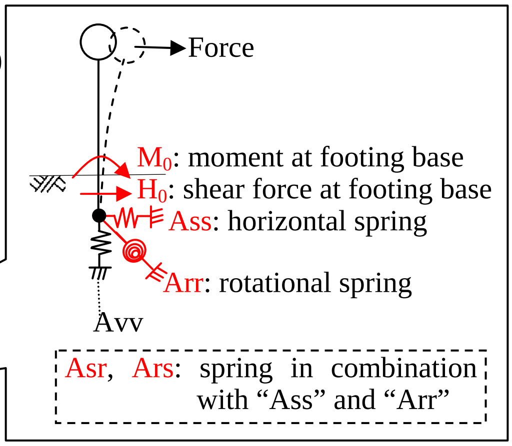
hp: height of the substructure (m)

E: Young’s modulus of the substructure (MPa)

I: moment of inertia of the substructure (m4)

δo: lateral displacement of footing (m);

$$\delta o = \frac{H_0 * Arr - M_0 * Asr}{Ass * Arr - Asr * Ars}$$



θo*ho: lateral displacement of footing caused by rotation of the pile cap (m);

$$\theta o = \frac{-H_0 * Ars + M_0 * Ass}{Ass * Arr - Asr * Ars}$$

The detail of the methodology is illustrated below.

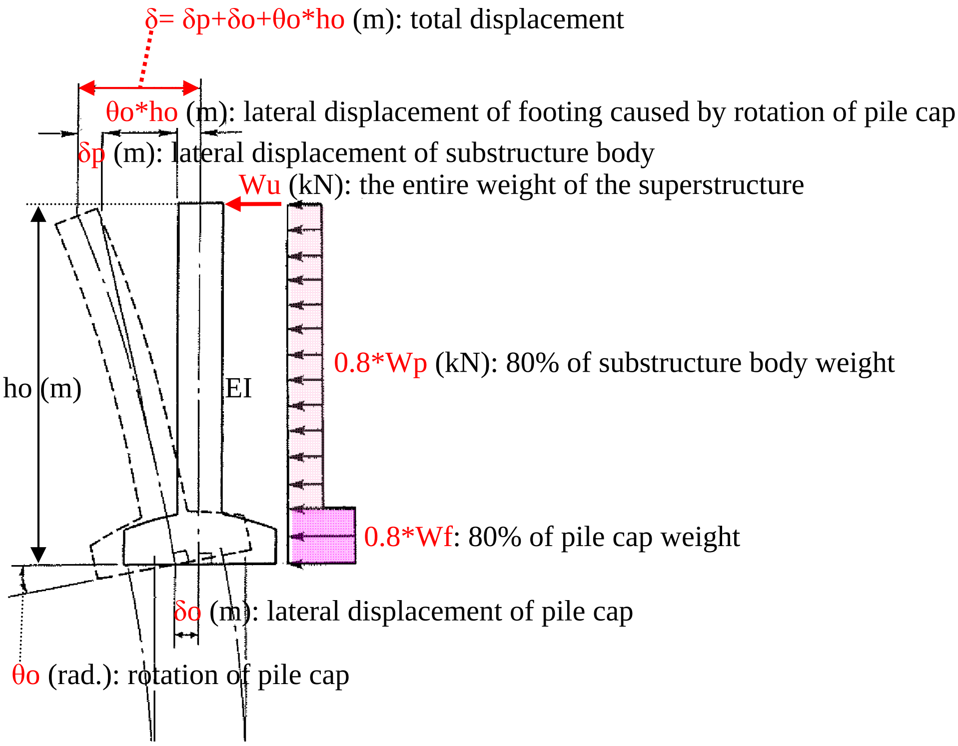


Figure 1.2.7-2 Displacement of the Substructure in the Longitudinal Direction

(2) Application Point of Seismic Inertial Force from Superstructure

In the design example, height from the top of the substructure body to application point of seismic inertial force from superstructure, “ho”, is defined as follows.

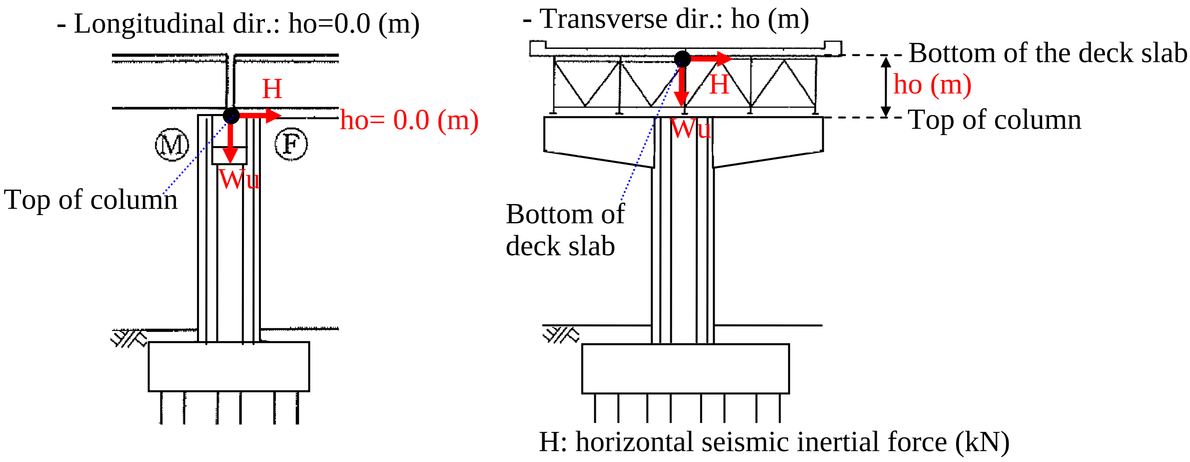


Figure 1.2.7-3 Application Point of Seismic Inertial Force of Superstructure

(3) Pier/Column Stiffness in Bridge Seismic Analysis BSDS (Commentary-C4.5.3)

The bridge shall be modeled to be consistent with the degrees-of-freedom chosen to represent the natural modes and frequencies of vibration. The stiffness of the elements of the model shall be defined to be consistent with the bridge being modeled.

In the design example, “a moment of inertia equal to one-half that of the uncracked section” is adopted as cracked section stiffness in bridge analysis for the consideration of nonlinear effects which decrease stiffness. The methodology derives from the following commentary of LRFD 2012.

> In seismic analysis, nonlinear effects which decrease stiffness, such as inelastic deformation and cracking, should be considered. Reinforced concrete columns and walls in Seismic Zones 2, 3, and 4 should be analyzed using cracked section properties. For this purpose, a moment of inertia equal to one-half that of the uncracked section may be used. (LRFD 2012, C4.7.1.3)

The image of cracked section stiffness is illustrated below.

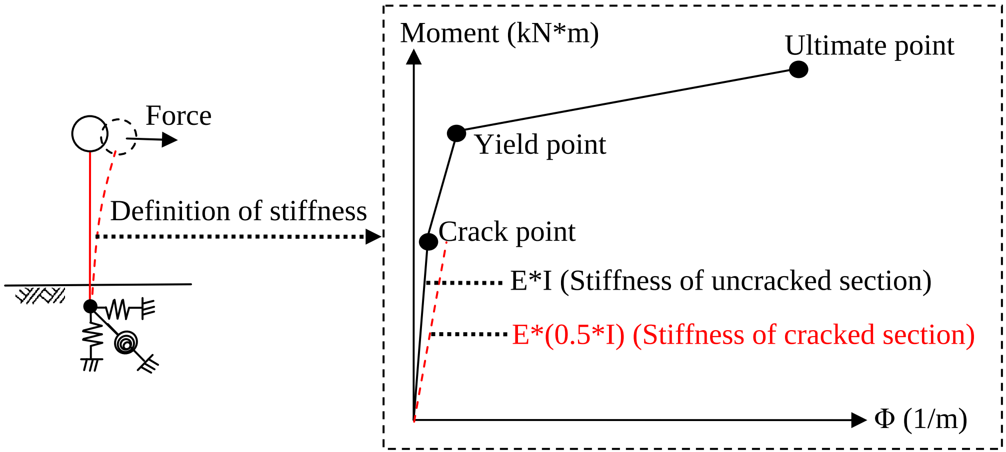


Figure 1.2.7-4 Image of Cracked Section Stiffness

1.2.7.5 Dynamic Spring Property of Pile Foundation BSDS (Article-4.4.3)

(1) Introduction to Modeling of Foundation Spring in Bridge Analysis

In order to obtain the vibration characteristics of structures, the consideration of the foundation spring in the analysis model is one of the most important issues for the realistic results. Although bridge analyses can be conducted with foundation springs fixed, the consideration of the realistic spring properties could contribute to the cost-effectiveness of the structure. As shown in the following figure, with the application of foundation spring, natural period of analysis model becomes longer, which results in the reduction of the response acceleration cost-effective design.

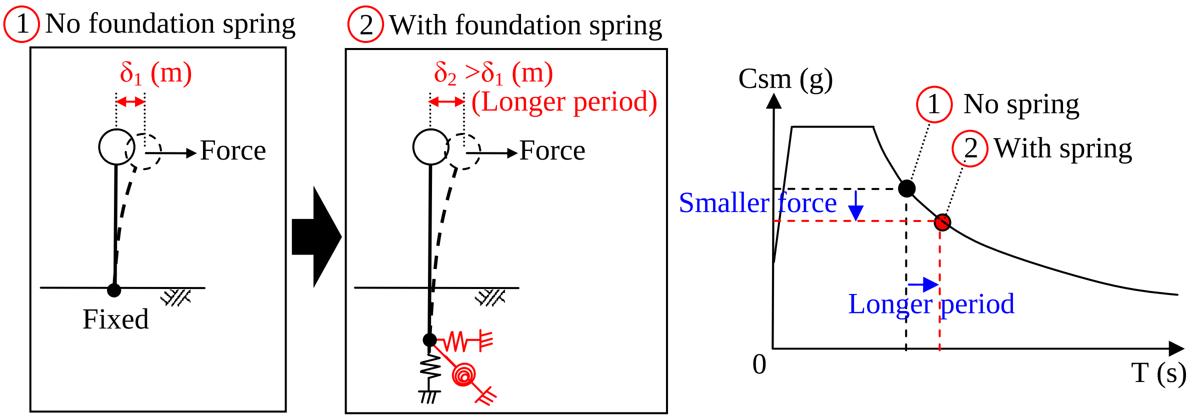


Figure 1.2.7-5 Effect of Foundation Spring on Seismic Designs

The foundation spring can be considered in analyses with the either type of models in the following figure. Both models are considered to generate the equivalent analysis results. The detail of the methodologies is explained from the next page.

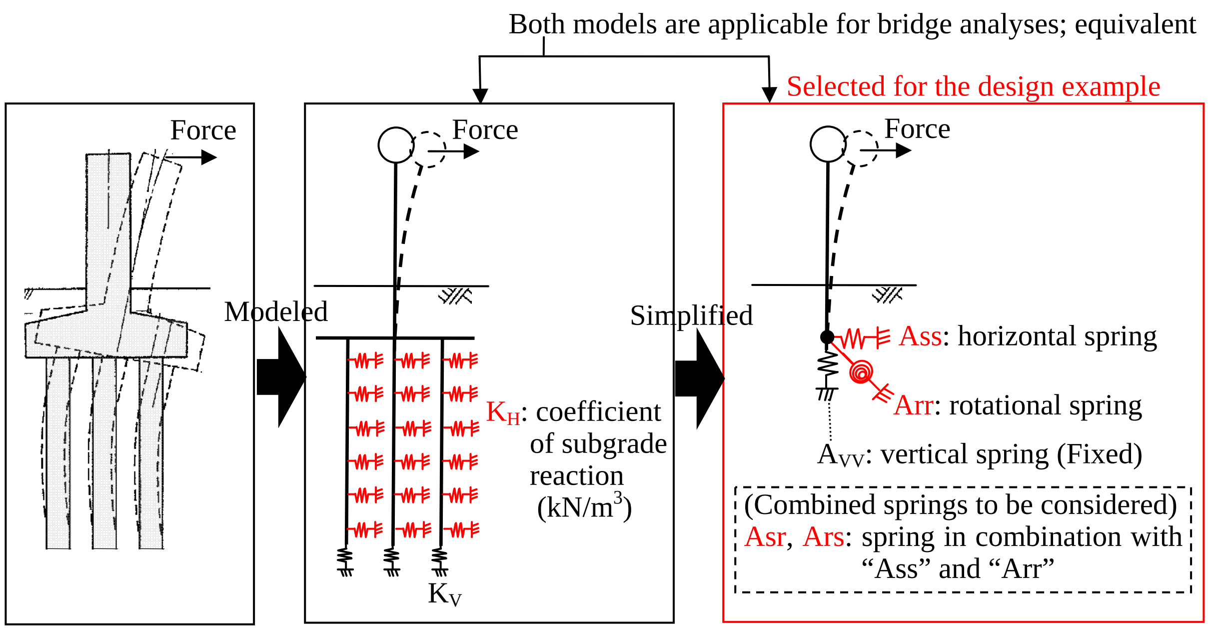


Figure 1.2.7-6 Modeling of Foundation Spring

(2) Coefficient of Subgrade Reaction, "K_H"

Coefficient of subgrade reaction, K_H, is defined with the following equation.

$$\boxed{K_H = K_{H0}\left(\frac{B_H}{0.3}\right)^{-3/4}}$$

Where,

K_{H0}: reference value of the coefficient of subgrade reaction in horizontal direction (kN/m3);

$$K_{H0} = \frac{E_D}{0.3}$$

B_H: equivalent loading width of foundation (m); $B_H = \sqrt{\frac{D}{\beta}}$

"1/β" implicates effective range of "K_H".

E_D: dynamic modulus of ground deformation (kN/m2); $E_D = 2*(1+\nu_D)*G_D$

G_D: dynamic shear modulus of ground deformation (kN/m2); $G_D = \frac{\gamma_t}{g}V_{SD}^2$

ν_D: dynamic Poisson's ratio of the ground (0.45 for soil above water, 0.5 for soil under water)

γt: unit weight of the ground (kN/m3)

g: acceleration of the gravity; g=9.8(ms2)

V_{SD}: shear elastic wave velocity of the ground (m/s); $V_{SD} = c_v * V_{si}$

c_v: modification factor based on degree of ground strain; $c_v \begin{cases} = 0.8\ (V_{si} < 300 m/s) \\ = 1.0\ (V_{si} \geq 300 m/s) \end{cases}$

Vsi: the average shear wave velocity of the i-th soil layer;

$$V_{si} \begin{cases} = 100 N_i^{1/3}\ (1 \leq N_i \leq 25) \\ \text{(for cohesive soil layers)} \\ = 80 N_i^{1/3} (1 \leq N_i \leq 50) \\ \text{(for sandy/cohesionless soil layers)} \end{cases}$$

If piles are modeled individually as shown in the following figure, "K_H" should be calculated with dynamic modulus of ground deformation, "E_D", of each layer.

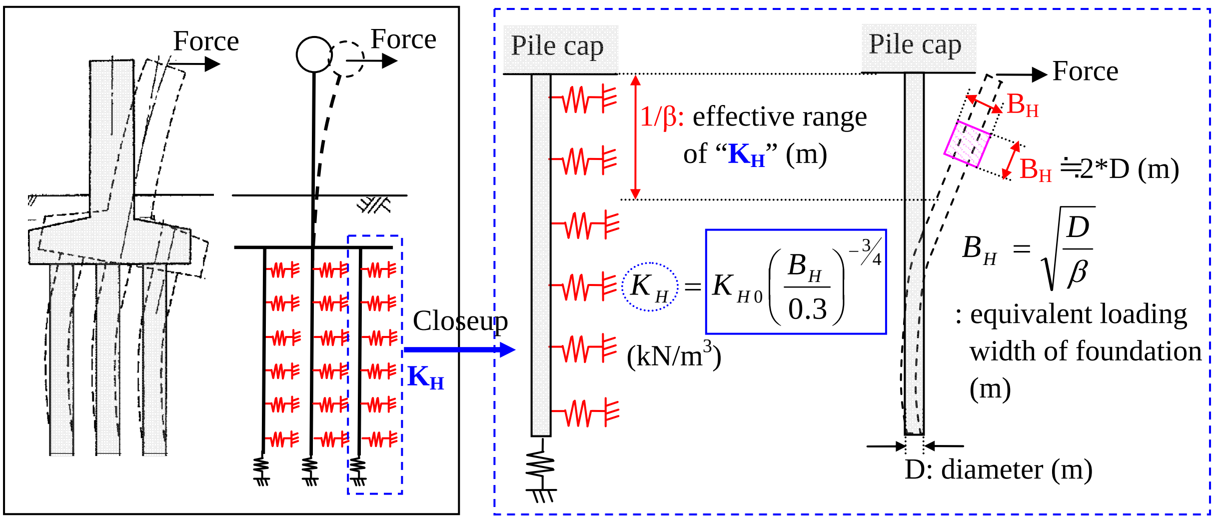


Figure 1.2.7-7 Coefficient of Subgrade Reaction, "K_H"

(3) Spring Properties of Foundation Structures

If the effect of foundation on analyses is focused on "1/β" range, which is the effective range of "K_H", foundation structure can be modeled as group of springs in one node as shown in the following figure. If this method is applied, "K_H" should be calculated with the average value of "E_D" in "1/β" range, "$(E_D)_\beta$".

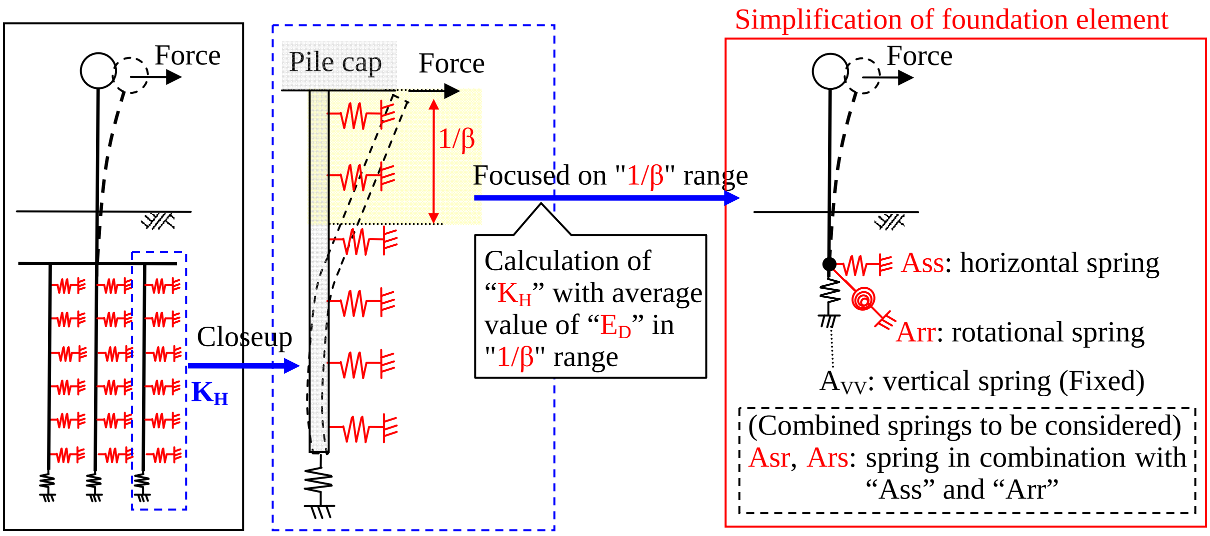


Figure 1.2.7-8 Coefficient of Subgrade Reaction, "K_H"

After the calculation of "K_H" with "$(E_D)_\beta$", β can be obtained with the following equation.

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

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= $4*E*I*\beta^3$ (kN/m)

K2= K3 = $2*E*I*\beta^2$ (kN/rad)

K4= $2*E*I*\beta$ (kN*m/rad)

Kv=a*Ap*E/L (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)

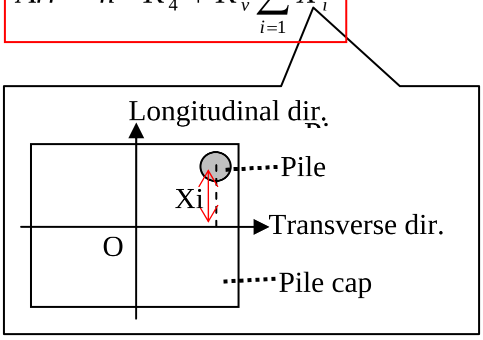
E: Young's modulus of elasticity of the pile (kN/mm2)

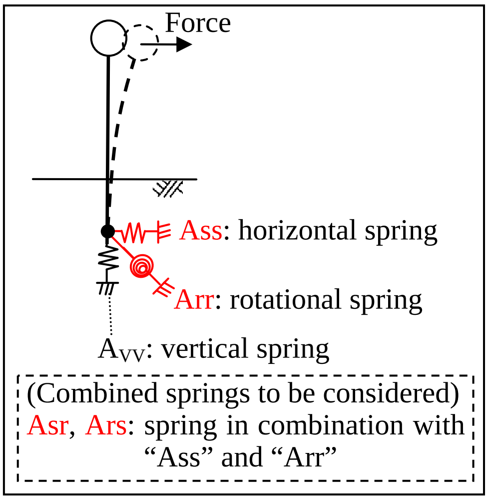
Finally, spring properties of entire pile foundation can be determined with the following equations.

$$Ass = n * K_1$$

$$Asr = Ars = -n * K_2$$

$$Arr = n * K_4 + K_v \sum_{i=1}^{n} X_i^2$$





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.

Step-1: Assumption of "$1/\beta_1$".
Generally, $4*D < (1/\beta_1) < 6*D$
Where, D: diameter of a pile

(Feed back)
Input $(1/\beta)$ into $(1/\beta_1)$ until $(1/\beta)$ becomes equal to $(1/\beta_1)$.

Step-2: Calculation of "B_H" with "$1/\beta_1$"

$$B_H = \sqrt{\frac{D}{\beta_1}}$$

Step-3: Calculation of "K_H"

$$K_H = K_{H0}\left(\frac{B_H}{0.3}\right)^{-3/4}$$

Where,

$$K_{H0} = \frac{(E_D)_\beta}{0.3}$$

$(E_D)_\beta$: average value of dynamic modulus of ground deformation within "1/β" range (kN/m2)

$$E_D = 2*(1+\nu_D)*G_D$$

$$G_D = \frac{\gamma_t}{g}V_{SD}^2$$

$$V_{SD} = c_v * V_{si}$$

$$c_v \begin{cases} = 0.8 \ (V_{si} < 300m/s) \\ = 1.0 \ (V_{si} \geq 300m/s) \end{cases}$$

$$V_{si} \begin{cases} = 100N_i^{1/3} \ (1 \leq N_i \leq 25) \\ \quad \text{(for cohesive soil layers)} \\ = 80N_i^{1/3} (1 \leq N_i \leq 50) \\ \quad \text{(for sandy soil layers)} \end{cases}$$

Step-4: Calculation of "β"

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

Compare value of $(1/\beta)$ with that of $(1/\beta_1)$.

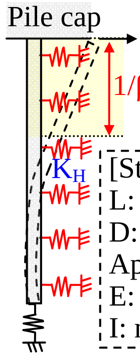
(Feed back)

Where,
E: Young's modulus of concrete piles (kN/m2)
I: moment of inertia of a pile (m4)

If $(1/\beta)$ becomes equal to $(1/\beta_1)$,

Step-5: Calculation of "K1, K2, K3, K4"
(Spring property of a pile)

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



[Structural properties of a pile]
L: length (m)
D: diameter (m)
Ap: cross-sectional area (m2)
E: Young's modulus (kN/mm2)
I: moment of inertia (m4)

Under the condition with "no battered pile"

Step-6: Calculation of "β"
(Spring property of entire pile foundation)

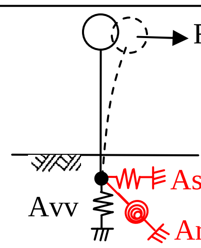
For pile foundation

$$Ass = n * K_1$$

$$Asr = Ars = -n * K_2$$

$$Arr = n * K_4 + K_v \sum_{i=1}^{n} X_i^2$$

Where, n: number of piles



Ass: horizontal spring
Arr: rotational spring

(Combined springs to be considered)
Asr, Ars: spring in combination with "Ass" and "Arr"

Figure 1.2.7-9 Determination Process of Dynamic Spring Property of Pile Foundation

1.2.8 Design Requirements and Applied Methodology

1.2.8.1 Combination of Seismic Force Effects BSDS (Article 5.2)

In the design example, combination of seismic force effects is considered in accordance with the following two provisions defined in BSDS.

1) The elastic seismic force effects on each of the principal axes of a component resulting from analyses in the two perpendicular directions shall be combined to form two load cases as follows:
   - 100 percent of the absolute value of the force effects in one of the perpendicular directions combined with 30 percent of the absolute value of the force effects in the second perpendicular direction, and
   - 100 percent of the absolute value of the force effects in the second perpendicular direction combined with 30 percent of the absolute value of the force effects in the first perpendicular direction.
2) Where foundation and/or column connection forces are determined from plastic hinging of the columns specified in Article 5.3.4.3, the resulting force effects may be determined without consideration of combined load cases specified herein.

1.2.8.2 Design Forces for Seismic Zone 3 and 4 BSDS (Article 5.3.4)

In the design example, design forces are determined in accordance with the following provision for Seismic Zone 3 and 4. Design force determination process of single column is explained in the design example.

1.2.8.3 Design Forces to be Applied BSDS (Article 5.3.4.1)

The design forces of each component shall be taken as the lesser of those determined using:

- (1) modified design forces (Article 5.3.4.2); or
- (2) inelastic hinging forces (Article 5.3.4.3),

for all components of a column, column bent and its foundation and connections.

1.2.8.4 Modified Design Forces BSDS (Article 5.3.4.2)

Modified design forces shall be determined as specified in Article 5.3.3, except that for foundations the R-factor shall be taken as 1.0.

1.2.8.5 Inelastic Hinging Forces BSDS (Article 5.3.4.3)

(1) Single Columns and Piers BSDS (Article 5.3.4.3.b)

1) Force effects shall be determined for the two principal axes of a column and in the weak direction of a pier or bent as follows:
   - Step-l: Determine the column overstrength moment resistance. Use a resistance factor, φ of 1.3 for reinforced concrete columns and 1.25 for structural steel columns. For both materials, the

applied axial load in the column shall be determined using Extreme Event Load Combination 1, with the maximum elastic column axial load from the seismic forces determined in accordance with Article 4.2.1 taken as EQ.

- Step-2: Using the column overstrength moment resistance, calculate the corresponding column shear force. For flared columns, this calculation shall be performed using the overstrength resistances at both the top and bottom of the flare in conjunction with the appropriate column height. If the foundation of a column is significantly below ground level, consideration should be given to the possibility of the plastic hinge forming above the foundation. If this can occur, the column length between plastic hinges shall be used to calculate the column shear force.

2) Force effects corresponding to a single column hinging shall be taken as:

- Axial Forces - Those determined using Extreme Event Load Combination 1, with the unreduced maximum and minimum seismic axial load of Article 4.2.1 taken as *EQ*.
- Moments - those calculated in Step 1.
- Shear Force - that calculated in Step 2.

a) Inelastic Hinging Forces (Pier Design Forces) **BSDS (Article 5.3.4.3.e)**

The design forces shall be those determined for Extreme Event Limit State Load Combination I, except where the pier is designed as a column in its weak direction. If the pier is designed as a column, the design forces in the weak direction shall be as specified in Article 4.2.2.4.3d and all the design requirements for columns, as specified in Section 5 (Concrete Structures) of the *AASHTO LRFD Bridge Design Specifications (2012)*, shall apply. When the forces due to plastic hinging are used in the weak direction, the combination of forces, specified in Article 4.2.1, shall be applied to determine the elastic moment which is then reduced by the appropriate R-factor.

b) Inelastic Hinging Forces (Foundation Design Forces) **BSDS (Article 5.3.4.3.f)**

The design forces for foundations including footings, pile caps and piles may be taken as either those forces determined for the Extreme Event Load Combination I, with the seismic loads combined as specified in Article 4.2.1, or the forces at the bottom of the columns corresponding to column plastic hinging as determined in Article 4.2.1.

1.2.8.6 Outline of Design Force Determination Process in the Design Example

To sum up the above requirements for design forces, design forces are determined as follows.

(1) Column Design Force

Step-1: Conduct bridges analysis and get resulted forces at column base in both longitudinal and transverse directions namely;

- M_L, M_T: bending moment at column base in longitudinal/transverse direction (kN*m)
- V_L, V_T: Shear force at column base in longitudinal/transverse direction (kN)

Step-2: Calculate the design forces considering the combination of seismic force effects inelastic hinging effect (R-factor) with the following equations.

- for longitudinal direction

$$M_{dL} = \sqrt{(1.0 * M_L)^2 + (0.3 * M_T)^2} / R \text{ : Design moment (kN*m)}$$

$$V_{dL} = \sqrt{(1.0 * V_L)^2 + (0.3 * V_T)^2} \text{ : Design shear force (kN)}$$

- for transverse direction

$$M_{dT} = \sqrt{(1.0 * M_T)^2 + (0.3 * M_L)^2} / R \text{ : Design moment (kN*m)}$$

$$V_{dT} = \sqrt{(1.0 * V_T)^2 + (0.3 * V_L)^2} \text{ : Design shear force (kN)}$$

Note: axial force at the column base is defined as,

Nd= Rd+Wp (kN)

Where,

Rd: reaction force of the pier (kN)

Wp: weight of pier column (kN)

The detail of the calculation process is illustrated in the following figure.

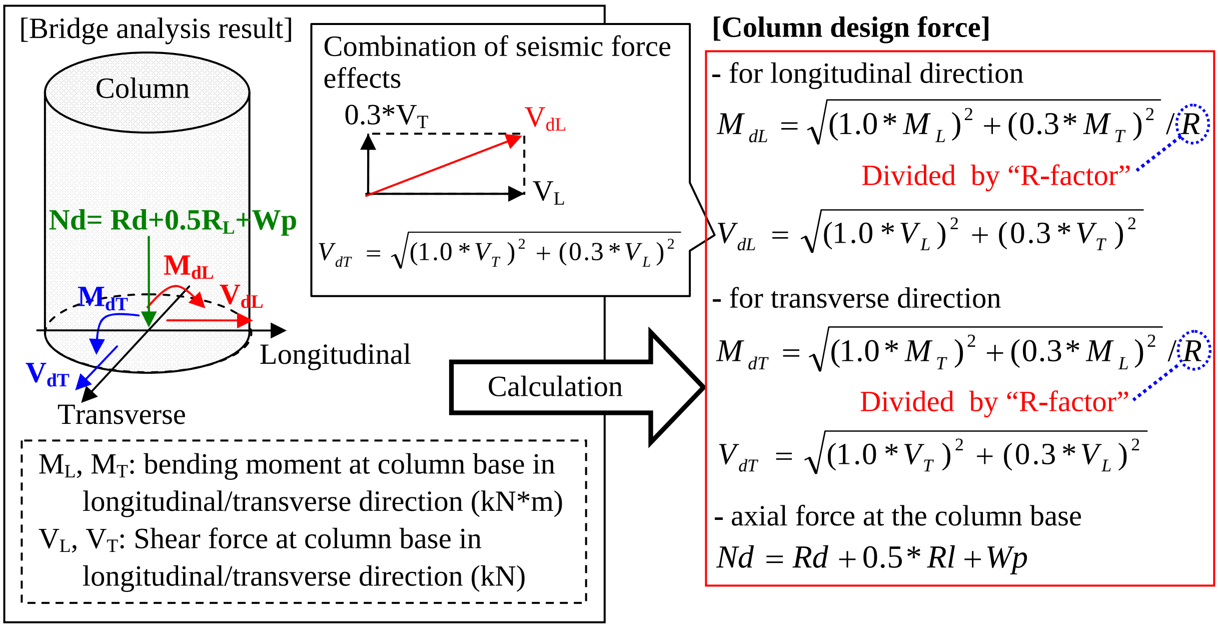


Figure 1.2.8-1 Determination of Column Design Force

(2) Foundation Design Force

Step-1: Create an axial force (N)-nominal moment (Mn) interaction diagram of the column and plot a dot on the curve where “N” is equal to design axial force, “Nd”. “ϕ*Mn” is regarded as “Mu” if the flexural resistance factor, ϕ is equal to 1.0.

$$Mu = 1.0 * Mn \ (\phi = 1.0)$$

Where,

Mu: ultimate bending moment strength of the column (kN*m)

Mn: nominal bending moment of the column (kN*m)

ϕ: flexural resistance factor of the column

Step-2: Determine the column overstrength moment resistance, "Mp", with the flexural resistance factor, ϕ of 1.3. "ϕ*Mn" is regarded as "Mp" with the flexural resistance factor, ϕ of 1.3;

$Mp = 1.3 * Mn\ (\phi = 1.3)$: the column overstrength moment resistance (kN*m)

Step-3: Using the column overstrength moment resistance, "Mp", calculate the corresponding column shear force, "Vp" at the column base with the appropriate column height;

$Vp = Mp / h$: the corresponding column shear force at column base (kN)

Where,

h: height from the application point of seismic inertial force from the superstructure to the column base (m)

Step-4: Add the effect of the footing weight to "the inelastic hinging forces" calculated above;

- for longitudinal/transverse direction

$Md = Mp + Mf = Mp + Ff * h_f / 2$: Design moment (kN*m)

$Vd = Vp + Ff = Vp + Wf * As$: Design shear force (kN)

Note: axial force at the footing base is defined as,

Nd'= Rd+Wp+Wf (kN)

Where,

Mf: moment caused by weight of the footing (kN*m)

Ff: seismic inertial force caused by weight of the footing (kN)

Wf: weight of pier column (kN)

hf: height of footing (m)

As: acceleration coefficient; As= Fpga*PGA (g)

The detail of the calculation process is illustrated in the following figure.

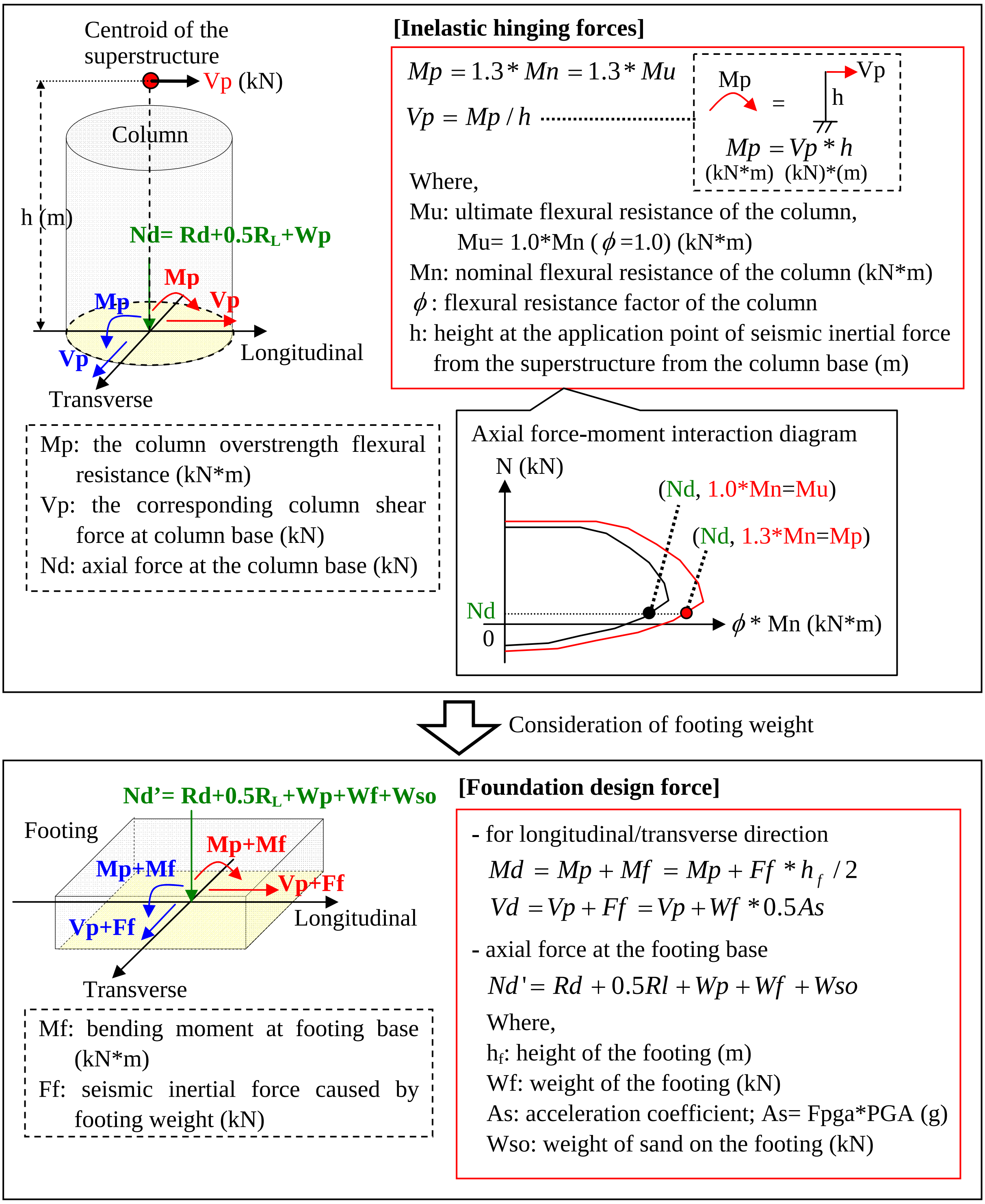


Figure 1.2.8-2 Determination of Foundation Design Force

1.2.8.7 Column Resistance LRFD (Section 5)

(1) Flexural Resistance

In the design example, flexural resistance of columns, "ϕ*Mn", is determined as follows.

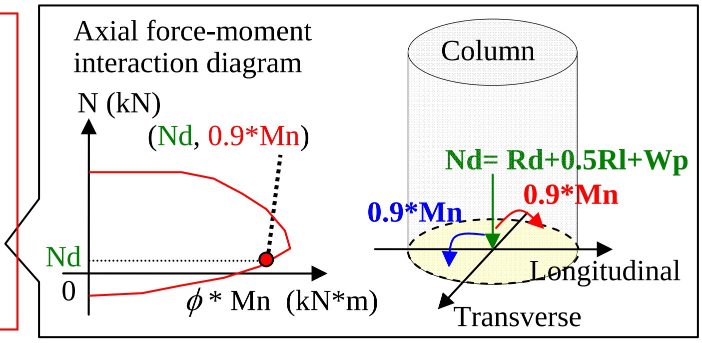
$$Mr = \phi * Mn$$

Where,

Mr: flexural resistance of the column (kN*m)

Mn: nominal flexural resistance of the column (kN*m)

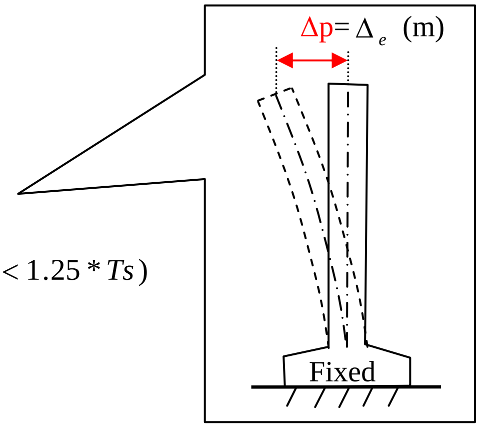
ϕ: flexural resistance factor of columns; ϕ = 0.9



(2) P-⊿ Requirements BSDS (Article 4.7)

The displacement of any column or pier in the longitudinal or transversal direction shall satisfy the following equation. As shown below, the equation can be interpreted as the requirement for maximum displacement verification.

$$\Delta * Pu < 0.25 * \varphi * Mn$$
$$\leftrightarrow \Delta < \frac{\varphi * Mn}{4 * Pu}$$



in which:

$\Delta = 12 * R_d * \Delta_e$

$$R_d \begin{cases} = \left(1 - \frac{1}{R}\right)\frac{1.25 * T_s}{T} + \frac{1}{R} & (T < 1.25 * Ts) \\ = 1 & (T \geq 1.25 * Ts) \end{cases}$$

Where,

Δ : Displacement of the point of contraflexure in the column or pier relative to the point of fixity for the foundation (m)

Δ_e : Displacement calculated from elastic seismic analysis (m)

T: Period of fundamental mode of vibration (sec.)

Ts: Corner period specified in BSDS Article 3.6.2 (sec.)

R: R-factor; R= 2.0

Pu: Axial load on column or pier (kN)

φ: Flexural resistance factor for column; φ= 0.9

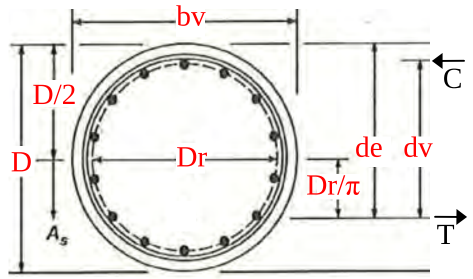
Mn: Nominal flexural strength of column or pier calculated at the axial load on the column or pier (kN*m)

(3) Shear Resistance

In the design example, shear resistance of columns, "ϕ*Vn", is determined with the following process.

Step-1: calculation of "Vn"
- Vn= Vc+Vs (kN)

Where,
Vn: nominal shear resistance (kN)
Vc: shear strength developed by concrete (kN);
Vc= 0.083*β*(f'c')$^{0.5}$*bv*dv
= 0 (to be on the conservative side)
Vs: shear strength developed by re-bars (kN);
Vs= (Av*fy*dv*cotθ)/s
β: factor indicating ability of diagonally cracked concrete to transmit tension and shear; β= 2.0
f'c': specified compressive strength of concrete (N/mm2)
bv: effective web width (mm)
dv: effective shear span (mm);
dv= 0.9*de= 0.9*(D/2+Dr/π)
fy: yield strength of transverse reinforcement (N/mm2)
θ: angle of inclination of diagonal compressive stress (degrees); θ= 45
s: spacing of transverse reinforcement (mm)



Simplified procedure is applied with the following values;
- β= 2.0
- θ= 45 (degrees)

Step-2: determination of "Vr"
- Vr: shear resistance of columns (kN);
Vr = ϕ*Vn

Where,
ϕ: shear resistance factor for columns; ϕ= 0.9

Figure 1.2.8-3 Determination Process of Shear Resistance of Columns

(4) Requirement for Minimum Reinforcement

[Minimum Required Longitudinal Reinforcement]

$\boxed{\rho_s \geq 0.0075}$ (for "seismic zone 3 & 4")

Where,
ρ_s: ratio of longitudinal reinforcement to gross area of concrete section; ρ_s= Aa/Ag
Aa : total area of longitudinal reinforcement (m2); Aa= As*n
As: cross-sectional area of single longitudinal reinforcement bar (m2)
n: number of longitudinal reinforcement bar
Ag: gross area of concrete section (m2)

[Minimum Required Transverse Reinforcement]

In the design example, minimum required transverse reinforcement is verified by the greater value of following two formulas.

Greater of $\boxed{\rho_s \geq 0.45 * \left(\frac{A_g}{A_c} - 1\right) * \frac{f_c'}{f_{yh}}}$ or $\boxed{\rho_s \geq 0.12 * \frac{f_c'}{f_{yh}}}$

Where,

ρ_s: ratio of transverse reinforcement to total volume of concrete core

Ag: gross area of concrete section (m2)

Ac: area of core measured to the outside diameter of transverse reinforcement (m2)

fc': specified concrete strength at 28 days (N/mm2)

fyh: yield strength of transverse reinforcement (N/mm2)

1.2.8.8 Minimum Seat Length Requirements BSDS (Article 4.6, 7.2)

Adequate measures against unseating of superstructures shall be taken when the superstructure separates structurally from the substructure, and with large relative displacements. Support lengths at expansion bearings without restrainers, shock transmission units (STUs) or dampers shall be designed to either accommodate the greater of the maximum calculated displacement (BSDS Article 4.1.3), except for bridges in Zone 1, or the empirical seating or support length, S_{EM}. I n the design example, design of seat length is explained, complying with the following equation of minimum seat length, S_{EM}.

$\boxed{S_{EM} = 0.7 + 0.005 * L}$

Where,

L: distance between two substructures for determining the seat length (m)

1.2.8.9 Definition of Ground Surface in Seismic Design BSDS (Article 3.5.2)

The ground surface to be considered in seismic design refers to the ground surface in which the design earthquake ground motion/forces specified in Article 3.6 is applied, with the assumption that the seismic forces acts only on the structures above it and excluding the structures existing below it.

Accordingly, the ground surface in the design example is defined as the following figure.

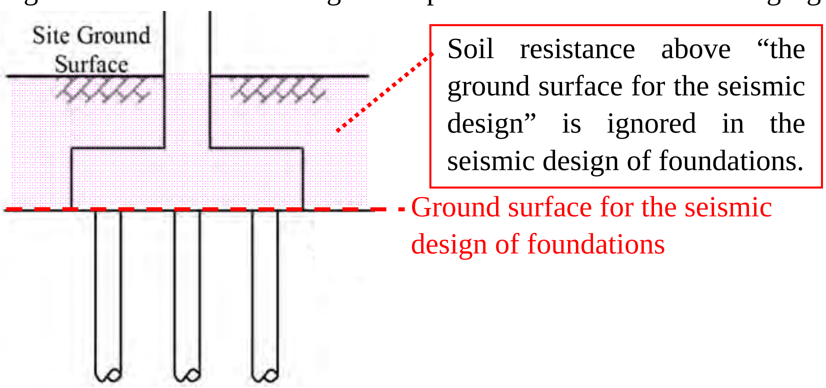
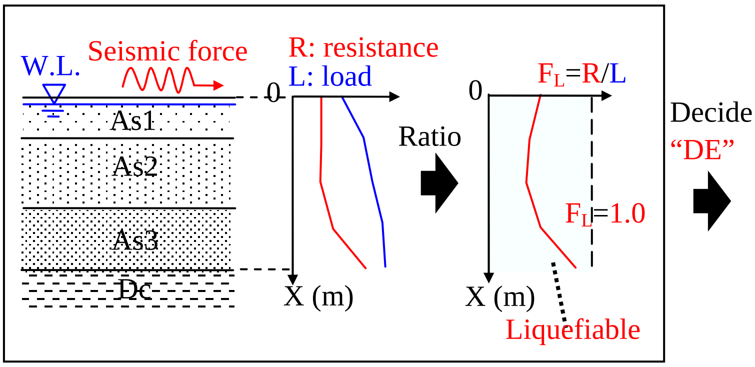


Figure 1.2.8-4 Definition of Ground Surface in Seismic Design

1.2.8.10 Assessment of Liquefaction Potential BSDS (Article 6.2.3 & 6.2.4)

(1) Application of FL method for assessment of liquefaction potential

In the design example, liquefaction potential is assessed with “FL Method”. In this method, possibility of liquefaction is assessed by the value of “F_L”, which is the ratio of dynamic shear strength ratio (resistance), “R”, to seismic shear stress ratio (load), “L”. As shown in the following figure, if “F_L” is 1.0 or less, the target layers are regarded as liquefiable. That means, liquefaction happens when seismic load in the ground, “L” is larger than ground resistance against liquefaction, “R”. As a result of the assessment, Soil coefficient reduction factor “DE” can be obtained. “DE” can be used in seismic design of pile foundation.



Soil coefficient reduction factor “DE”

F_L	X (m)	DE	
		R≦0.3	0.3<R
F_L≦1/3	0≦X≦10	0	1/6
	10<X≦20	1/3	1/3
1/3<F_L≦2/3	0≦X≦10	1/3	2/3
	10<X≦20	2/3	2/3
2/3<F_L≦1	0≦X≦10	2/3	1
	10<X≦20	1	1

Figure 1.2.8-5 FL Method for Assessment of Liquefaction Potential

(2) Three requirements to judge the need of liquefaction potential assessment

For alluvial sandy layers which have all of the following three conditions, liquefaction potential assessment shall be conducted. The detail is illustrated in the following figure.

1) Saturated soil layers which have ground water level higher than 10m below the ground surface and located at a depth less than 20m below the ground surface.
2) Soil layers which contains fine content (FC) of 35% or less, or soil layers which have plasticity index (Ip) less than 15, even if FC is greater than 35%.
3) Soil layers which have mean particle size (D_{50}) less than 10mm and particle size at 10% pass on the grading curve (D_{10}) is less than 1mm.

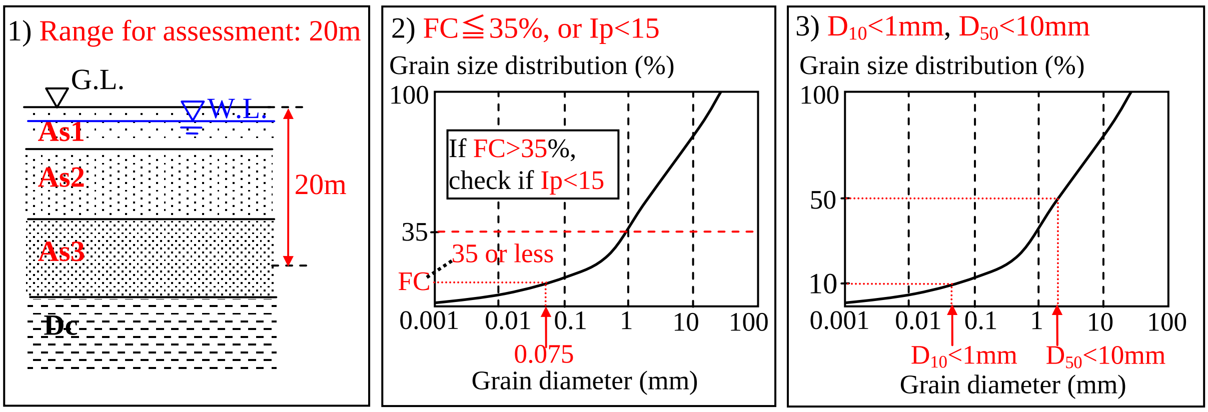


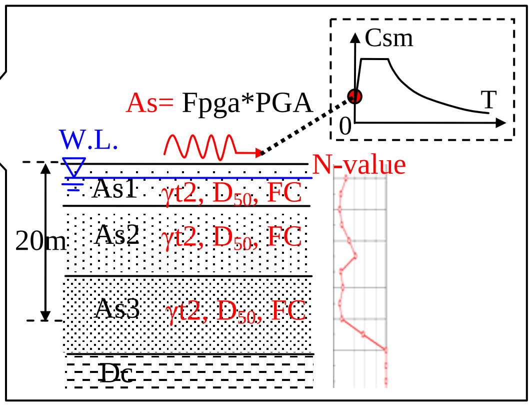
Figure 1.2.8-6 Three Requirements to Judge the Need of Liquefaction Potential

(3) Assessment liquefaction potential by "FL Method"

Liquefaction potential is assessed with the following methodological process in the design example.

Step-1: Determination of design parameters
- As=Fpga*PGA: Acceleration Coefficient (g)
- water level (m)

[Soil parameters of each layer]
- γt2: unit weight of soil below water level (kN/m3); γt2=γt+1
- γt1: unit weight of soil above water level (kN/m3)
- D_{50}: mean particle size (mm)
- FC: fine content (%)
- N-value (SPT result)



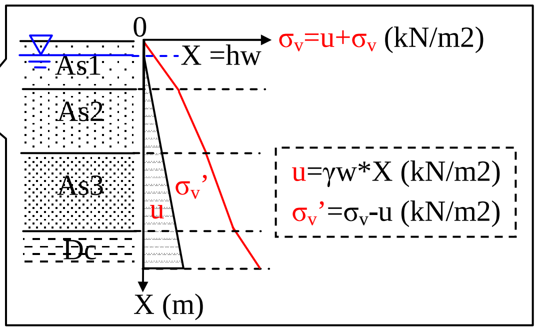
Step-2: Calculation of "σ_v"&"σ_v'"
- Total overburden pressure σ_v (kN/m2);

$$\sigma_v = \gamma_{t1} * h_W + \gamma_{t2} * (X - h_W)$$

- Effective overburden pressure σ_v' (kN/m2);

$$\sigma_v' = \sigma_v - u = \sigma_v - \gamma_w * X$$

Where,
γw: unit weight of water (kN/m3) ; γw =10
hw: depth at water level (m)



Step-3: Calculation of "Na"
(Na: modified N-value considering grain size effect)
- for sand layers

$$Na = C1 * N1 + C2$$

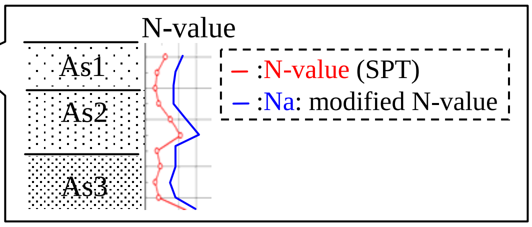
Where,
$N1 = 170 * N / (\sigma_v' + 70)$

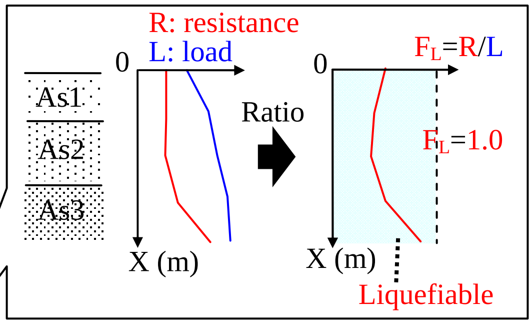
$$C1 = \begin{cases} 1 & (0 \leqq FC < 10) \\ (FC+40)/50 & (10 \leqq FC < 60) \\ FC/20-1 & (60 \leqq FC) \end{cases}$$

$$C2 = \begin{cases} 0 & (0 \leqq FC < 10) \\ (FC-10)/18 & (10 \leqq FC) \end{cases}$$

- for gravel layers

$$Na = \{1 - 0.36 \log_{10}(D_{50}/2)\} N1$$





Step-4: Calculation of "F_L"

$\boxed{F_L = R/L}$ → If $F_L<1$, liquefiable. Determination of "DE"

Where,
R: dynamic shear strength ratio (resistance);

$$R = \begin{cases} 0.0882\sqrt{Na/1.7} & (Na < 14) \\ 0.0882\sqrt{Na/1.7} + 1.6*10^{-6}(Na-14)^{4.5} & (14 \le Na) \end{cases}$$

L: seismic shear stress ratio (load);

$$L = (1 - 0.015 * X) * As * \sigma_v / \sigma_v'$$

X: depth from ground surface (m)

Soil coefficient reduction factor "DE"

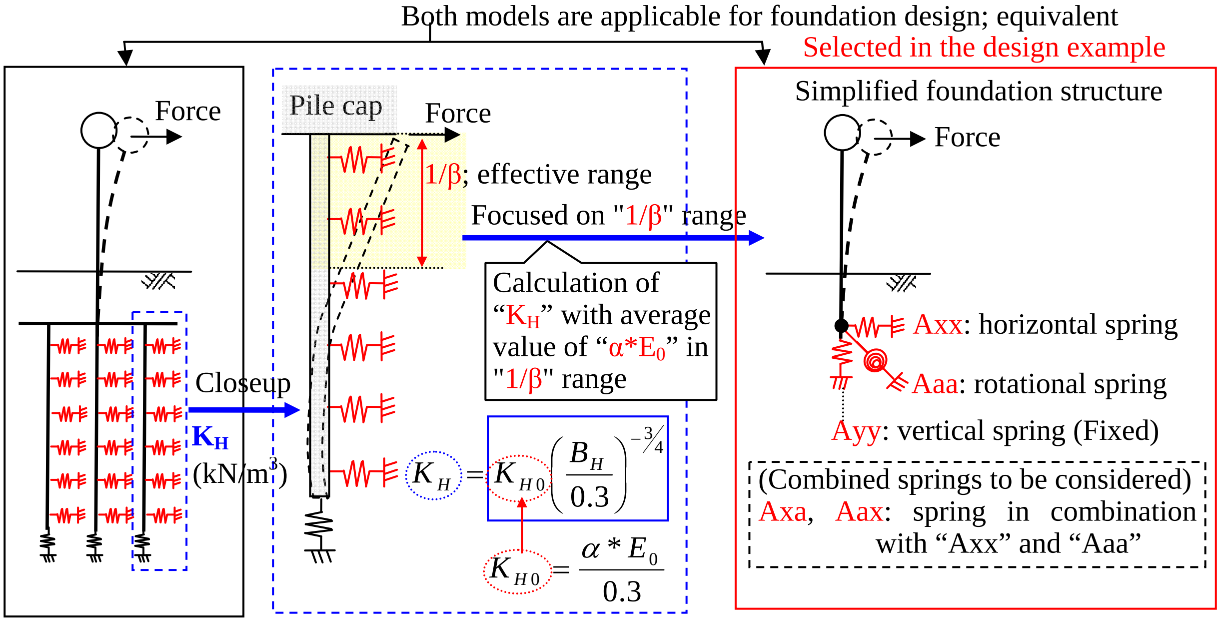
F_L	X (m)	DE R≦0.3	DE 0.3<R
$F_L \leqq 1/3$	0≦X≦10	0	1/6
	10<X≦20	1/3	1/3
$1/3 < F_L \leqq 2/3$	0≦X≦10	1/3	2/3
	10<X≦20	2/3	2/3
$2/3 < F_L \leqq 1$	0≦X≦10	2/3	1
	10<X≦20	1	1

Figure 1.2.8-7 Assessment Process of Liquefaction Potential by "FL Method"

1.2.8.11 Seismic Design Methodology of Pile Foundation   BSDS (Article 5.4.3)

(1) Modeling of Pile Foundation Structures

As already explained in the analysis methodology section, foundation structures can be modeled by either of types in the following figure. Simplified model is applied in the design example.



$\alpha * E_0$: modulus of deformation (kN/m2)

Figure 1.2.8-8 Modeling for Foundation Design

(2) Coefficient of Subgrade Reaction, "K_H" for Foundation Design

Coefficient of subgrade reaction, "K_H", is defined with the following equation. The determination process of spring properties for foundation design is almost same as that for bridge analysis except that "E_D" should be replaced with "$\alpha * E_0$" in the calculation formula of "K_{H0}".

$$K_H = K_{H0}\left(\frac{B_H}{0.3}\right)^{-3/4}$$

For simplified model focused on "1/β" range, "K_H" should be calculated with the average value of "$\alpha * E_0$" in "1/β" range, "$(\alpha * E_0)_\beta$"

Where,

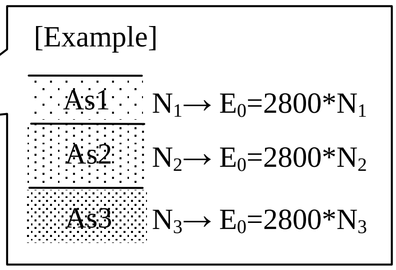
K_{H0}: reference value of the coefficient of subgrade reaction in horizontal direction (kN/m3);

$$K_{H0} = \frac{\alpha * E_0}{0.3} \quad \text{(for bridge analysis, } K_{H0} = \frac{E_D}{0.3}\text{)}$$

α: coefficient for the estimation of subgrade reaction coefficient;

$$\alpha \begin{cases} = 1 \text{ (Normal state)} \\ = 2 \text{ (Under earthquake)} \end{cases}$$

E_0: modulus of deformation (kN/m2); E_0= 2800*(N-value)



B_H: equivalent loading width of foundation (m);

$$B_H = \sqrt{\frac{D}{\beta}}$$

"1/β" implicates effective range of "K_H".

(3) Spring Properties of Foundation Structures

Spring properties of pile foundation structures are determined with the following process.

Step-1: Assumption of "$1/\beta_1$".
Generally, $4*D < (1/\beta_1) < 6*D$
Where, D: diameter of a pile

(Feed back)
Input $(1/\beta)$ into $(1/\beta_1)$ until $(1/\beta)$ becomes equal to $(1/\beta_1)$.

Step-2: Calculation of "B_H" with "$1/\beta_1$"

$$B_H = \sqrt{\frac{D}{\beta_1}}$$

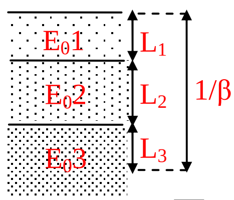
Step-3: Calculation of "K_H"

$$K_H = K_{H0}\left(\frac{B_H}{0.3}\right)^{-3/4}$$

Where,

$$K_{H0} = \frac{(\alpha * E_0)_\beta}{0.3}$$

$(\alpha * E_0)_\beta$: average value of dynamic modulus of ground deformation within "1/β" range



$$(\alpha * E_0)_\beta = \frac{\sum(\alpha * E_0 i * Li)}{1/\beta}$$

Step-4: Calculation of "β"

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

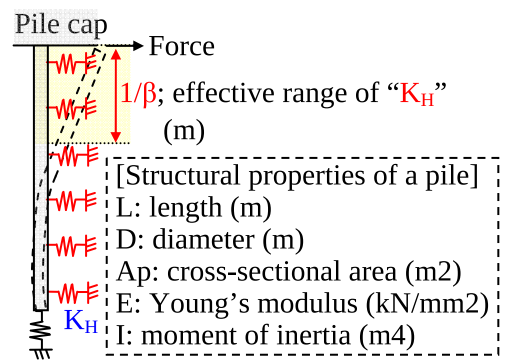
→ Compare value of $(1/\beta)$ with that of $(1/\beta_1)$. (Feed back)

Where,
E: Young's modulus of concrete piles (kN/m2)
I: moment of inertia of a pile (m4)

If $(1/\beta)$ becomes equal to $(1/\beta_1)$,

Step-5: Calculation of "K1, K2, K3, K4"
(Spring property of a pile)

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

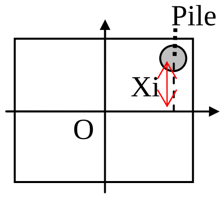


Under the condition with "no battered pile"

Step-6: Calculation of "β"
(Spring property of entire pile foundation)
For pile foundation

$$Axx = n * K_1$$
$$Axa = Ars = -n * K_2$$
$$Aaa = n * K_4 + K_v \sum_{i=1}^{n} X_i^2$$
$$Ayy = n * K_V$$

Where, n: number of piles



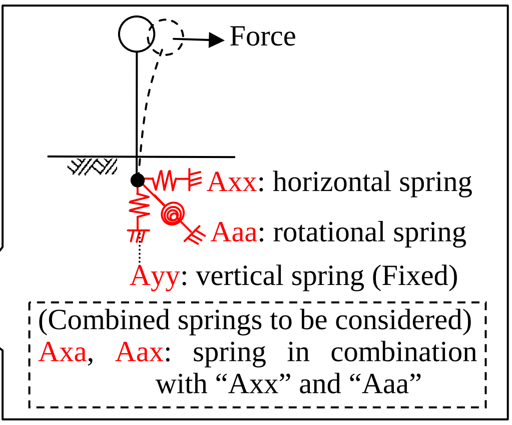


Figure 1.2.8-9 Determination Process of Spring Property of Pile Foundation

(4) Reaction Force and Displacement of Each Pile

Reaction force and displacement of each pile can be obtained with the following procedure.

[The relationship among displacements, foundation stiffness and design forces]

Axx*δx+Axy*δy+Axa*α = Vd

Ayx*δx+Ayy*δy+Aya*α = Nd

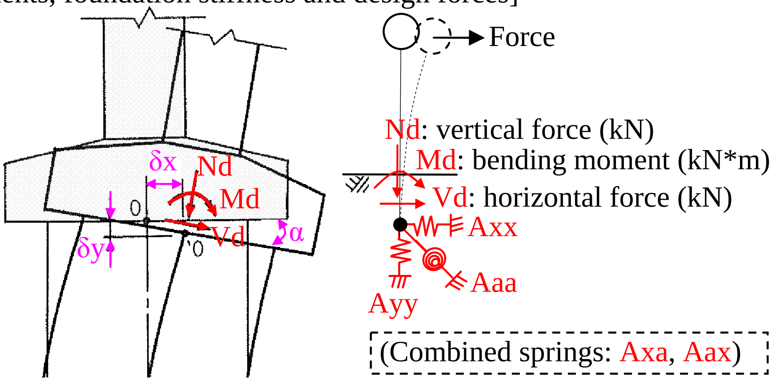
Aax*δx+Aay*δy+Aaa*α = Md

[Unknown parameters]

δx: lateral displacement (m)

δy: vertical displacement (m)

α: rotational angle (degrees)



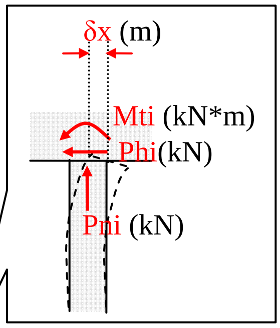
The solution of the above equations

Step-1: calculation of "displacement and rotation" of footing

$$\delta_x = \frac{V_d * A_{aa} - M_d * A_{xa}}{A_{xx} * A_{aa} - A_{xa} * A_{ax}}$$: lateral displacement at the origin "O" (m)

$$\delta_y = \frac{N_d}{A_{yy}}$$: vertical displacement at the origin "O" (m)

$$\alpha = \frac{-V_d * A_{ax} + M_d * A_{xx}}{A_{xx} * A_{aa} - A_{xa} * A_{ax}}$$: rotational angle of the footing (degrees)



Step-2: calculation of "reaction force and displacement" at each pile head

Pni= Kv*(δy+α*Xi): axial force acting on "pile head in i-th row" (kN)

Phi= K1*δx - K2*α: horizontal force acting on "pile head in i-th row" (kN)

Mti= -K3*δx + K4*α: bending moment acting on "pile head in i-th row" (kN*m)

Step-3: graphing of "reaction force and displacement" of each pile

1) Rigid-pile-head connection

$$y = \frac{P_{hi}}{2 * E * I * \beta^3} e^{-\beta * X} \{(1 + \beta * h_0) * \cos(\beta * X) - \beta * h_0 * \sin(\beta * X)\}$$

: horizontal displacement of single pile (m)

$$M = \frac{-P_{hi}}{\beta} e^{-\beta * X} \{\beta * h_0 * \cos(\beta * X) - (1 + \beta * h_0) * \sin(\beta * X)\}$$

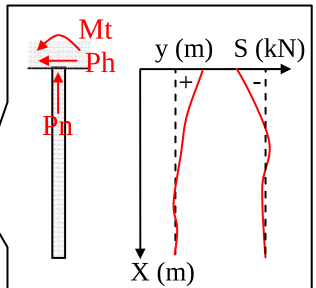
: bending moment of single pile (kN*m)

$$S = -P_{hi} * e^{-\beta * X} \{\cos(\beta * X) - (1 + 2 * \beta * h_0) * \sin(\beta * X)\}$$

: shear force of single pile (kN)

2) Hinge-pile-head connection

$$M = \frac{P_{hi}}{\beta} e^{-\beta * X} \sin(\beta * X)$$: bending moment of a pile (kN*m)



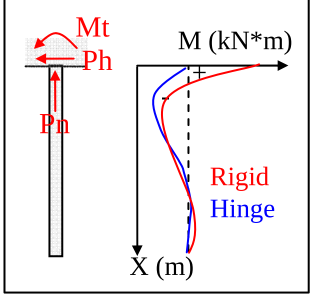
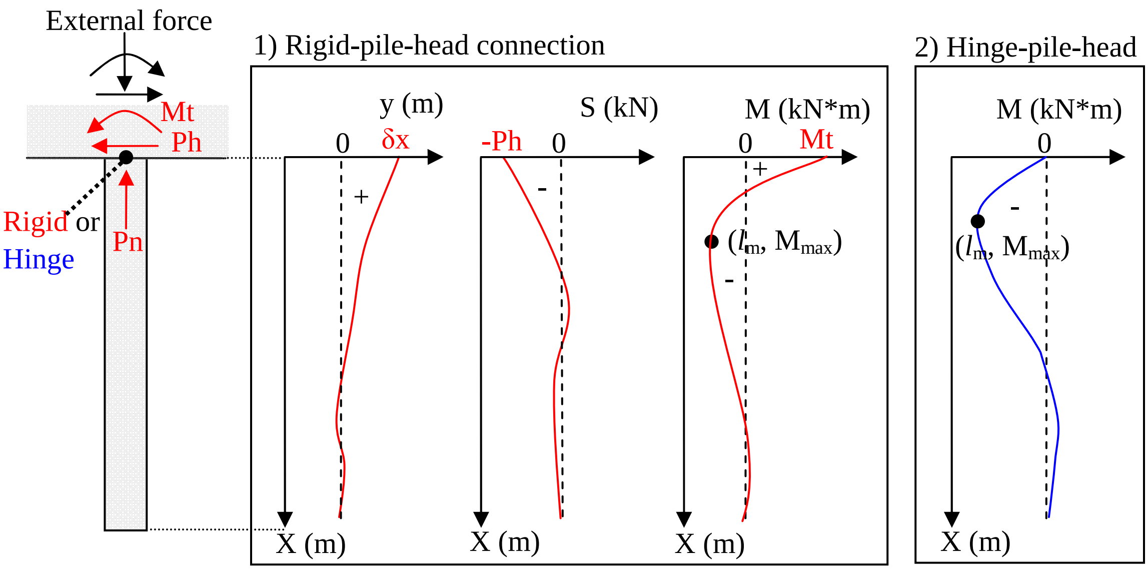


Figure 1.2.8-10 Reaction Force and Displacement of Each Pile

The detail of graphing of “reaction force and displacement” of each pile is illustrated below.



Maximum moment under the ground (rigid-pile-head connection)

$$M_{\max} = \frac{-P_{hi}}{2*\beta}\sqrt{(1+2*\beta*h_0)^2+1}*e^{-\beta*lm}$$

$$l_m = \frac{1}{\beta}\tan^{-1}\left(\frac{1}{1+2*\beta*h_0}\right)$$

$$h_0 = M_t / P_{hi}$$

: converted pile projection length (m)

Maximum moment under the ground (hinge-pile-head connection)

$$M_{\max} = \frac{-P_{hi}}{\beta}e^{-\beta*lm}*\sin(\beta*X)$$

$$l_m = \frac{\pi}{4*\beta}$$

Figure 1.2.8-11 Reaction Force and Displacement of Each Pile

(5) Verification of Reaction Force and Displacement of Each Pile

In the design of pile foundation, the following four items shall be verified.

1) Verification of Horizontal Displacement

The horizontal displacement at the pile head, δx, should be limited within 15 to less than 20mm;

δx≦15 to less than 20 (mm)

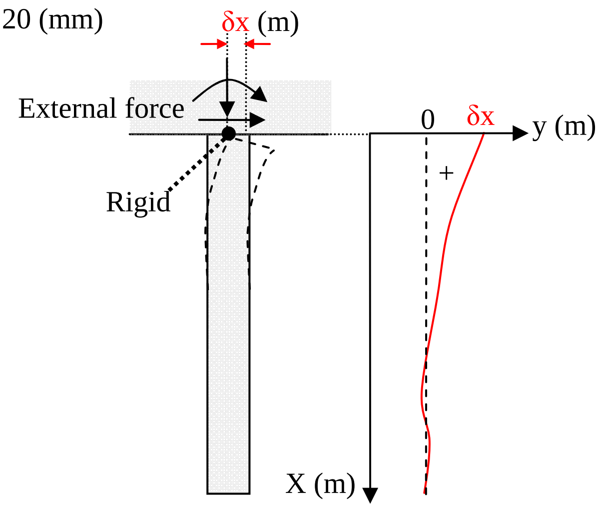


Figure 1.2.8-12 Verification of Horizontal Displacement

2) Verification of Bearing Capacity and Capacity against Axial Pull-out Force

The bearing capacity and capacity against axial pull-out force shall be verified with the following equations.

- verification of bearing capacity

$(P_{ni})_{\max} < R_R$

- verification of capacity against axial pull-out force

$(P_{ni})_{\min} < P_R$

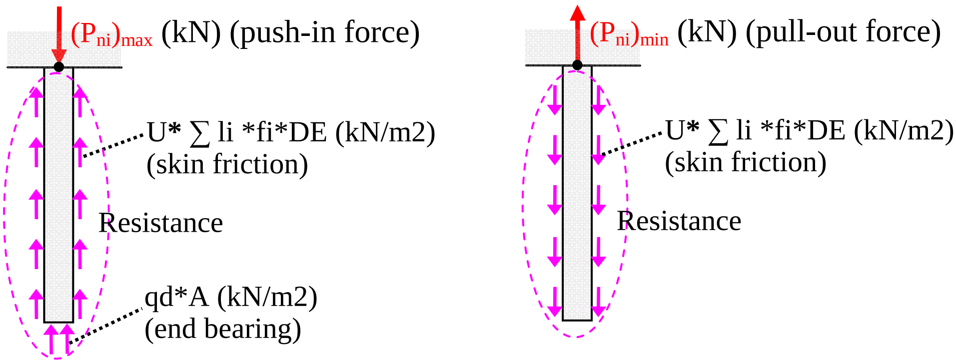


Figure 1.2.8-13 Bearing Capacity and Capacity against Axial Pull-out Force

Where,

[Bering capacity]

$(P_{ni})_{max}$: maximum axial load at the pile head (kN)

R_R: bearing capacity against axial force of single pile (kN); $R_R = \gamma*(\phi*Rn-Ws)+Ws-W$

γ: modification coefficient depending on nominal bearing resistance estimation method; γ=1.0

ϕ: resistance factor for bearing capacity under extreme event limit state; $\phi = 0.65$

Rn: nominal resistance of pile (kN); Rn= qd*A+U* ∑ li *fi*DE

qd: ultimate end bearing capacity intensity (kN/m2)

A: sectional area of a pile (m2); A= $(\pi*D^2)/4$

D: pile diameter (m)

U: perimeter of the pile (m); U= π*D

li: thickness of i-th layer considered for the bearing capacity (m)

fi: skin friction of i-th layer (kN/m2)

DE: reduction factor of i-th layer's soil parameters considering liquefaction effects

Ws: weight of soil inside the pile (kN)

W: effective weight of the pile and soil inside it; W= (γc - γw)*A*L

[Capacity against axial pull-out force]

$(P_{ni})_{min}$: maximum axial pull-out force at the pile head (kN)

P_R: factored axial pull-out resistance of single pile (kN); $P_R = -\phi*Pn+W$

ϕ: resistance factor for Capacity against axial pull-out force under extreme event limit state; $\phi = 0.50$

Pn: nominal axial pull-out resistance of single pile (kN); Pn= U* ∑ li *fi*DE

3) Verification of Flexural Capacity LRFD (Section 5)

Both "bending moment at the pile head" and "bending moment of pile under the ground" shall be limited to less than or equal to the flexural of piles, 0.9*Mn;

$Max(Mt, M_{max}) \leqq 0.9*Mn$ (kN*m)

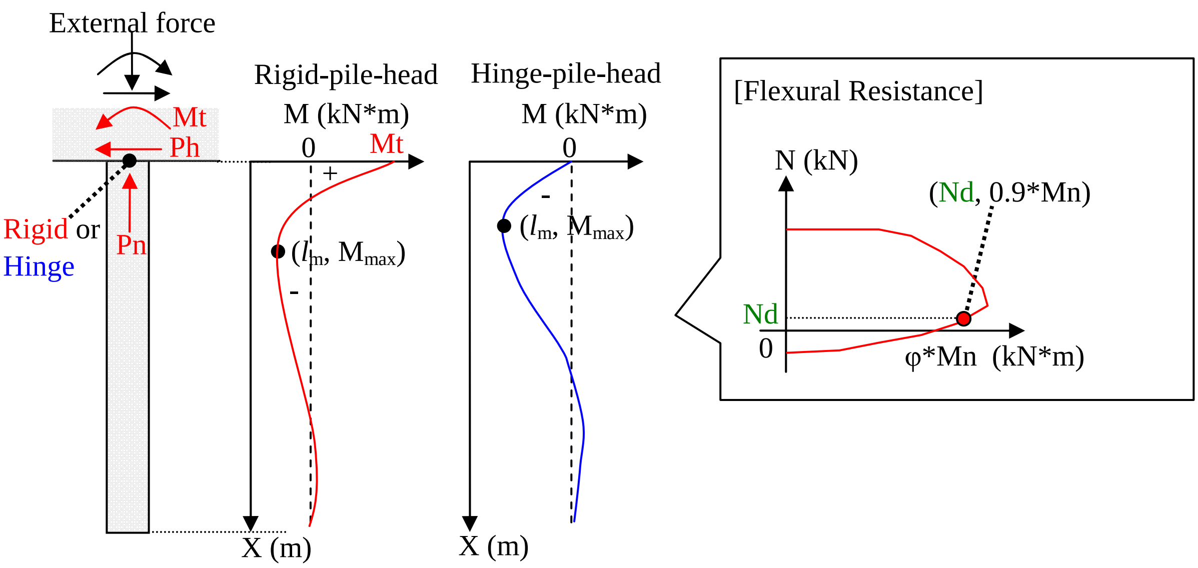


Figure 1.2.8-14 Verification of Flexural Resistance of Single Pile

4) Verification of Shear Force LRFD (Section 5)

Shear force at the pile head shall be limited to less than or equal to shear resistance of piles, "Vr". The shear resistance is determined with the following process (see "column resistance" for the detail).

Step-1: calculation of "Vn"
- Vn= Vc+Vs (kN)

Where,
Vn: nominal shear resistance (kN)
Vc: shear strength developed by concrete (kN);
  $Vc = 0.083*\beta*(f'c')^{0.5}*bv*dv$
  $= 0$ (to be on the conservative side)
Vs: shear strength developed by rebars (kN);
  $Vs = (Av*fy*dv*\cot\theta)/s$

Step-2: determination of "Vr"
- Vu: shear resistance of columns (kN);
  $Vr = 0.9*Vn$

Figure 1.2.8-15 Determination Process of Shear Resistance of Single Pile

5) Requirement for Minimum Reinforcement Verification LRFD (Section 5)

The following two equations are applied for the minimum reinforcement verification of longitudinal and transverse reinforcement.

[Verification formula for minimum required longitudinal reinforcement]

- Within confinement length (potential plastic hinging region)

$\boxed{\rho_s \geq 0.0075}$ (for “seismic zone 3 & 4”)

Note: confinement length extends from the underside of the pile cap over a length of not less than 2.0 pile diameters or 0.6 (m).

Where,

ρ_s: ratio of longitudinal reinforcement to gross area of concrete section; ρ_s=Aa/Ag

Aa : total area of longitudinal reinforcement (m2); Aa= As*n

As: cross-sectional area of single longitudinal reinforcement bar (m2)

n: number of longitudinal reinforcement bar

Ag: gross area of concrete section (m2)

- Below confinement length

$\boxed{\rho_s \geq 0.005}$

Note: longitudinal reinforcement shall be provided in the upper end of the pile for 2.5 (m) with a minimum steel ratio of 0.005 provided by at least four bars.

[Verification formula for minimum required transverse reinforcement]

$$\boxed{\rho_s \geq 0.12 * \frac{f_c^{'}}{f_{yh}}}$$

Where,

ρ_s: ratio of transverse reinforcement to total volume of concrete core

fc’: specified concrete strength at 28 days (N/mm2)

fyh: yield strength of transverse reinforcement (N/mm2)

2. Seismic Design Example of A Pier (Simply-supported Bridge)

2.1 Outline of the Design

Seismic design of a pier is conducted based on ‘Force-based Design Method’ with liquefiable ground condition. From the next page, overall design and calculation of the pier is explained along with the following process.

1. Determination of Structural Conditions
2. Determination of Load Condition
3. Determination of Ground Condition
4. Determination of Design Acceleration Response Spectra
5. Confirmation of Seismic Performance Zone
   (Confirmation of Design Requirements)
6. Determination of Bridge Importance Category
   (Determination of Response Modification factor)
7. Assessment of Liquefaction Potential
8. Initial Bridge Analysis and Column Design
   - Initial Bridge Analysis without Foundation Spring
   - Substructure Design Based on Initial Analysis Result
9. Bridge Analysis with Foundation Spring
   (Determination of Design Forces)
10. Determination of Design Displacement and
    Determination of Design Seat Length
11. Seismic Design of Column
12. Seismic Design of Foundation

Figure 2.1-1 Outline of Seismic Design of a Pier

2.2 Design Conditions

2.2.1 Structural Conditions

Structural conditions for the design are as follows.

- Bridge type: simply-supported composite steel I-shaped girder bridge
- Span length: 30.0 (m)
- Total road width: 10.5 (m)
- Skew angle: 90 degrees (non-skewed straight bridge)
- Pier type: single circular column
- Pier height: 11.9 (m) (column height: 10.0 (m))
- Foundation type: cast-in-place pile foundation
- Centroid of the superstructure: 2.0m from the column top

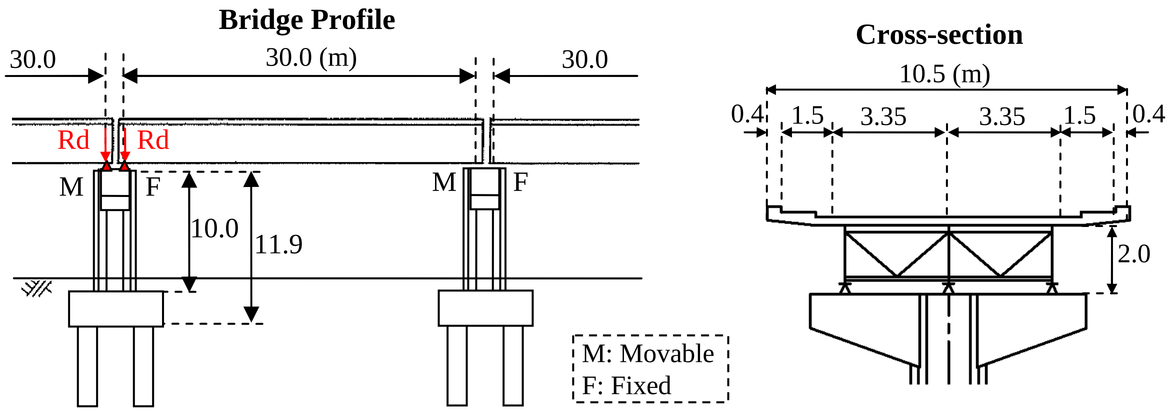


Figure 2.2.1-1 Bridge Profile and Superstructure Cross-section

2.2.2 Load Condition

(1) Reaction Force

Reaction forces for the design are as follows.

- Reaction force caused by “dead load”: Rd= 2900 (kN/pier)
- Reaction force caused by “live load”: Rl= 1800 (kN/pier)

Reaction force caused by dead load of the superstructure is determined by the following calculation.

- Bridge type: simply-supported I-shaped steel girder bridge
- Span length: L = 30.0 (m)
- Total road width W = 10.5 (m)

Superstructure	Unit load, q		Length, *l*		Member		No. of		Line load,
Railing	0.6	kN/m	-	num			2	nos	1.2
Asphalt Pavement	22.5	kN/m3	10.5	m	0.10	m	1	nos	23.6
Deck Slub	24.0	kN/m3	10.5	m	0.21	m	1	nos	52.9
Steel Members	170	kg/m2	10.5	m			1	nos	17.9

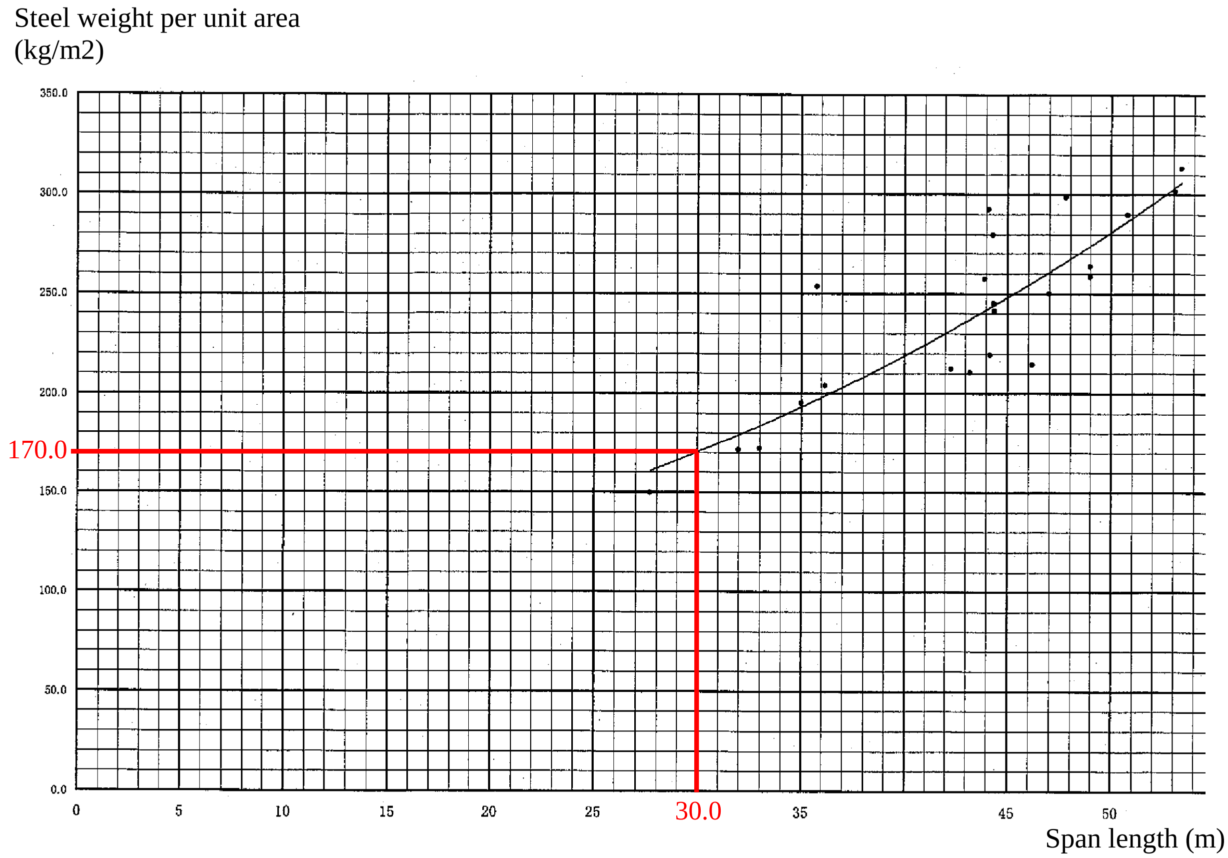
Sum: qt = 95.6 (kN/m)

Total load at the pier = qt*L = 2900 (kN); rounded up for modification

Reaction force, Rd = qt*L/2 = 1450 (kN)

Note: Rd=(Total load)/(No. of bearig lines at the pier)

Unit weight of the superstructure is assumed with the following graph.



Source: Guideline for Basic Planning of Steel Superstructures (Japan Bridge Association)

Figure 2.2.2-1 Relationship between Steel Weight of Composite Steel I-shaped Girder Per Unit Area and Bridge Span Length

The image of dead load condition is illustrated below.

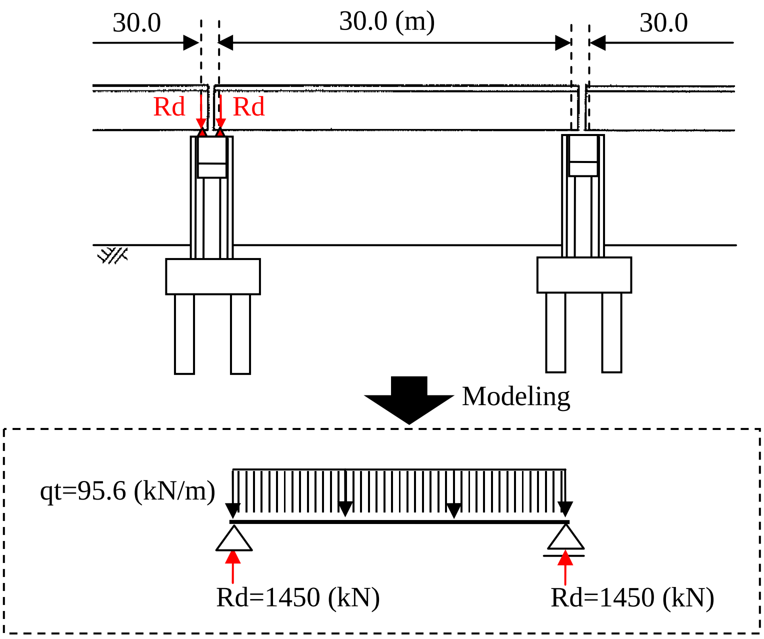


Figure 2.2.2-2 Image of Load Condition

(2) Dead Load of the Pier

The dead loads of each pier component are calculated as follows.

1) Coping

W1= (1.0+2.0)*3.25/2*1.9*2*24.0

= 437.8 (kN)

2) Column

W2= 3.14*(2.1/2)^2*10.0*24.0

= 831.0 (kN)

3) Coping + Column

Wp= W1+W2

= 831.0+437.8

= 1269.0 (kN); rounded up

4) Footing

Wf= 4.5*7.0*1.9*24.0

= 1436.0 (kN)

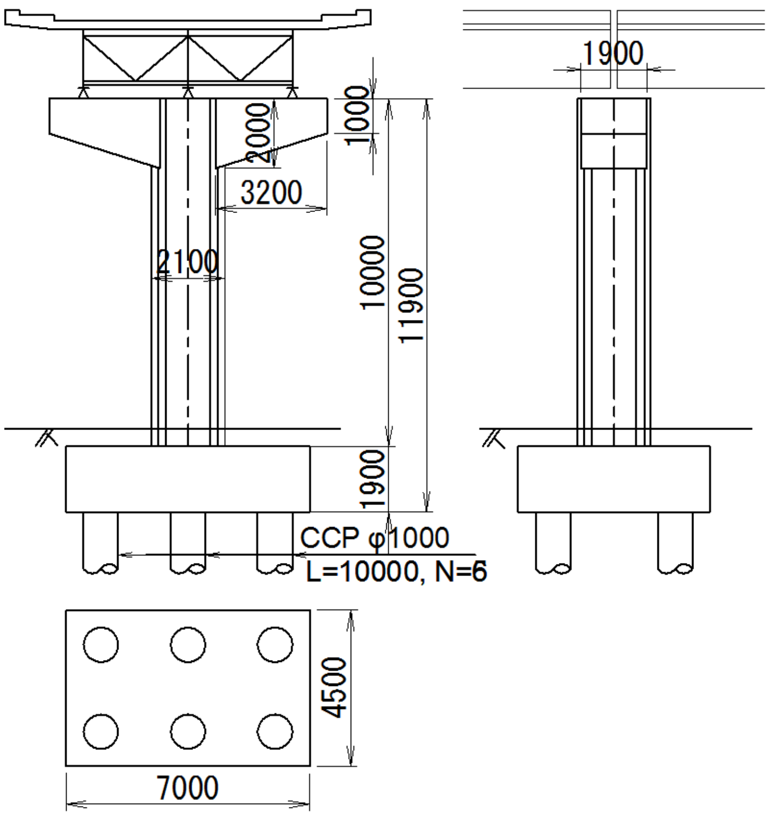
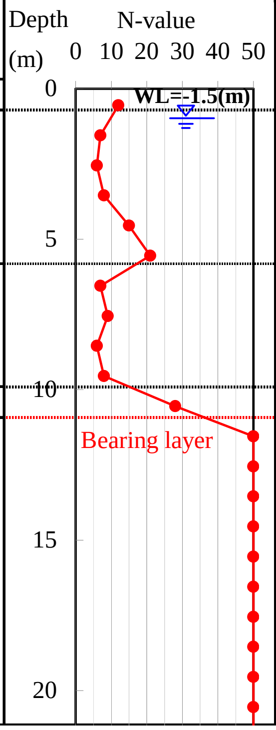


Figure 2.2.2-3 Dimension of the Pier

1) Ground Condition (Liquefiable Ground Condition: Pile Foundation)

In order to explain the design process of pile foundation at the liquefiable site, the ground condition of Lambingan Bridge is chosen.

α= 4, Eo= 2800*N



Layer symbol	Layer Thickness (m)	N-value Blows	N-value Ave.	γt (kN/m²)	FC (%)	D50 (mm)	C (kN/m²)	φ (°)	α*Eo (kN/m²)	Vsn (m/sec)
Bs	1	12	12	17	0.9	0.74	0	35	8,400	183
As	5	7	11	17	17.3	0.14	0	34	7,700	178
		6		17	28.0	0.12				
		8		17	12.0	0.21				
		15		17	7.3	0.42				
		21		17	7.3	0.20				
Ac	4	7	7	15	58.1	-	44	0	4,900	191
		9		15	77.1	-				
		6		15	66.9	-				
		8		15	51.0	-				
WGF	1	28	28	17	0.2	2.38	-	37	19,600	292
GF	10	50	50	21	0.5	0.60	173	21	39,530	292
		50		21	-	-				
		50		21	-	-				
		50		21	-	-				
		50		21	-	-				
		50		21	-	-				
		50		21	-	-				
		50		21	-	-				
		50		21	-	-				
		50		21	-	-				

Layer Symbol	Soil characteristics
Bs	Medium sand; brown colored; and with broken shell fragments
As	Silty fine sand; soft or loose; relatively high water content; and mostly gray colored
Ac	Sandy clay or clayey sand; dark-gray colored; and moderate water content
WGF	Weathered rock; strongly weathered (probably tuff); gray colored; and sand-like
GF	Tuff breccia, tuffs, and tuffaceous sandstones; brownish-gray colored; 11 m 17 m: strongly weathered portion; and below 17 m: fresh and/or welded portion
	Fine sand; with broken shell fragments; including fines and gravel; and dark-gray or brownish-gray colored
	No core recovered; and probably fine sand
	Strongly welded tuff; and black colored

Figure 2.2.2-4 Ground Condition for Pile Foundation Design

Soil type of the ground is classified as “Type-II” as shown below.

Table 2.2.2-1 Result of Soil Classification

Layer		Layer thickness Hi (m)	N-value	Vsi (m/s)	Hi/Vsi (s)
Bs	Sand	1.0	12	183	0.005
As	Sand	5.0	11	178	0.028
Ac	Cray	4.0	7	191	0.021
WGF	Rock	1.0	28	292	0.003
		$T_G = 4*\Sigma(H/Vs)$			0.232
		Soil Type			Type-II

Criteria for the classification

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

Where,

T_G: characteristic value of ground (s); $T_G = 4\sum_{i=1}^{n}\frac{H_i}{V_{si}}$

Hi: i-th layer thickness (m)

Vsi: average shear elastic wave velocity of the i-th layer (km/s);

$$V_{si}\begin{cases} = 100N_i^{1/3}\ (1 \le N_i \le 25) & \text{(for cohesive soil layer)} \\ = 80N_i^{1/3}(1 \le N_i \le 50) & \text{(for sandy soil layer)} \end{cases}$$

2.2.3 Design Acceleration Response Spectra

The design acceleration spectra is formed as follows, corresponds to the site condition of Lambingan Bridge.

[Step-1: Identification of specific values of acceleration coefficients on the maps]

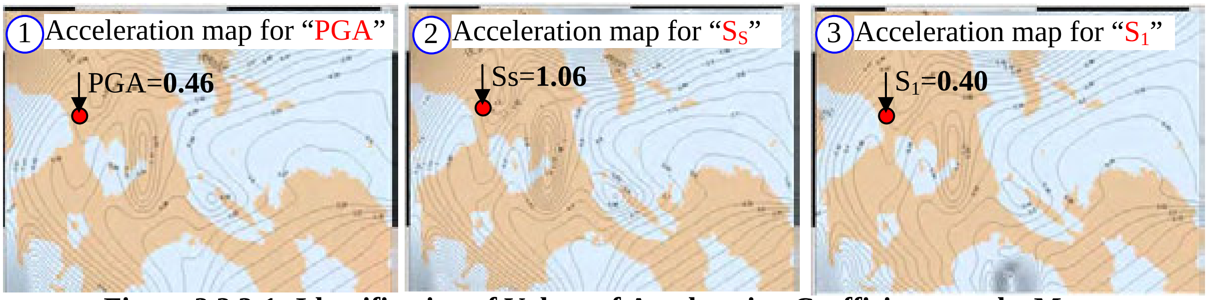


Figure 2.2.3-1 Identification of Values of Acceleration Coefficients on the Maps

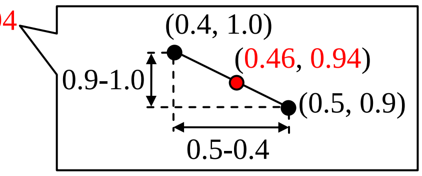
[Step-2: Identification of specific values of site factors]

		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}= 1.0+(0.9-1.0)/(0.50-0.40)*(0.46-0.40)

= 0.94

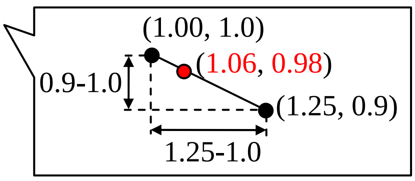


		2 Fa (site factor for Ss)					
Ss (T=0.2)		Ss< 0.25	Ss= 0.50	Ss= 0.75	Ss= 1.00	Ss= 1.25	Ss≥ 2.0
Soil type	I	1.2	1.2	1.1	1.0	1.0	1.0
	II	1.6	1.4	1.2	1.0	0.9	0.85
	III	2.5	1.7	1.2	0.9	0.8	0.75

[Linear interpolation of Fa]

Fa= 1.0+(0.9-1.0)/(1.25-1.00)*(1.06-1.00)

= 0.98



		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

Fv=1.6

Figure 2.2.3-2 Calculation of Site Factors

[Step-3: 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)

[Step-4: Form spectra by connecting the plotted points (Csm=S_{D1}/T, if “Ts<T”)]

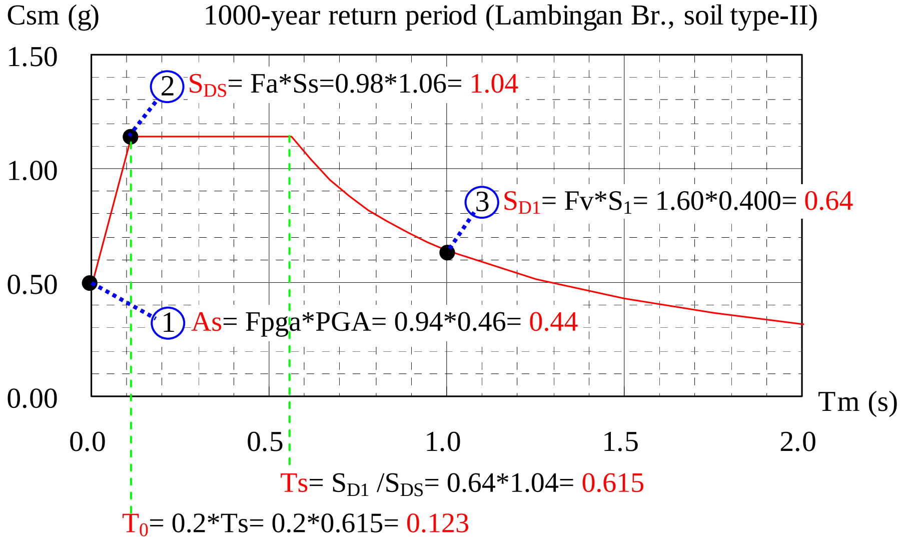
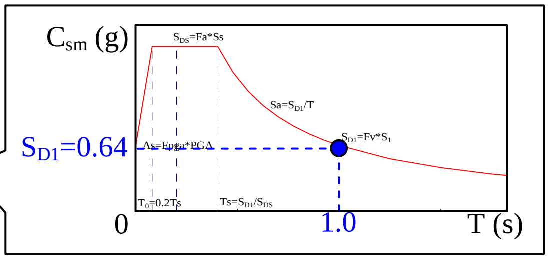


Figure 2.2.3-3 Acceleration response Spectra (Lambingan Br., Soil Type-II)

2.2.4 Confirmation of Seismic Performance Zone

As shown in the following figure, the target bridge is categorized as “Seismic Zone 4 (SZ-4)” with the value of S_{D1}, 0.64. In accordance with the following classification result, “multimode elastic method (MM)” shall be applied for multispan bridges whose importance is categorized as “Essential”. However, in the design example, “uniform load elastic method (UL)” is applied for simplification of analysis. All the other design requirements for SZ-4 are followed in the design.

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
$0.50 \leq S_{D1}$	SZ-4



Confirmation of 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

MM = multimode elastic method

Figure 2.2.4-1 Seismic Performance Zone and Minimum Analysis Requirements

2.2.5 Confirmation of Response Modification Factors

As for R-factor, R=2.0 is applied to design of single column in accordance with the requirement for "Essential Bridge" shown in the following table.

Table 2.2.5-1 Response Modification Factors for Substructures

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

2.2.6 Assessment of Liquefaction Potential

Out of the layers of the site ground, only layer "As" satisfies the following three requirements for liquefaction potential assessment.

1) soil layer which classified into alluvial sand

2) soil layer which contains fine content (FC) less than 35%

3) soli layer which has mean particle size (D_{50}) less than 10mm and particle size at 10% pass on the grading curve (D_{10}) is less than 1mm

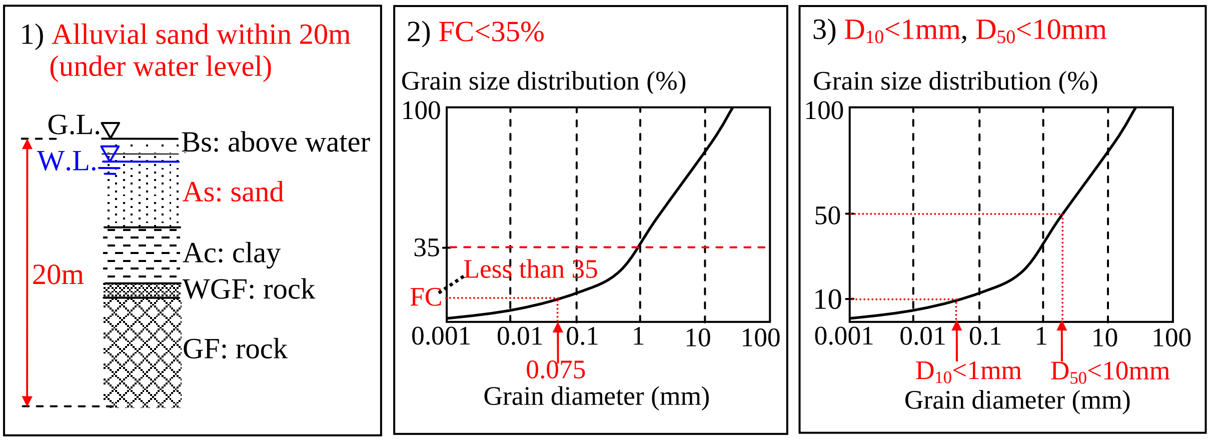


Figure 2.2.6-1 Confirmation for Necessity of Liquefaction Potential Assessment

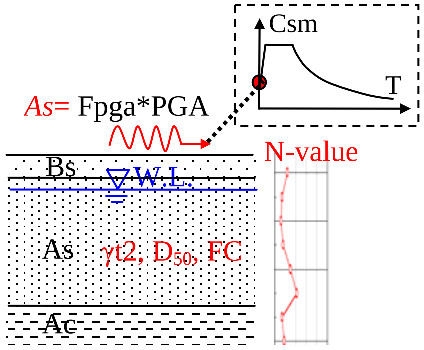
The liquefaction potential of layer "As" is assessed with the following four steps.

[Step-1. Determination of design parameters]

- *As*= Fpga*PGA= 0.44 (g): acceleration coefficient
- Water level: -1.5 (m) from the ground surface
- Soil parameters of layer "As"

Depth X (m)	Layer symbol	N-value	γt2 (kN/m3)	D50 (mm)	FC (%)
0.00	Bs				
1.00	As	12	18.0	0.74	0.9
2.00		7	18.0	0.14	17.3
3.00		6	18.0	0.12	28.0
4.00		8	18.0	0.21	12.0
5.00		15	18.0	0.42	7.3
6.00		21	18.0	0.20	7.3

[Design parameters]



Where,

γt2: unit weight of soil below water level (kN/m3);

γt2= γt+1

γt1: unit weight of soil above water level (kN/m3)

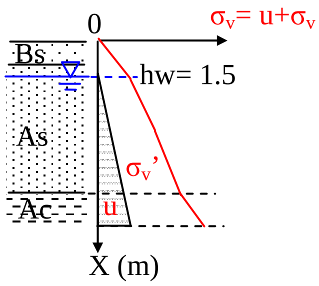
D_{50}: mean particle size (mm)

FC: fine content (%)

N-value (SPT result)

[Step-2. Calculation of "σ_v & σ_v' "]

Depth X (m)	Layer symbol	N-value	γt2 (kN/m3)	γt1=γt2-1 (kN/m3)	σv (kN/m2)	σv' (kN/m2)
0.00	Bs				0.0	0.0
1.00	As	12	18.0	17.0	17.0	17.0
2.00		7	18.0	17.0	34.5	29.5
3.00		6	18.0	17.0	52.5	37.5
4.00		8	18.0	17.0	70.5	45.5
5.00		15	18.0	17.0	88.5	53.5
6.00		21	18.0	17.0	106.5	61.5



u= γw*X (kN/m2)

σ_v'= σ_v - u (kN/m2)

[Calculation Example]

At depth X= 6.0 (m),

$\sigma_v = \gamma_{t1} * h_w + \gamma_{t2} * (X - h_w)$

= 17*1.5+18*(6-1.5)

= 106.5 (kN/m2)

$\sigma_v' = \sigma_v - \gamma_w * (X - h_w)$

= 106.5-10*(6-1.5)

= 61.5 (kN/m2)

Where,

σ_v: total overburden pressure (kN/m2);

$$\sigma_v = \gamma_{t1} * h_W + \gamma_{t2} * (X - h_W)$$

σ_v': effective overburden pressure (kN/m2);

$$\sigma_v' = \sigma_v - u = \sigma_v - \gamma_w * X$$

γ_w: unit weight of water (kN/m3) ; γ_w =10

h_w: depth at water level (m); h_w= 1.5

[Step-3. Calculation of "Na"]

Depth X (m)	Layer symbol	N-value	D50 (mm)	FC (%)	σv' (kN/m2)	N1	C1	C2	Na
0.00	Bs				0.0		-	-	-
1.00	As	12	0.74	0.9	17.0	23.448	1.000	0.000	23.4
2.00		7	0.14	17.3	29.5	11.960	1.146	0.406	14.1
3.00		6	0.12	28.0	37.5	9.488	1.360	1.000	13.9
4.00		8	0.21	12.0	45.5	11.775	1.040	0.111	12.4
5.00		15	0.42	7.3	53.5	20.648	1.000	0.000	20.6
6.00		21	0.20	7.3	61.5	27.148	1.000	0.000	27.1

Where,

Na: modified N-value considering grain size effect

- for sand layers

$$Na = C1 * N1 + C2$$

Where,

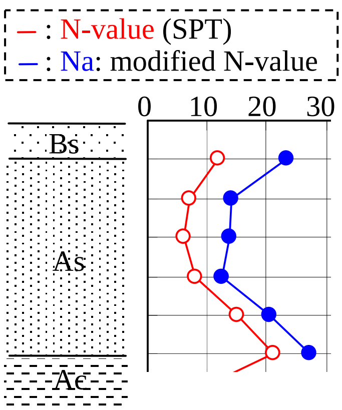
N1= 170*N/($\sigma_v{}'$+70)

$$C1 = \begin{cases} 1 & (0 \leqq FC < 10) \\ (FC+40)/50 & (10 \leqq FC < 60) \\ FC/20-1 & (60 \leqq FC) \end{cases}$$

$$C2 = \begin{cases} 0 & (0 \leqq FC < 10) \\ (FC-10)/18 & (10 \leqq FC) \end{cases}$$

- for gravel layers

$$Na = \{1 - 0.36 \log_{10}(D_{50}/2)\} N1$$



[Calculation Example]

At depth X=2.0 (m),

N1= 170*N/($\sigma_v{}'$+70)

= 170*7/(29.5+70)

= 11.96

C1= (FC+40)/50

= (17.3+40)/50

= 1.146

C2= (FC-10)/18

= (17.3-10)/18

= 0.406

Therefore,

Na= C1*N1+C2

= 1.146*111.96+0.406

= 14.11

[Step-4. Calculation of "F_L"]

Depth X (m)	Layer symbol	σv (kN/m2)	σv' (kN/m2)	Na	R	L	F_L
0.00	Bs	0.0	0.0	-	-	-	-
1.00	As	17.0	17.0	23.4	-	-	-
2.00		34.5	29.5	14.1	0.254	0.499	0.509
3.00		52.5	37.5	13.9	0.252	0.588	0.429
4.00		70.5	45.5	12.4	0.238	0.641	0.371
5.00		88.5	53.5	20.6	0.315	0.673	0.469
6.00		106.5	61.5	27.1	0.526	0.693	0.758

Where,

$$F_L = R / L$$

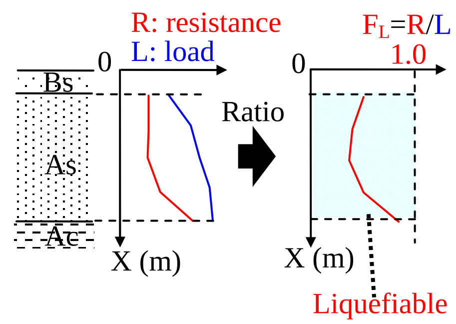
R: dynamic shear strength ratio (resistance);

$$R = \begin{cases} 0.0882\sqrt{Na/1.7} & (Na < 14) \\ 0.0882\sqrt{Na/1.7} + 1.6*10^{-6}(Na-14)^{4.5} & (14 \le Na) \end{cases}$$

L: seismic shear stress ratio (load);

$$L = (1 - 0.015 * X) * As * \sigma_v / \sigma_v'$$

X: depth from ground surface (m)



[Calculation Example]

At X= 2.0 (m),

Na= 14.1 > 14 then,

$R= 0.0882\sqrt{Na/1.7+1.6*10^{-6}}$

$= 0.0882\sqrt{14.1*1.7+1.6*10^{-6}}$

$= 0.254$

$L= (1-0.015*X)*As*\sigma_v/\sigma_v'$

$= (1-0.015*2.0)*0.44*34.5/29.5$

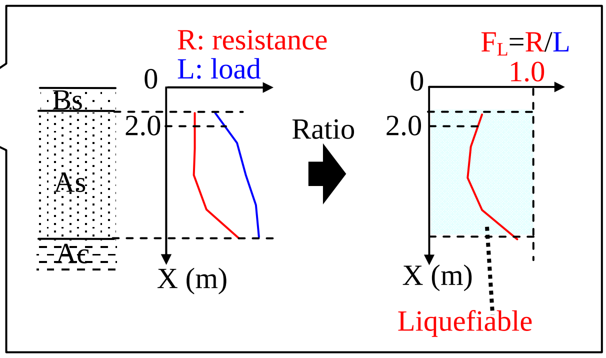
$= 0.499$

Therefore,

$F_L= R/L$

$= 0.254/0.499$

$= 0.509 < 1.0$ (Liquefiable)



[Step-4'. Determination of "DE"]

As shown below, soil coefficient reduction factor, "DE", can be determined with the average value of "R" and "F_L". "DE" is applied in pile foundation design in order to consider the reduction of soil parameters due to liquefaction effect.

Result of liquefaction potential assessment

Depth X (m)	Layer symbol	R	R (Ave.)	L	F_L	F_L (Ave.)
0.00	Bs	-	-	-	-	-
1.00	As	-	-	-	-	-
2.00		0.254		0.499	0.509	
3.00		0.252		0.588	0.429	
4.00		0.238	0.317	0.641	0.371	0.507
5.00		0.315		0.673	0.469	
6.00		0.526		0.693	0.758	

0≦X≦10 0.3<R $1/3<F_L≦2/3$

Determination of "DE"

Soil coefficient reduction factor "DE"

F_L	X (m)	DE	
		R≦0.3	0.3<R
$F_L≦1/3$	0≦X≦10	0	1/6
	10<X≦20	1/3	1/3
$1/3<F_L≦2/3$	0≦X≦10	1/3	2/3
	10<X≦20	2/3	2/3
$2/3<F_L≦1$	0≦X≦10	2/3	1
	10<X≦20	1	1

DE=2/3

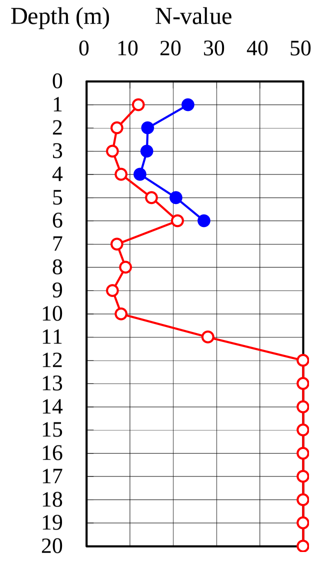
Figure 2.2.6-2 Determination of Soil Coefficient Reduction Factor

The overall calculation of liquefaction potential assessment is summarized in the next page.

Table 2.2.6-1 Result of Liquefaction Potential Assessment (Ground Condition at Lambingan Br. Site)

Water Lv. 1.50 (m)   As=Fpga*PGA 0.44 (g)

Depth X (m)	Layer symbol	N-value	Soil detail	γt2 (kN/m3)	γt1=γt2-1 (kN/m3)	D50 (mm)	FC (%)	σv (kN/m2)	σv' (kN/m2)	N1	C1	C2	Na
0.00	Bs							0.0	0.0		-	-	-
1.00	As	12	1 Alluvial (sand)	18.0	17.0	0.74	0.9	17.0	17.0	23.448	1.000	0.000	23.4
2.00		7	1 Alluvial (sand)	18.0	17.0	0.14	17.3	34.5	29.5	11.960	1.146	0.406	14.1
3.00		6	1 Alluvial (sand)	18.0	17.0	0.12	28.0	52.5	37.5	9.488	1.360	1.000	13.9
4.00		8	1 Alluvial (sand)	18.0	17.0	0.21	12.0	70.5	45.5	11.775	1.040	0.111	12.4
5.00		15	1 Alluvial (sand)	18.0	17.0	0.42	7.3	88.5	53.5	20.648	1.000	0.000	20.6
6.00		21	1 Alluvial (sand)	18.0	17.0	0.20	7.3	106.5	61.5	27.148	1.000	0.000	27.1
7.00	Ac	7	5 Non-liequefiable	16.0	-	-	-	122.5	67.5	8.655	-	-	-
8.00		9	5 Non-liequefiable	16.0	-	-	-	138.5	73.5	10.662	-	-	-
9.00		6	5 Non-liequefiable	16.0	-	-	-	154.5	79.5	6.823	-	-	-
10.00		8	5 Non-liequefiable	16.0	-	-	-	170.5	85.5	8.746	-	-	-
11.00	WGF	28	5 Non-liequefiable	18.0	-	-	-	188.5	93.5	29.113	-	-	-
12.00	GF	50	5 Non-liequefiable	22.0	-	-	-	210.5	105.5	48.433	-	-	-
13.00		50	5 Non-liequefiable	22.0	-	-	-	232.5	117.5	45.333	-	-	-
14.00		50	5 Non-liequefiable	22.0	-	-	-	254.5	129.5	42.607	-	-	-
15.00		50	5 Non-liequefiable	22.0	-	-	-	276.5	141.5	40.189	-	-	-
16.00		50	5 Non-liequefiable	22.0	-	-	-	298.5	153.5	38.031	-	-	-
17.00		50	5 Non-liequefiable	22.0	-	-	-	320.5	165.5	36.093	-	-	-
18.00		50	5 Non-liequefiable	22.0	-	-	-	342.5	177.5	34.343	-	-	-
19.00		50	5 Non-liequefiable	22.0	-	-	-	364.5	189.5	32.755	-	-	-
20.00		50	5 Non-liequefiable	22.0	-	-	-	386.5	191.6	32.492	-	-	-



— : N-value (SPT)
— : Na: modified N-value

Depth X (m)	Layer symbol	N-value	Soil detail	R	L	F_L
0.00	Bs			-	-	-
1.00	As	12	Alluvial (sand)	-	-	-
2.00		7	Alluvial (sand)	0.254	0.499	0.509
3.00		6	Alluvial (sand)	0.252	0.588	0.429
4.00		8	Alluvial (sand)	0.238	0.641	0.371
5.00		15	Alluvial (sand)	0.315	0.673	0.469
6.00		21	Alluvial (sand)	0.526	0.693	0.758
7.00	Ac	7	Non-liequefiable	-	-	-
8.00		9	Non-liequefiable	-	-	-
9.00		6	Non-liequefiable	Ave. -	-	Ave. -
10.00		8	Non-liequefiable	0.317 -	-	0.465 -
11.00	WGF	28	Non-liequefiable	-	-	-
12.00	GF	50	Non-liequefiable	-	-	-
13.00		50	Non-liequefiable	-	-	-
14.00		50	Non-liequefiable	-	-	-
15.00		50	Non-liequefiable	-	-	-
16.00		50	Non-liequefiable	-	-	-
17.00		50	Non-liequefiable	-	-	-
18.00		50	Non-liequefiable	-	-	-
19.00		50	Non-liequefiable	-	-	-
20.00		50	Non-liequefiable	-	-	-

Depth (m)	Layer	R (Ave.)	FL (Ave.)	DE
0.00 – 7.00	As	0.317	0.507	2/3
7.00 – 11.00	Ac	-	-	-
11.00 – 12.00	WGF	-	-	-
12.00 – 20.00	GF	-	-	-

Liquefiable layer

Soil coefficient reduction factor "DE"

F_L	X (m)	DE: R≦0.3	DE: 0.3<R
F_L≦1/3	0≦X≦10	0	1/6
	10<X≦20	1/3	1/3
1/3<F_L≦2/3	0≦X≦10	1/3	2/3
	10<X≦20	2/3	2/3
2/3<F_L≦1	0≦X≦10	2/3	1
	10<X≦20	1	1

2.3 Initial Bridge Analysis and Substructure Design

2.3.1 Recommended Design Procedure

As the first step of seismic design, initial dimension of structures is recommended to be calculated as follows.

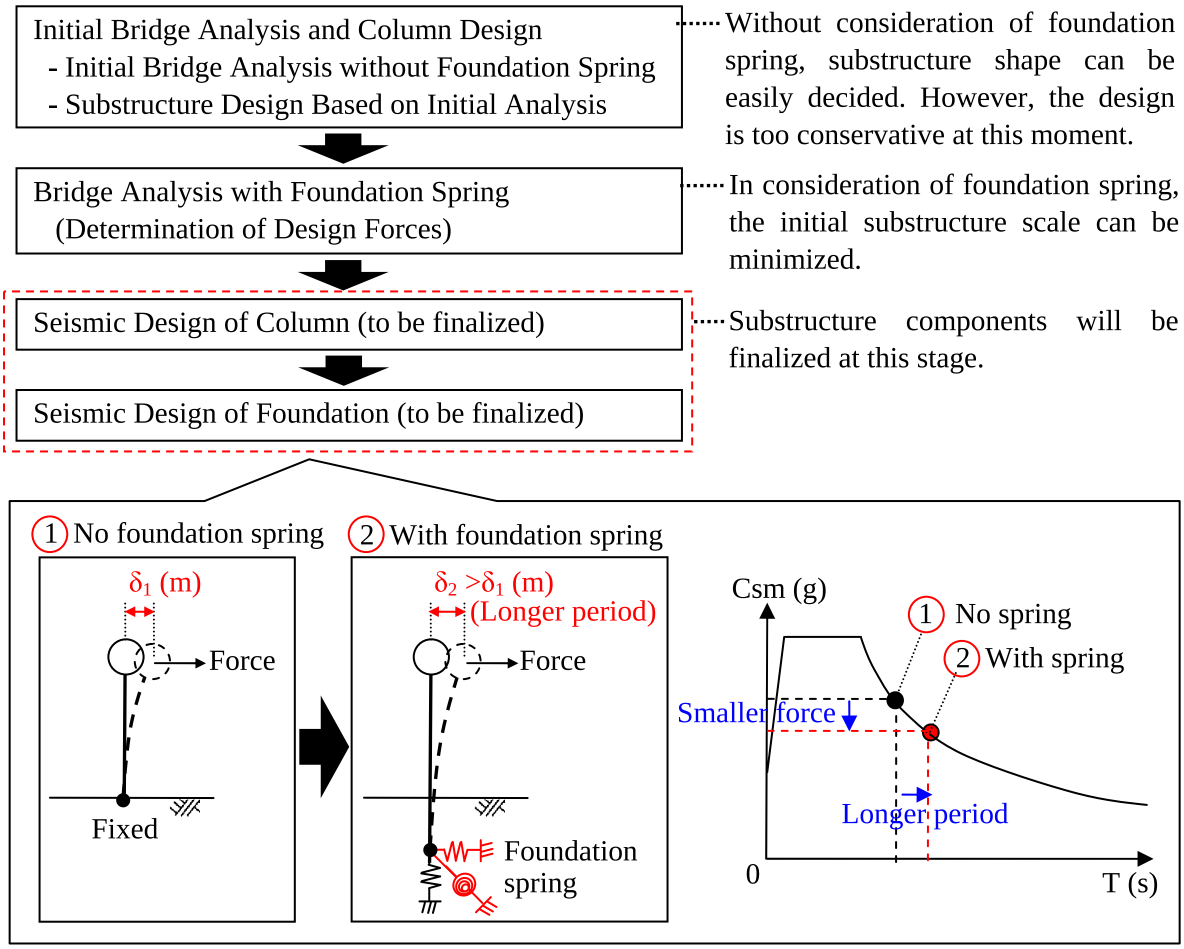


Figure 2.3.1-1 Recommended Design Procedure

2.3.2 Initial Bridge Analysis

As initial bridge analysis, natural period of the bridge structure is calculated with the single degree of freedom method without modeling foundation spring. The detail of analysis is explained only for longitudinal direction which dominates of the substructure dimension.

Analysis methodology and analysis result are shown from the next page.

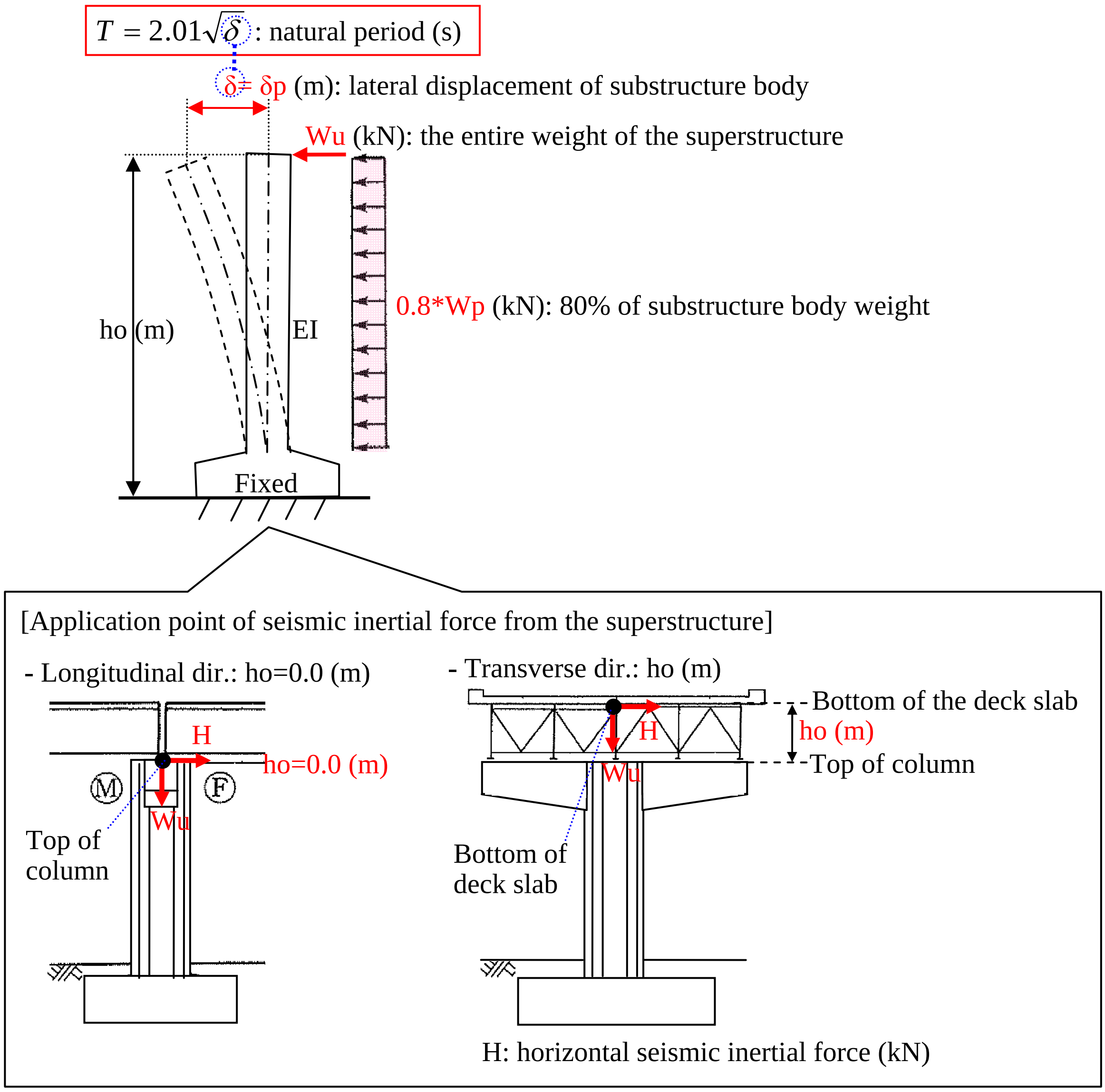


Figure 2.3.2-1 Single Degree of Freedom Method for Natural Period Acquisition

- Bridge analysis for longitudinal direction -

[Step-1. Calculation of deflection due to column bending, "δp"]

$$\delta_p = \frac{Wu * h^3}{3 * E * Ic} + \frac{0.8 * W_p * h_p^3}{8 * E * Ic}$$

$$= \frac{2900 * 10^3}{3 * 25000000 * 0.477} + \frac{0.8 * 1269 * 10^3}{8 * 25000000 * 0.477}$$

$$= 0.092 \text{ (m)}$$

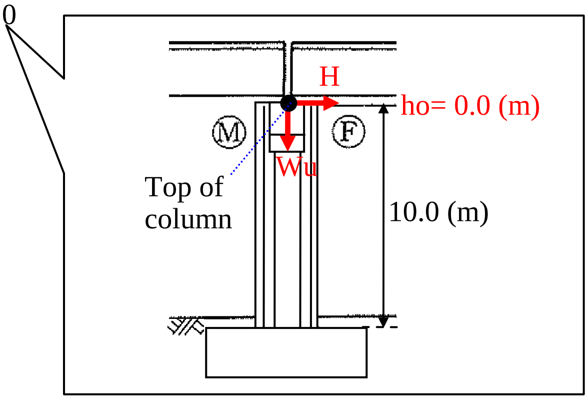
Where,

H: seismic inertial force from the superstructure (kN); H=Wu*Csm= 2900*1.0= 2900

Wu: superstructure weight undertaken by the pier under EQ; Wu= 2900

Csm: seismic coefficient (g); Csm= 1.0

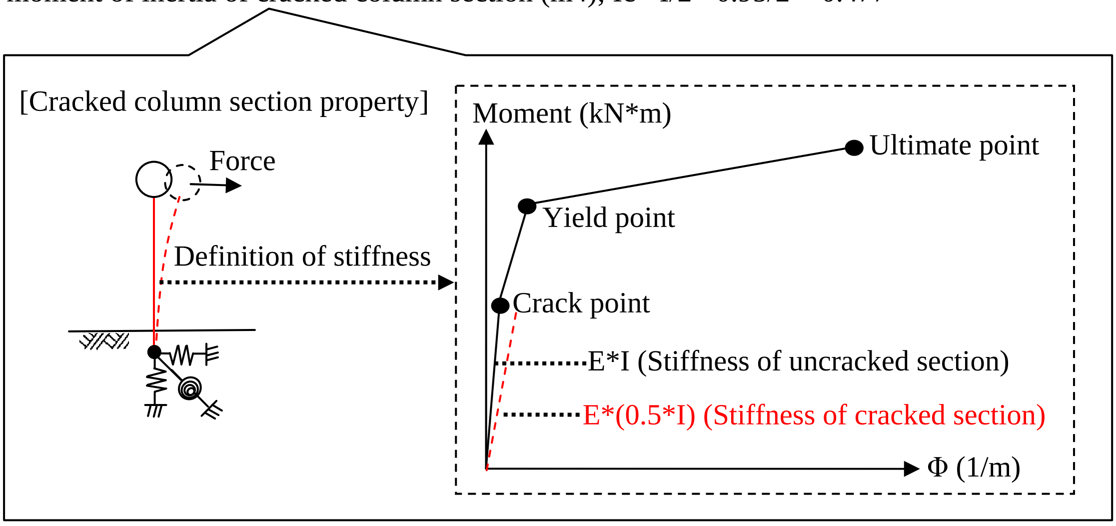
h: height from column base to application point of seismic inertial force from the superstructure (m); h= 10.0



E: concrete modulus of elasticity (kN/m2); E= 25000000

I: moment of inertia of the column (m4); I= π*(D/2)^4/4= 3.14*(2.1/2)^4/4= 0.95

Ic: moment of inertia of cracked column section (m4); Ic= I/2= 0.95/2 = 0.477



D: column diameter (m); D= 2.1

Wp: weight of pier coping and column (kN); Wp= 1269

hp: column height (m); hp= 10.0

[Step-2. Calculation of natural period]

$$T = 2.01\sqrt{\delta}$$
$$= 2.01\sqrt{0.092}$$
$$= 0.61 \text{ (s)}$$

[Step-3. Determination of design seismic coefficient]

Csm= 1.04 (g)

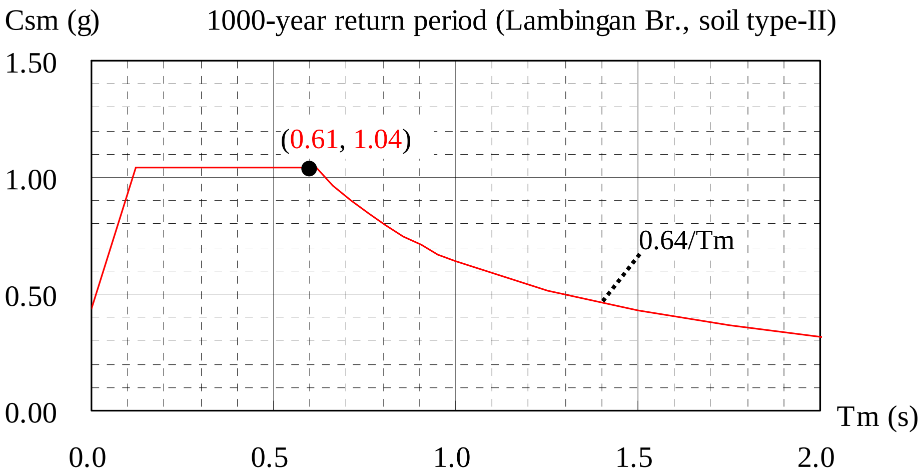


Figure 2.3.2-2 Design Seismic Coefficient for Longitudinal Direction

2.3.3 Initial Substructure Design

Based on the initial bridge analysis, the following substructure shape is decided. Calculation detail of initial substructure design is skipped here to avoid the repetition of explanation of design process.

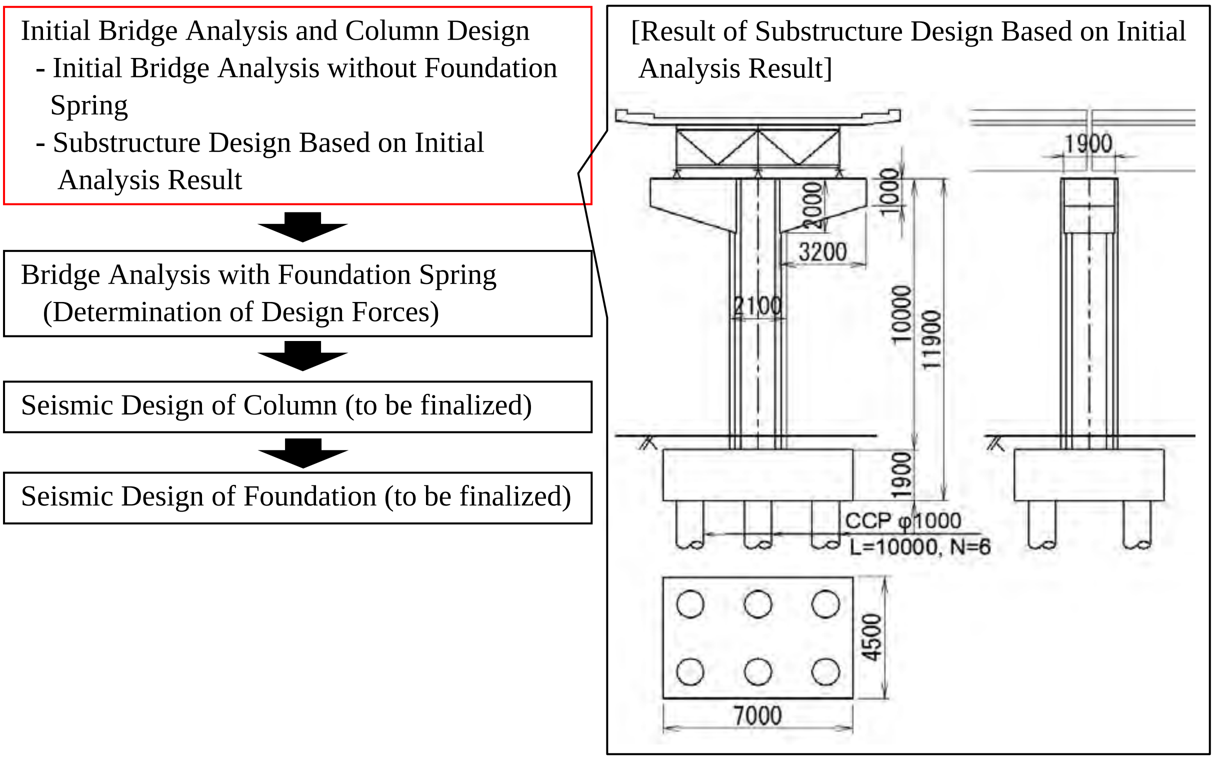


Figure 2.3.3-1 Initial Assumption of Pier Dimension

The bridge analysis with foundation spring is explained with the above substructure condition from the next page.

2.4 Bridge Analysis with Foundation Spring (Determination of Design Forces)

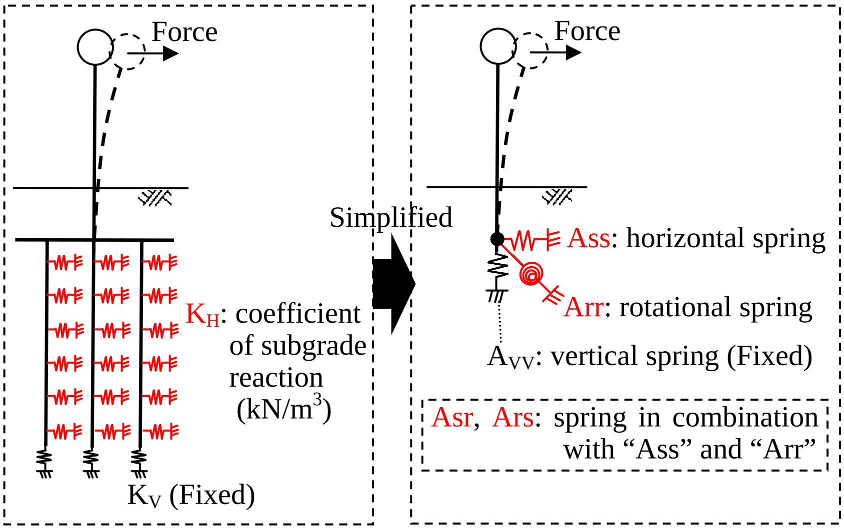
2.4.1 Spring Properties of the Pile Foundation

[Step-0. Preliminary calculation of design parameters]

Design parameters for pile foundation springs are summarized as follows.

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)
As	Sand	3.50	11	178	0.8	142	17.0	34978	0.5	104934
Ac	Cray	4.00	7	191	0.8	153	15.0	35830	0.5	107490
WGF	Cray	1.00	28	292	0.8	233	17.0	94174	0.5	282522
GF	Cray	10.00	50	292	0.8	233	21.0	116333	0.5	348999

Pile foundation springs can be determined with the above parameters.



[Calculation example]

For layer "As",

Vsi= 80*Ni$^{1/3}$

= 80*11$^{1/3}$

= 178 (m/s)

V_{SD}= Cv*Vsi

= 0.8*178

= 142 (m/s)

G_D= γt*V_{SD}^2/g

= 17*142^2/9.8

= 34978 (kN/m2)

E_D= 2*(1+ν_D)*G_D

= 2*(1+0.5)*34978

= 104934 (kN/m2)

Where,

E_D: dynamic modulus of ground deformation (kN/m2); $E_D = 2*(1+\nu_D)*G_D$

G_D: dynamic shear modulus of ground deformation (kN/m2); $G_D = \dfrac{\gamma_t}{g} V_{SD}^2$

ν_D: dynamic Poisson's ratio of the ground (0.45 for soil above water, 0.5 for soil under water)

γt: unit weight of the ground (kN/m3)

g: acceleration of the gravity; g= 9.8(ms2)

$V_{SD:}$ shear elastic wave velocity of the ground (m/s); $V_{SD} = c_v * V_{si}$

c_v: modification factor based on degree of ground strain; $c_v \begin{cases} = 0.8\ (V_{si} < 300m/s) \\ = 1.0\ (V_{si} \geq 300m/s) \end{cases}$

Vsi: the average shear wave velocity of the i-th soil layer;

$$V_{si} \begin{cases} = 100 N_i^{1/3}\ (1 \leq N_i \leq 25) \\ \text{(for cohesive soil layers)} \\ = 80 N_i^{1/3} (1 \leq N_i \leq 50) \\ \text{(for sandy/cohesionless soil layers)} \end{cases}$$

[Step-1. Assumption of “1/β” and initial calculation of “B_H”]

Typically, “1/β” for bridge analyses is 3 times as large as the pile diameter. Therefore, the parameter “β” is initially assumed as “3”; $(1/\beta_1) = 3.0$

Then, equivalent loading width of foundation, “B_H”, is initially determined with the following equation.

$B_H = \sqrt{D/\beta 1}$

$\quad = \sqrt{1.0*3.0}$

$\quad = 1.732$ (m)

Where,

D: pile diameter (m); D= 1.0

[Step-2. Initial calculation of “K_H”]

Coefficient of subgrade reaction, K_H, is calculated as follows. In the calculation process, average value of “E_D” within “1/β” range, “$(E_D)_\beta$”, should be applied, focusing on the effective range of “K_H” The image of relationship among parameters is illustrated in the following figure.

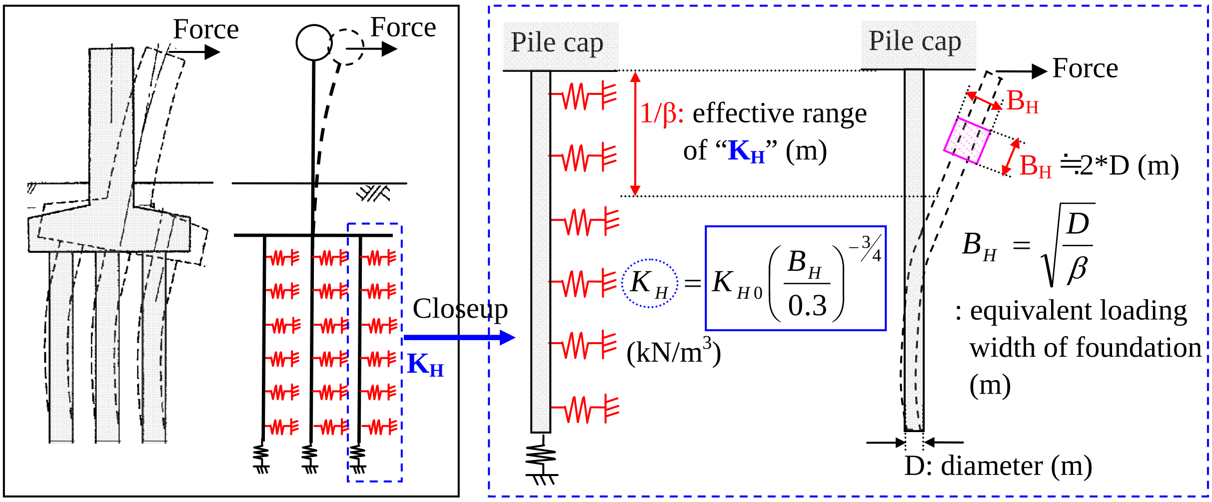


Figure 2.4.1-1 Coefficient of Subgrade Reaction, “K_H”

$$(E_D)_\beta = \frac{\sum E_D i * Li}{1/\beta}$$

$\quad = \Sigma(104934*3.0)/3.0$

$\quad = 104934$ (kN/m2)

Then,

$K_{H0} = (1/0.3)*(E_D)_\beta$

$\quad = (1/0.3)* 104934$

$\quad = 349780$ (kN/m3)

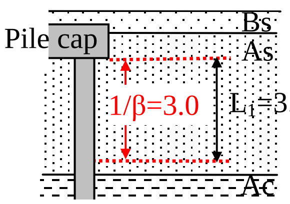
Therefore,

$K_H = K_{H0}*(B_H/0.3)^{-3/4}$

$\quad = 349780*(1.732/0.3)^{-3/4}$

$\quad = 93913$ (kN/m3)

In this case, only one layer within “1/β” range



L_1=3.0 (m), E_{D1}=104934 (kN/m2)

E_Di: dynamic modulus of ground deformation of i-th layer (kN/m2)

Li: Thickness of i-th layer (m)

[Step-3. Initial calculation of “β”]

The pile specific parameter, “β”, is initially calculated as follows.

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

$$= \sqrt[4]{\frac{93913 * 1.0}{4 * 25000000 * 0.049}}$$

$= 0.372\ (m^{-1})$

Then,

$(1/\beta) = 1/0.372$

$= 2.688\ (m)$

Where,

E: Young’s modulus of elasticity; E=25000000 (kN/m2)

I: moment of inertia of a pile;

$I = \pi*(D/2)^4/4$

$= 3.14*(1/2)^4/4$

$= 0.049\ (m4)$

[Step-4. Comparison of “1/β” and “$1/\beta_1$”]

The value of initially assumed “$1/\beta_1$” must be compared with value of “1/β”. In this case, “$1/\beta_1$” is 3.0 while “1/β” is 2.689. If the two values are different, process of step-1-4 shall be iterated using resulted “1/β” replaced with “$1/\beta_1$” until “$1/\beta_1$” becomes equal to “1/β”. The value of “1/β” is finalized as follows.

[Step-1: Assumption of “1/β” and calculation of “B_H”]

(1) $(1/\beta_1) = 2.660$

(2) $B_H = \sqrt{D/\beta_1} = 1.631\ (m)$

[Step-2: Calculation of “K_H”]

(1) $(E_D)_\beta = \frac{\sum E_D i * Li}{1/\beta} = 104934\ (kN/m2)$

(2) $K_{H0} = (1/0.3)*(E_D)_\beta = 349780\ (kN/m3)$

(3) $K_H = K_{H0}*(B_H/0.3)^{-3/4} = 98242\ (kN/m3)$

[Step-3: Calculation of “β”]

(1) $\beta = \sqrt[4]{\frac{K_H * D}{4 * E * I}} = 0.376\ (m^{-1})$

(2) $(1/\beta) = 2.660\ (m) = (1/\beta_1)$ (Finalized)

[Step-5. Calculation of spring properties of single pile]

Without pile projection over the ground surface, spring properties of single pile can be determined as follows.

- Spring property of single pile in horizontal/rotational direction

$K1= 4*E*I*\beta^3$

$= 4*25000000*0.049*0.376^3$

$= 260471$ (kN/m)

$K2= K3 = 2*E*I*\beta^2$

$= 2*25000000*0.049*0.376^2$

$= 346371$ (kN/rad)

$K4= 2*E*I*\beta$

$= 2*25000000*0.049*0.376$

$= 921200$ (kN*m/rad)

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.

- Spring property of single pile in vertical direction

Kv= a*Ap*E/L

= 0.16*0.785*25000000/10.0

= 314000 (kN/m)

Where,

Kv: axial spring constant of a pile

a: modification factor; for CIP pile,

a= 0.031*(L/D)-0.15

= 0.031*(10.0/1.0)-0.15

= 0.16

L: pile length (m); L= 10.0

D: pile diameter (m); D= 1.0

Ap: net cross-sectional area of a pile (m2);

$Ap= \pi*(D/2)^2$

$= 3.14*(1.0/2)^2$

= 0.785 (m2)

[Step-6. Calculation of the entire pile foundation structure]

If the effect of foundation on analyses is focused on "1/β" range, which is the effective range of “K_H”, foundation structure can be modeled as group of springs in one node as shown in the following figure.

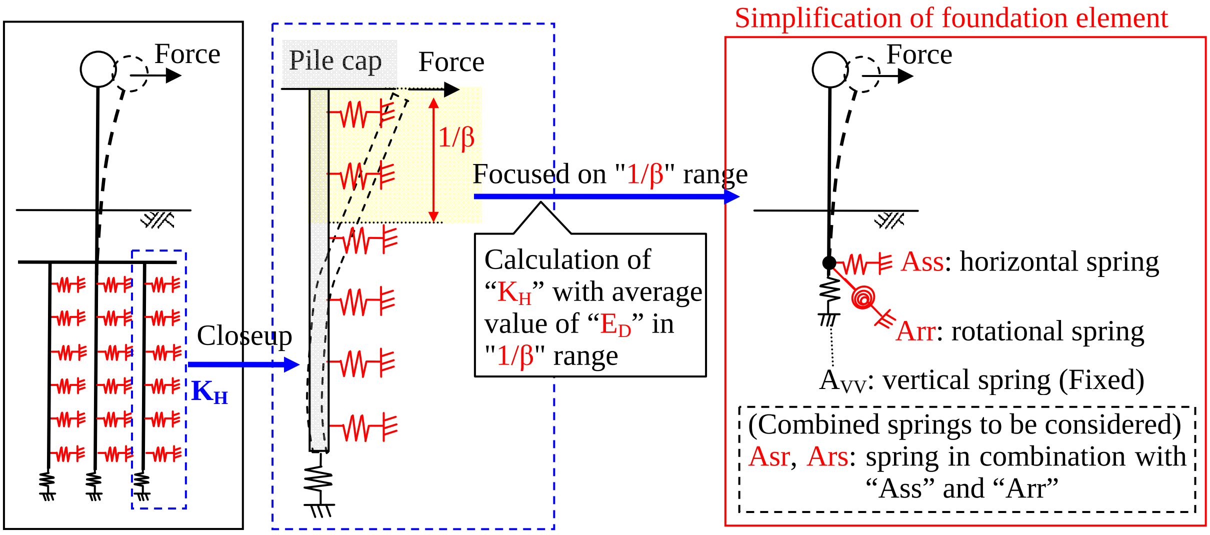
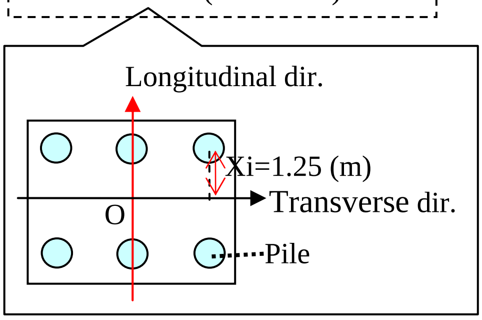


Figure 2.4.1-2 Modeling of Pile Foundation

Therefore, the spring properties of the entire pile foundation are calculated as follows.

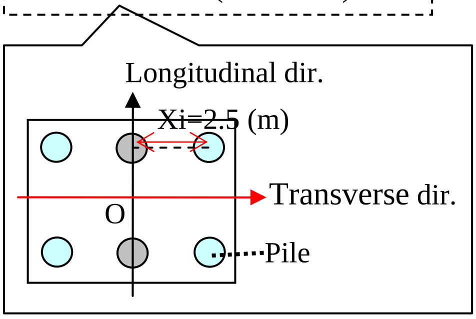
- For longitudinal direction

Ass= n*K1
= 6*260471
= 1562827 (kN/m)
Asr= Ars= -n*K2
= -6*346371
= -2078227 (kN/rad)
Arr= n*K4+KvΣXi^2
= n*K4+Kv*Xi^2*n
= 6*921200+314000*1.25^2*6
= 8470950 (kN*m/rad)



- For transverse direction

Ass= n*K1
= 6*260471
= 1562827 (kN/m)
Asr= Ars= -n*K2
= -6*346371
= -2078227 (kN/rad)
Arr= n*K4+KvΣXi^2
= n*K4+Kv*Xi^2*n
= 6*921200+314000*2.5^2*4
= 13377200 (kN*m/rad)



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 "pile head in i-th row" (m)

2.4.2 Bridge Analysis

2.4.2.1 Bridge Analysis Methodology

As shown below, natural period of the bridge is calculated with the single degree of freedom method.

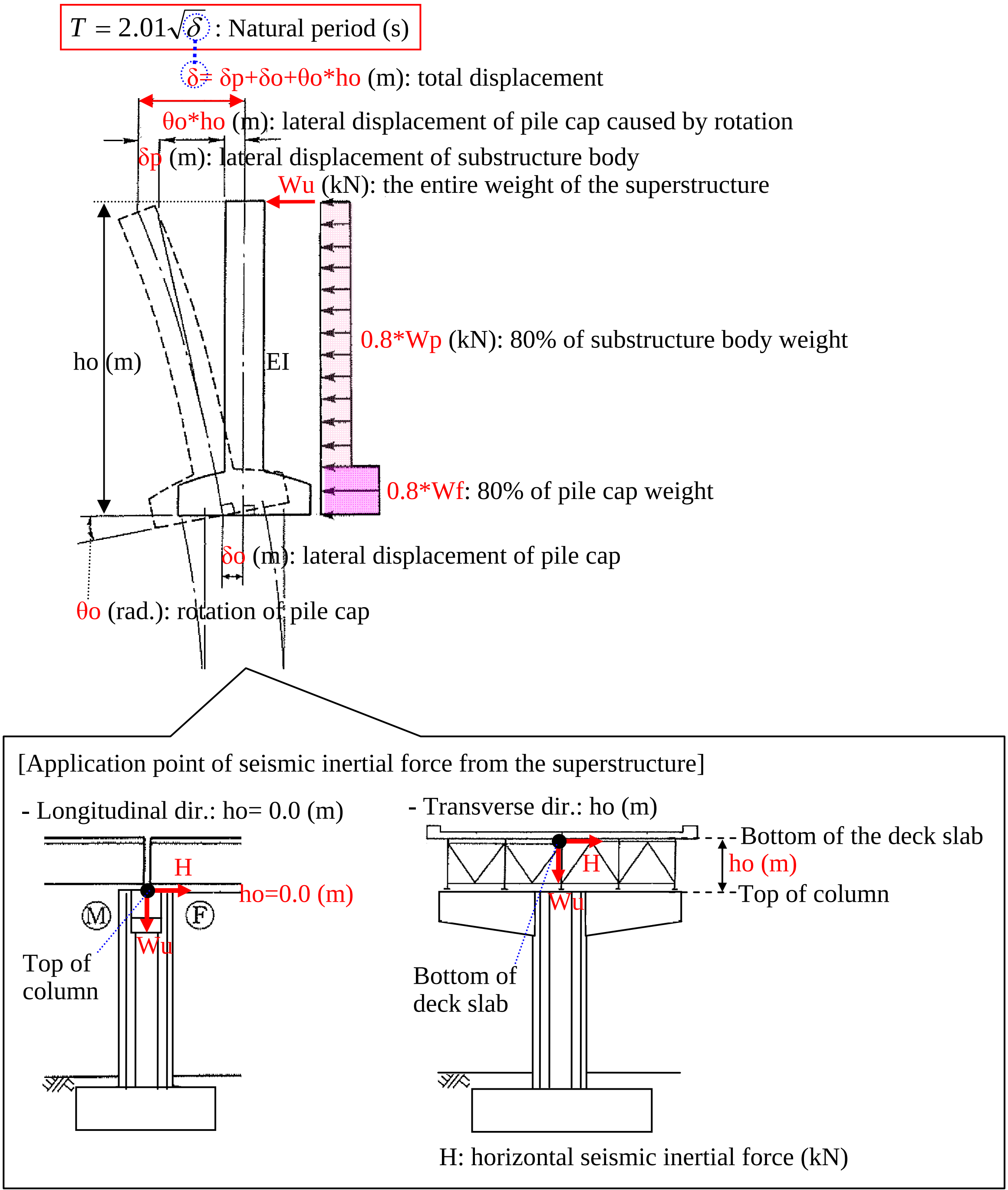


Figure 2.4.2-1 Single Degree of Freedom Method for Natural Period Acquisition

2.4.2.2 Bridge Analysis for Longitudinal Direction

[Step-1. Calculation of deflection due to column bending, "δp"]

$$\delta_p = \frac{Wu * h^3}{3 * E * Ic} + \frac{0.8 * W_p * h_p^3}{8 * E * Ic}$$
$$= \frac{2900 * 10^3}{3 * 25000000 * 0.477} + \frac{0.8 * 1343 * 10^3}{8 * 25000000 * 0.477}$$
$$= 0.092\ (m)$$

Where,

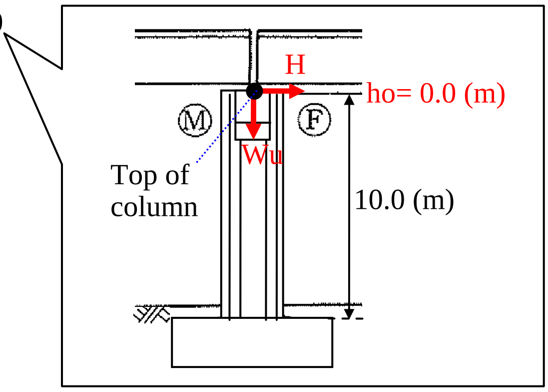
H: seismic inertial force from the superstructure (kN); H= Wu*Csm= 2900*1.0= 2900

Wu: superstructure weight undertaken by the pier under EQ; Wu= 2900

Csm: seismic coefficient (g); Csm= 1.0

h: height from column base to application point of seismic inertial force from the superstructure (m);

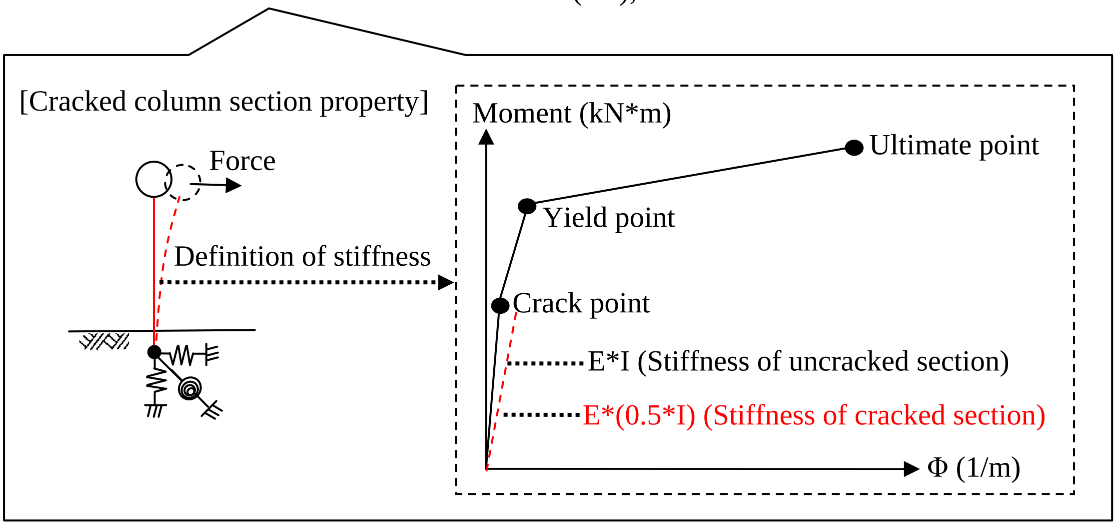
h=10.0



E: concrete modulus of elasticity (kN/mm2)

I: moment of inertia of the column (m4); I= π*(D/2)^4/4= 3.14*(2.1/2)^4/4= 0.95

Ic: moment of inertia of cracked column section (m4); Ic= I/2= 0.95/2 = 0.477



D: column diameter (m); D= 2.1

Wp: weight of pier coping and column (kN); Wp= 1269

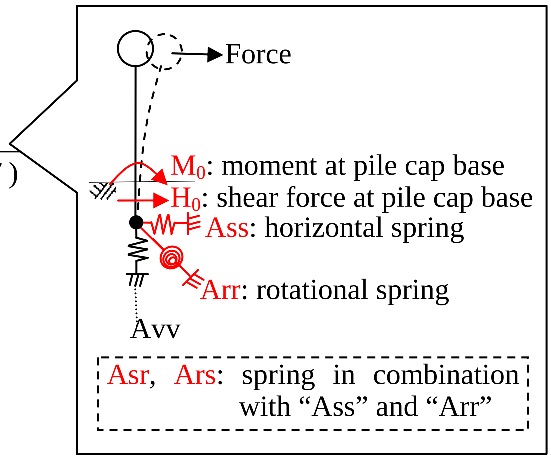
hp: column height (m); hp=10.0

[Step-2. Lateral displacement of pile cap, δo]

$$\delta o = \frac{H_0 * Arr - M_0 * Asr}{Ass * Arr - Asr * Ars}$$

$$= \frac{5064 * 8470950 - 42606 * (-2078227)}{1562827 * 8470950 - (-2078227) * (-2078227)}$$

$$= 0.015 \text{ (m)}$$



Where,

Ho: horizontal force at the pile cap base (kN);

Ho= Wu+0.8*(Wp+Wf)

= 2900+0.8*(1269+1436)

= 5064

Mo: bending moment at the pile cap base (kN*m);

Mo= Wu*ho+0.8*Wp*(0.5*hp+hf)+0.8*Wf*0.5*hf

= 2900*11.9+0.8*1269*(0.5*10.0+1.9)+0.8*1436*0.5*1.9

= 42606

ho: height from pile cap base to application point of seismic inertial force (m); ho=11.9

hf: height of the pile cap (m); hf= 1.9

Wf: weight of the pile cap (kN); Wf= 1436

Ass= 1562827 (kN/m)

Asr, Ars= -2078227 (kN/rad)

Arr= 8470950 (kN*m/rad)

[Step-3. Lateral displacement of pile cap due to rotation of the structure, "θo*ho"]

θo*ho= 0.009*11.9

= 0.107 (m)

Where,

$$\theta o = \frac{-H_0 * Ars + M_0 * Ass}{Ass * Arr - Asr * Ars}$$

$$= \frac{-5046 * (-2078227) + 42606 * 1562827}{1562827 * 8470950 - (-2078227) * (-2078227)}$$

$$= 0.009 \text{ (m)}$$

[Step-4. Calculation of total displacement, "δ"]

δ= δp+δo+θo*ho

= 0.092+0.015+0.107

= 0.213 (m)

[Step-5. Calculation of natural period]

$$T = 2.01\sqrt{\delta}$$
$$= 2.01\sqrt{0.213}$$
$$= 0.93 \text{ (s)}$$

[Step-6. Determination of design seismic coefficient]

Csm= 0.64/Tm

= 0.64/0.93

= 0.69 (g)

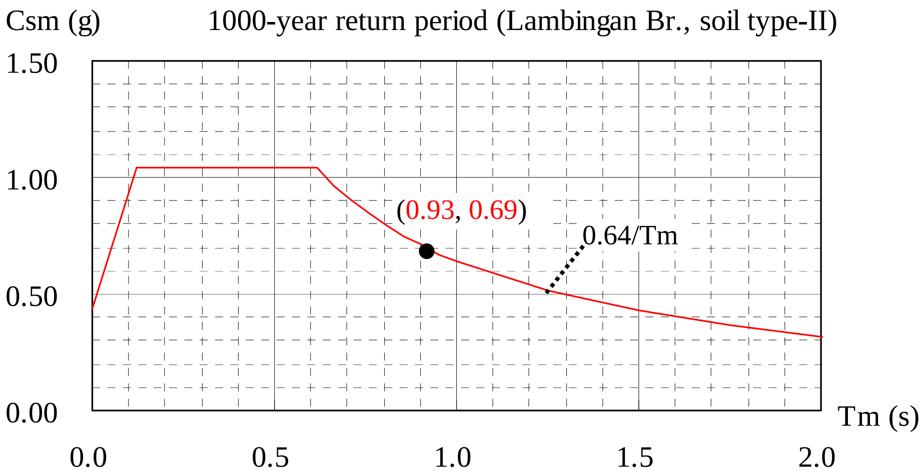


Figure 2.4.2-2 Design Seismic Coefficient for Longitudinal Direction

2.4.2.3 Bridge Analysis for Transverse Direction

[Step-1. Calculation of deflection due to column bending, “δp”]

$$\delta_p = \frac{Wu * h^3}{3 * E * Ic} + \frac{0.8 * W_p * h_p^3}{8 * E * Ic}$$

$$= \frac{2900 * 12^3}{3 * 25000000 * 0.477} + \frac{0.8 * 1269 * 10^3}{8 * 25000000 * 0.477}$$

$$= 0.150 \text{ (m)}$$

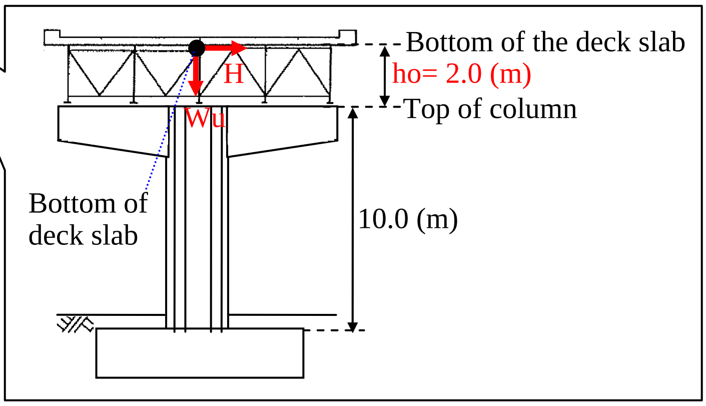
Where,

H: seismic inertial force from the superstructure (kN); H= Wu*Csm= 2900*1.0= 2900

Wu: superstructure weight undertaken by the pier under EQ; Wu= 2900

Csm: seismic coefficient (g); Csm= 1.0

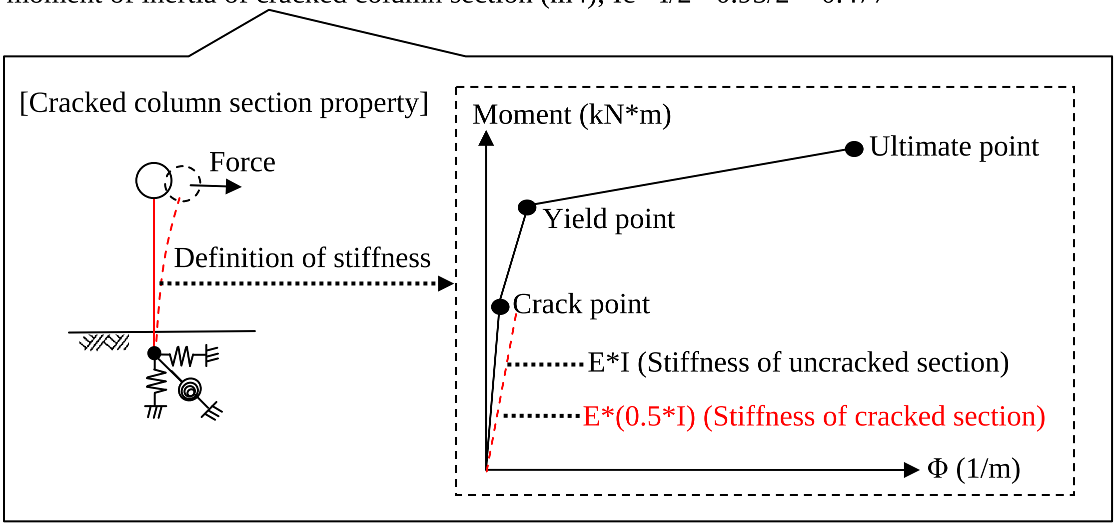
h: height from column base to application point of seismic inertial force from the superstructure (m); h= 12.0



E: concrete modulus of elasticity (kN/mm2)

I: moment of inertia of the column (m4); I= $\pi*(D/2)^4/4$= $3.14*(2.1/2)^4/4$= 0.95

Ic: moment of inertia of cracked column section (m4); Ic= I/2= 0.95/2 = 0.477



D: column diameter (m); D= 2.1

Wp: weight of pier coping and column (kN); Wp= 1269

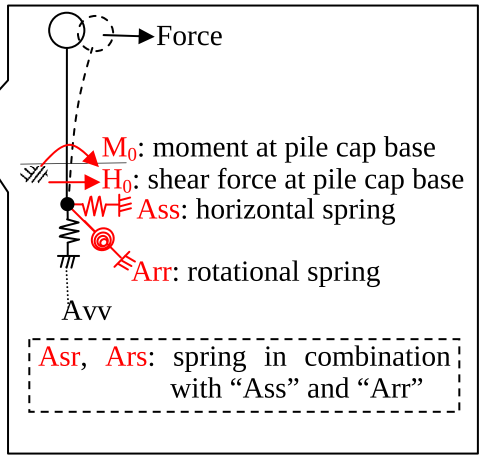
hp: column height (m); hp= 10.0

[Step-2. Lateral displacement of pile cap, δo]

$$\delta o = \frac{H_0 * Arr - M_0 * Asr}{Ass * Arr - Asr * Ars}$$

$$= \frac{5064 * 13377200 - 48407 * (-2078227)}{1562827 * 13377200 - (-2078227) * (-2078227)}$$

= 0.010 (m)



Where,

Ho: horizontal force at the pile cap base (kN);

Ho= Wu+0.8*(Wp+Wf)

= 2900+0.8*(1269+1436)

= 5064

Mo: bending moment at the pile cap base (kN*m);

Mo= Wu*ho+0.8*Wp*(0.5*hp+hf)+0.8*Wf*0.5*hf

= 2900*13.9+0.8*1269*(0.5*10.0+1.9)+0.8*1436*0.5*1.9

= 48407

ho: height from pile cap base to application point of seismic inertial force (m); ho= 13.9

hf: height of the pile cap (m); hf= 1.9

Wf: weight of the pile cap (kN); Wf= 1436

Ass= 1567010 (kN/m)

Asr, Ars= -2081934 (kN/rad)

Arr= 13382127 (kN*m/rad)

[Step-3. Lateral displacement of pile cap due to rotation of the structure, “θo*ho”]

θo*ho= 0.005*13.9

= 0.070 (m)

Where,

$$\theta o = \frac{-H_0 * Ars + M_0 * Ass}{Ass * Arr - Asr * Ars}$$

$$= \frac{-5064 * (-2078227) + 48407 * 1562827}{1562827 * 13377200 - (-2078227) * (-2078227)}$$

= 0.005 (m)

[Step-4. Calculation of total displacement, “δ”]

δ= δp+δo+θo*ho

= 0.150+0.010+0.070

= 0.230 (m)

[Step-5. Calculation of natural period]

$$T = 2.01\sqrt{\delta}$$
$$= 2.01\sqrt{0.230}$$
$$= 0.96 \text{ (s)}$$

[Step-6. Determination of design seismic coefficient]

Csm= 0.64/Tm

= 0.64/0.96

= 0.66 (g)

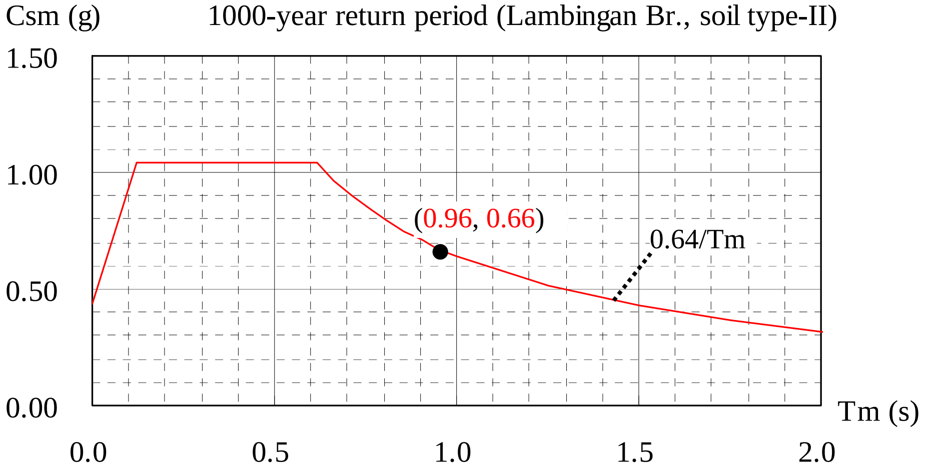


Figure 2.4.2-3 Design Seismic Coefficient for Transverse Direction

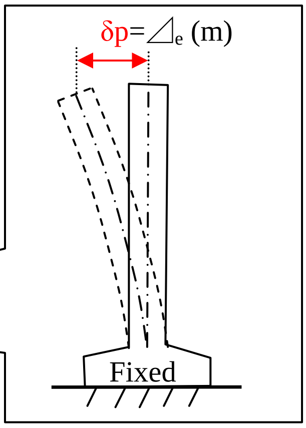
2.5 Determination of Design Displacement and Design Seat Length

2.5.1 Determination of Design Displacement

Design displacement, ⊿ is calculated as follows, based on the result of bridge analysis in transverse direction which generated larger displacement.

[Step-1. Calculation of Column Displacement (transverse direction)]

$$\delta_p = \frac{H * h^3}{3 * E * Ic} + \frac{0.8 * W_p * h_p^3}{8 * E * Ic}$$
$$= \frac{1914 * 12^3}{3 * 25000000 * 0.477} + \frac{0.8 * 1269 * 10^3}{8 * 25000000 * 0.477}$$
$$= 0.103 \text{ (m)} = ⊿_e$$



Where,

H: seismic inertial force from the superstructure (kN); H= Wu*Csm= 2900*0.69= 1914

Wu: superstructure weight undertaken by the pier under EQ; Wu= 2900

Csm: seismic coefficient (g); Csm= 0.69 (bridge analysis result)

[Step-2. Calculation of Design Displacement, ⊿]

⊿= 12*R_d*⊿$_e$

= 12*1.0*0.103

= 1.24 (m)

Where,

R_d= 1.0 (T= 0.96> 1.25*Ts= 0.769)

⊿$_e$: Displacement calculated from elastic seismic analysis (m)

T: Period of fundamental mode of vibration (sec.)

Ts: Corner period specified in BSDS Article 3.6.2 (sec.)

R: R-factor; R= 2.0

Pu: Axial load on column or pier (kN); Pu= Rd+0.5*Rl= 3800

φ: Flexural resistance factor for column; φ= 0.9

Mn: Nominal flexural strength of column or pier calculated at the axial load on the column or pier (kN*m)

2.5.2 Determination of Design Seat Length

The minimum required seat length is calculated as follows.

S_{EM} = 0.7+0.005*L

= 0.7+0.005*30.0

= 0.85 (m)

Where,

L: distance between two substructures for determining the seat length (m) ; L= 30.0

Therefore, coping thickness is designed as the following figure.

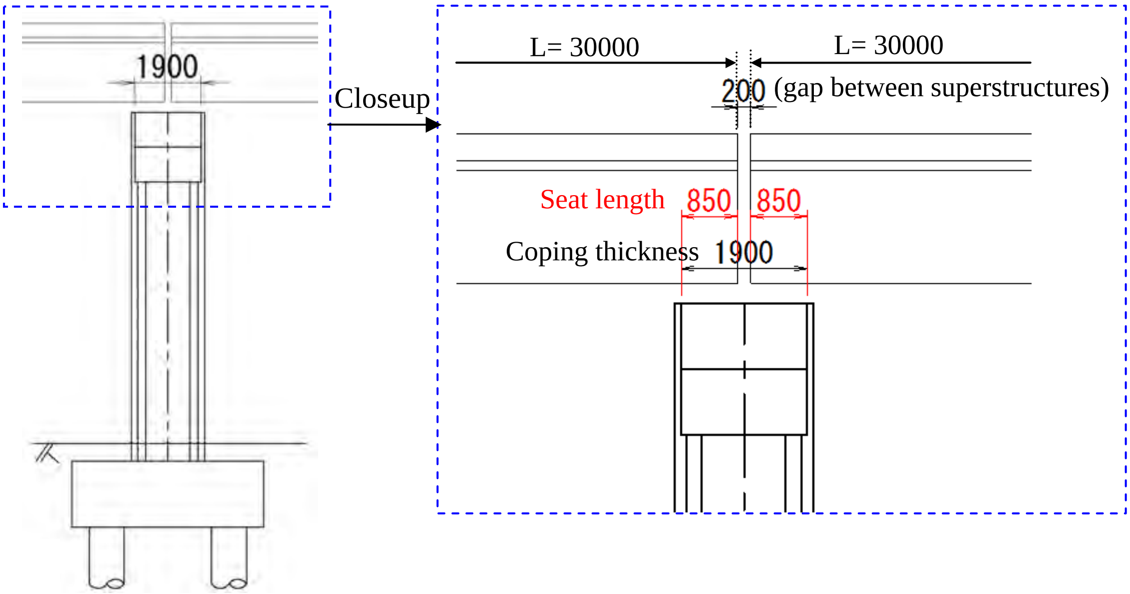


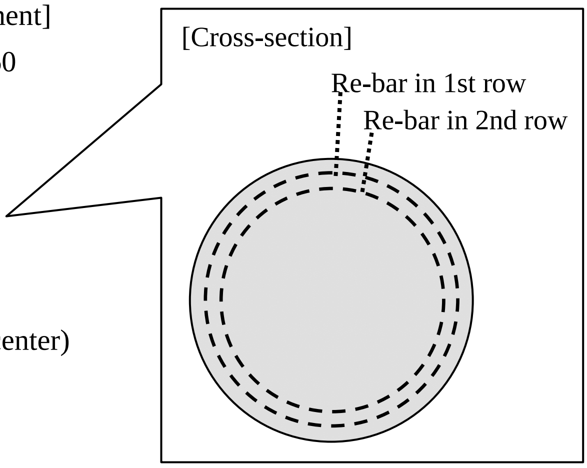
Figure 2.5.2-1 Design Seat Length and Coping Thickness

2.6 Seismic Design of the Column

2.6.1 Design Condition

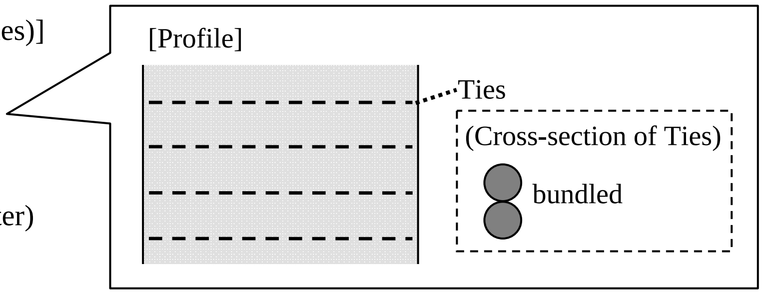
[Longitudinal reinforcement]

- fy= 415 (MPa); Grade60
- Diameter: 32 (mm)
- No. of re-bars
  - 1st row: 48 (nos)
  - 2nd row: 24 (nos)
- Cover meter
  (column edge to rebar center)
  - 1st row: 150 (mm)
  - 2nd row: 250 (mm)



[Transverse reinforcement (ties)]

- fy= 415 (MPa); Grade60
- Diameter: 20 (mm)
- Spacing: 100 (mm)
  (re-bar center to re-bar center)



2.6.2 Column Design Force

[List of forces]

Seismic Forces in Longitudinal dir.

	W	Csm	H=W*Csm	h	M=H*h
	(kN)	-	(kN)	(m)	(kN*m)
Superstructure	2,900	0.690	2,001	10.0	20,010
Pier	1,269	0.690	876	7.0	6,129
	Sum:	V_L=	2,894	M_L=	26,139

Seismic Forces in Transverse dir.

	W	Csm	H=W*Csm	h	M=H*h
	(kN)	-	(kN)	(m)	(kN*m)
Superstructure	2,900	0.660	1,914	12.0	22,968
Pier	1,269	0.660	838	7.0	5,863
	Sum:	V_T=	2,771	M_T=	28,831

Where,

W: weight of structures (kN)

H: horizontal seismic inertial force (kN)

M: bending moment (kN*m)

V_L, V_T: shear force at the column base (kN)

M_L, M_T: bending moment at the column base (kN*m)

Design force for the column design is calculated as follows.

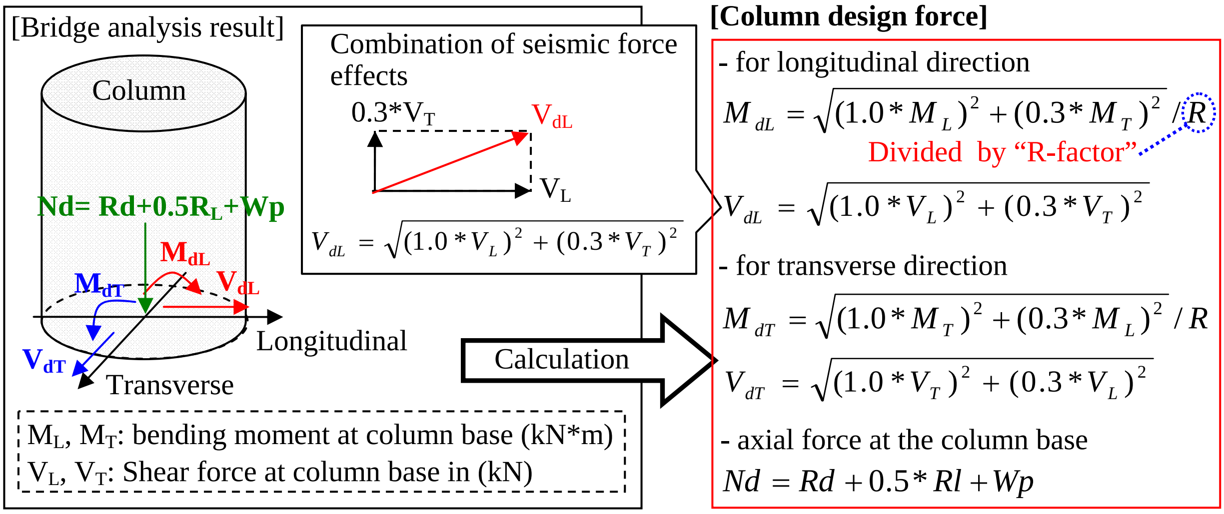


Figure 2.6.2-1 Determination of Column Design Force

[Design force for longitudinal direction]

- Nd: design axial force for N-M interaction diagram (kN);

Nd= Rd+0.5*Rl+Wp

= 2900+0.5*1800+1269

= 5069 (kN)

- Vd: design shear force

$$V_{dL} = \sqrt{(1.0*V_L)^2 + (0.3*V_T)^2}$$
$$= \sqrt{(1.0*2894)^2 + (0.3*2771)^2}$$
$$= 3011 \ (kN)$$

- Md: design bending moment

$$M_{dL} = \sqrt{(1.0*M_L)^2 + (0.3*M_T)^2} / R$$
$$= \sqrt{(1.0*26139)^2 + (0.3*28831)^2} / 2.0$$
$$= 14939 \ (\mathrm{kN*m})$$

[Design force for transverse direction]

- Nd: design axial force for N-M interaction diagram (kN);

Nd= Rd+0.5*Rl+Wp

= 2900+0.5*1800+1269

= 5069 (kN)

- Vd: design shear force

$$V_{dT} = \sqrt{(1.0*V_T)^2 + (0.3*V_L)^2}$$
$$= \sqrt{(1.0*2771)^2 + (0.3*2894)^2}$$
$$= 2903 \ (\mathrm{kN})$$

- Md: design bending moment

$$M_{dT} = \sqrt{(1.0*M_T)^2 + (0.3*M_L)^2} / R$$
$$= \sqrt{(1.0*28831)^2 + (0.3*26139)^2} / 2.0$$
$$= 13767 \ (\mathrm{kN*m})$$

2.6.3 Verification of Flexural Resistance

Flexural capacity of the column is verified against the design force demand as follows.

[Design force]

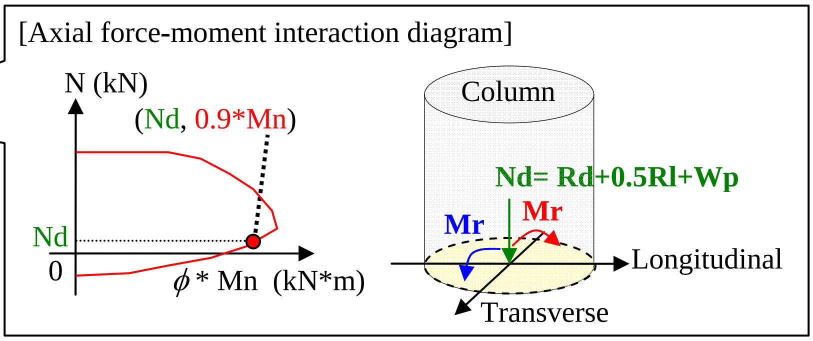
- longitudinal direction: Md= 14939 (kN*m)
- transverse direction: Md= 13767 (kN*m)
- axial force: Nd= 5069 (kN)

[Flexural resistance]

Mr= ϕ *Mn

= 0.9*21618

= 19456 (kN*m)



Where,

Mr: flexural resistance of the column (kN*m)

ϕ : resistance factor; ϕ = 0.9

Mn: nominal flexural resistance (kN*m); Mn= 21618

Note: ϕ *Mn is obtained from "the N-M interaction diagram" shown below

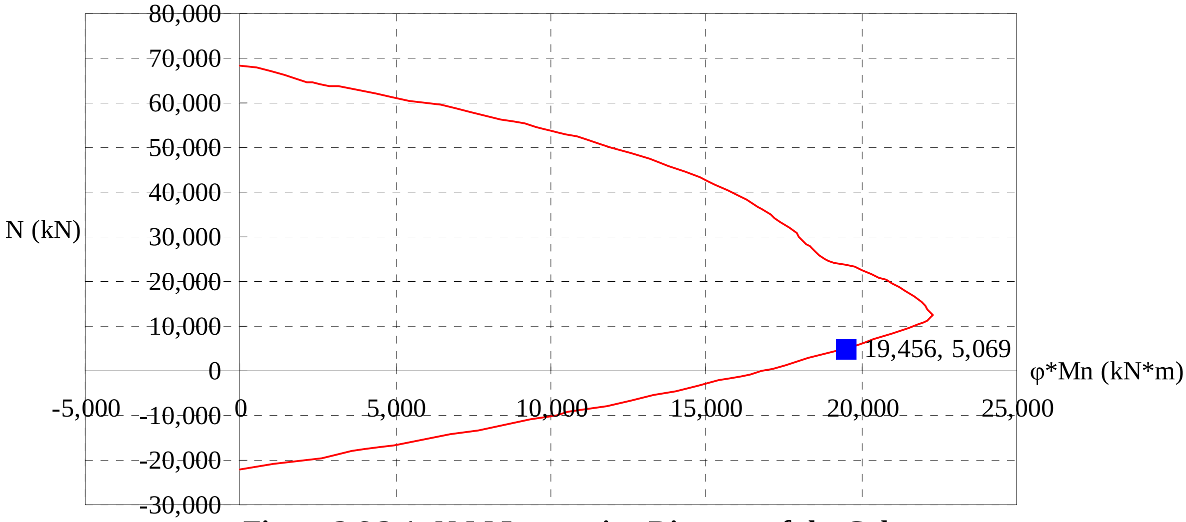


Figure 2.6.3-1 N-M Interaction Diagram of the Column

[Capacity verification]

-for longitudinal direction

Md= 14939 < 19456 (OK)

(0.77) (1.00)

-for transverse direction

Md= 13767 < 19456 (OK)

(0.71) (1.00)

2.6.4 Verification of Column Displacement

Displacement requirement of the column is verified using the following formula.

$$\boxed{\begin{aligned} &\Delta * Pu < 0.25 * \varphi * Mn \\ &\leftrightarrow \Delta < \frac{\varphi * Mn}{4 * Pu} \end{aligned}}$$

[Design Displacement]

$\Delta = 12 * R_d * \Delta_e$

$= 12*1.0*0.103$

$= 1.24$ (m)

Where,

R_d= 1.0 (T= 0.96> 1.25*Ts= 0.769)

Δ_e : Displacement calculated from elastic seismic analysis (m)

[Verification of displacement]

$$\frac{\varphi * Mn}{4 * Pu} = \frac{19456}{4 * 5069}$$

$$= 1.28$$

Therefore,

$\Delta = 1.24 < 1.28$ (m) (OK)

2.6.5 Verification of Shear Resistance

Shear capacity of the column is verified against the design force demand as follows.

[Design force]

- longitudinal direction: Vd= 3011 (kN)
- transverse direction: Vd= 2903 (kN)

[Shear resistance]

- Step-1: calculation of “Vn”

Vn= Vc+Vs

= 0+6345

= 6345 (kN)

Where,

Vn: nominal shear resistance (kN)

Vc: shear strength developed by concrete (kN);

Vc= 0.083*β*$\sqrt{f'c'}$*bv*dv

= 0 (kN) (to be on the conservative side)

Vs: shear strength developed by re-bars (kN);

Vs= (Av*fy*dv*cotθ)/s

= (1256.0*415*cot45)/120

= 6345 (kN)

β: factor indicating ability of diagonally cracked concrete to transmit tension and shear

θ: Angle of inclination of diagonal compressive stress (degrees); θ= 45

f'c’: specified compressive strength of concrete (N/mm2); fc’=28

bv: effective web width (mm); bv= 2100

dv: effective shear span (m);

dv= 0.9*de= 0.9*(D/2+Dr/π)

= 0.9*(2100/2+1800/3.14)

= 1461 (mm)

D: Gross diameter of the column (mm); D= 2100

Dr: Diameter of the circle passing through the centers of longitudinal re-bars (mm); Dr= 1800

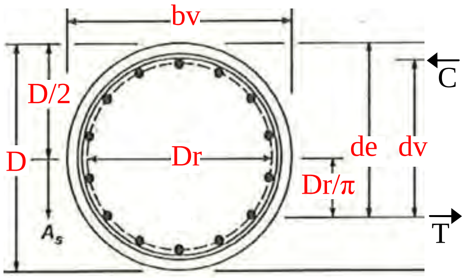
fy: yield strength of transverse reinforcement (N/mm2); fy= 415

Av: area of shear reinforcement within distance "s" (mm2); Av= (314.0*2)*2=1256.0 (diameter: 20)

s: spacing of transverse reinforcement (mm); s= 120

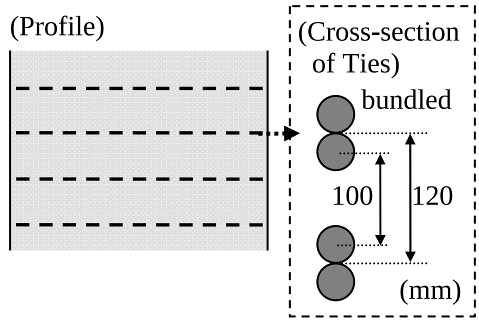
Simplified procedure is applied with the following values;
- β=2.0
- θ=45

[Definition of parameters]



C: compression
T: tension

[Bundling of two re-bars]



- Step-2: determination of “Vr”

Vr= ϕ*Vn

= 0.9*6345

= 5710 (kN)

Where,

Vr: shear resistance of the column (kN)

ϕ: resistance factor; ϕ=0.9

[Capacity verification]
-for longitudinal direction
Vd= 3011 < 5710 (OK)
(0.54) (1.00)
-for transverse direction
Vd= 2903 < 5710 (OK)
(0.52) (1.00)

2.6.6 Verification of Minimum Required Reinforcement

2.6.6.1 Verification of Minimum Required Longitudinal Reinforcement

The longitudinal reinforcement ratio is verified to be not less than 1.0% as shown below.
ρs= Aa/Ag
= 0.059/3.461
= 0.017 > 0.010 (OK)
Where,
ρs: ratio of longitudinal reinforcement to gross area of concrete section
Aa: total area of longitudinal reinforcement (m2); Aa= As*n= $(819/10^6)$*72 = 0.059
As: cross-sectional area of single longitudinal reinforcement bar (mm2); As= 819
n: number of longitudinal reinforcement bar; n= 72
Ag: gross area of concrete section (m2); Ag= $\pi*(D/2)^2$= $3.14*(2.1/2)^2$= 3.461
D: diameter of column (m); D= 2.1

2.6.6.2 Verification of Minimum Required Transverse Reinforcement

Amount of minimum required transverse reinforcement is verified as follows.
[Step-1: calculation of minimum required reinforcement-1]

$$\rho_s \geq 0.45*\left(\frac{A_g}{A_c}-1\right)*\frac{f_c^{'}}{f_{yh}}$$

$$= 0.45*\left(\frac{3.461}{2.543}-1\right)*\frac{28}{415}$$

$$= 0.0110$$

Where,
Ag: gross area of concrete section (m2);
Ag= $\pi*(D/2)^2$
= $3.14*(2.1/2)^2$
= 3.461
Ac: area of core measured to the outside diameter of transverse reinforcement (m2);
Ac= $\pi*(Dr/2)^2$
= $3.14*(1.8/2)^2$
= 2.543

f'c': specified compressive strength of concrete (N/mm2); fc'=28

fyh: yield strength of transverse reinforcement (N/mm2); fy= 415

[Step-2: calculation of minimum required reinforcement-2]

$$\rho_s \geq 0.12 * \frac{f_c^{'}}{f_{yh}}$$

$$= 0.12 * \frac{28}{415}$$

$$= 0.0081$$

[Step-3: calculation of "ρs" and verification of minimum required reinforcement]

$$\rho_s = \frac{\pi * D_r * (A_v / 2)}{\pi * (D_r / 2)^2 * s} = \frac{4 * (A_v / 2)}{D_r * s}$$

$$= \frac{4 * (1256.0 / 2)}{1800 * 120}$$

$$= 0.0116 > 0.0110 \ (OK)$$

Where,

ρs: ratio of transverse reinforcement to total volume of concrete core

s: spacing of transverse reinforcement (mm); s= 120

Av: area of shear reinforcement within distance "s" (mm2); Av= 1256.0 (=314.0*4, As= 314.0)

Note: bundling of two re-bars is applied.

Dr: diameter of the circle passing through the centers of longitudinal re-bars (mm); Dr= 1800

2.7 Seismic Design of the Pile Foundation

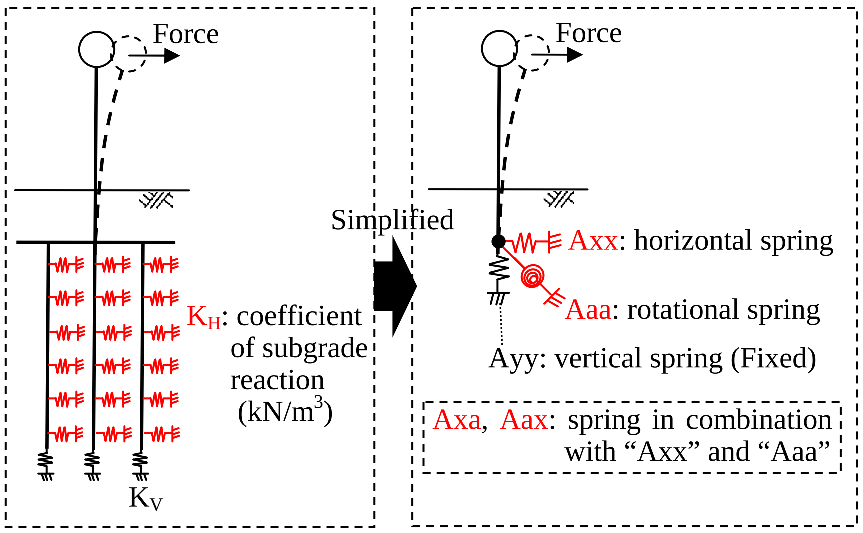
2.7.1 Spring Properties of the Pile Foundation

[Step-0. Preliminary calculation of design parameters]

Design parameters for pile foundation springs are summarized as follows.

Layer symbol	Layer type	Layer thickness Li (m)	N-value	α*Eo (kN/m2)		γ (kN/m3)		fi (kN/m2)	DE
				Normal (α=4)	EQ (α=8)	γt	γ'		
As	Sand	3.50	11	30,800	61,600	18.0	9.0	55	0.667
Ac	Cray	4.00	7	19,600	39,200	16.0	7.0	70	-
WGF	Rock	1.00	28	78,400	156,800	18.0	9.0	150	-
GF	Rock	10.00	50	140,000	280,000	22.0	13.0	150	-

Pile foundation springs can be determined with the above parameters.



[Calculation example]

For layer "As",
E_0= 2800*(N-value)
= 2800*11
= 30800 (kN/m2)
Therefore,
α*E_0 (under normal)
= 2*30800
= 61600 (kN/m2)

Note:
Considering "DE",
E_0*DE
= 30800*0.667
= 20544 (kN/m2)

Where,

α: coefficient for the estimation of subgrade reaction coefficient;

α= 1 (Normal state), 2 (Under earthquake)

E_0: modulus of deformation (kN/m2); E_0= 2800*(N-value)

γt: unit weight of soil (kN/m3)

γ': effective unit weight of soil (kN/m3); γ'= γt - γ_w

γ_w: unit weight of water (kN/m3)

fi: maximum shaft resistance of i-th layer considering pile shaft resistance (kN/m2)

DE: soil coefficient reduction factor obtained from liquefaction potential assessment

F_L	X (m)	DE	
		R≦0.3	0.3<R
F_L≦1/3	0≦X≦10	0	1/6
	10<X≦20	1/3	1/3
1/3<F_L≦2/3	0≦X≦10	1/3	2/3
	10<X≦20	2/3	2/3
2/3<F_L≦1	0≦X≦10	2/3	1
	10<X≦20	1	1

→ DE=2/3

[Step-1. Assumption of "1/β" and initial calculation of "B_H"]

Typically, "1/β" for foundation design is 3-6 times as large as the pile diameter. Therefore, the parameter "β" is initially assumed as "4"; $(1/\beta_1)= 4.0$

Then, equivalent loading width of foundation, "B_H", is initially determined with the following equation.

$B_H = \sqrt{D/\beta 1}$

$= \sqrt{1.0*4.0}$

$= 2.0$ (m)

Where,

D: pile diameter (m); D= 1.0

[Step-2. Initial calculation of "K_H"]

Coefficient of subgrade reaction, K_H, is calculated as follows. In the calculation process, average value of "$\alpha*E_0$" within "1/β" range, "$(\alpha*E_0)_\beta$", should be applied, focusing on the effective range of "K_H". The image of relationship among parameters is illustrated in the following figure.

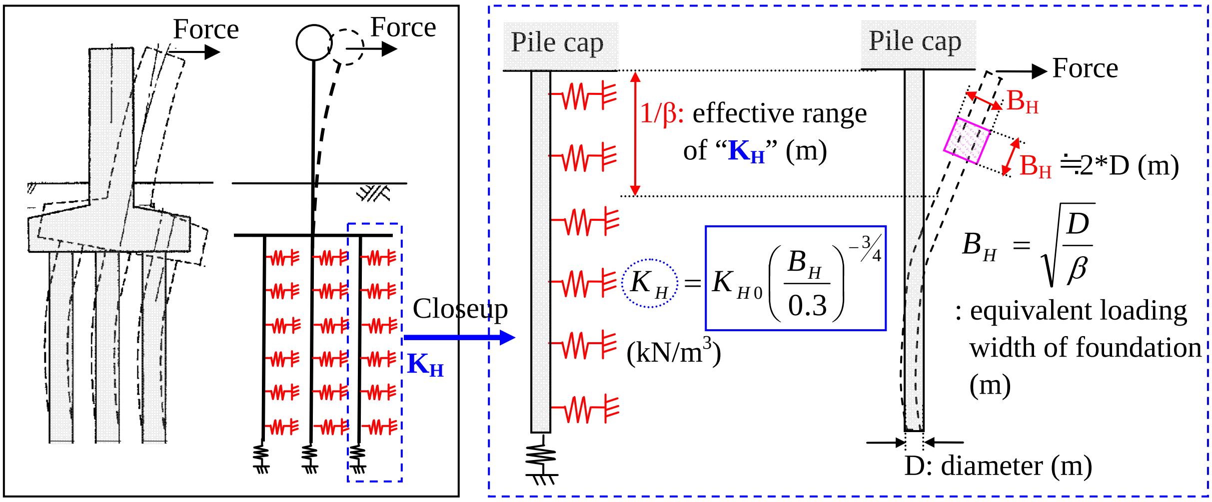


Figure 2.7.1-1 Coefficient of Subgrade Reaction, "K_H"

$$(\alpha * E_0)_\beta = \frac{\sum(\alpha * E_0 * DE)i * Li}{1/\beta}$$

$= \Sigma(20544*3.5+19600*0.5)/4.0$

$= 20426$ (kN/m2)

Then,

$K_{H0}= (1/0.3)*(\alpha*E_0)_\beta$

$= (1/0.3)* 20426$

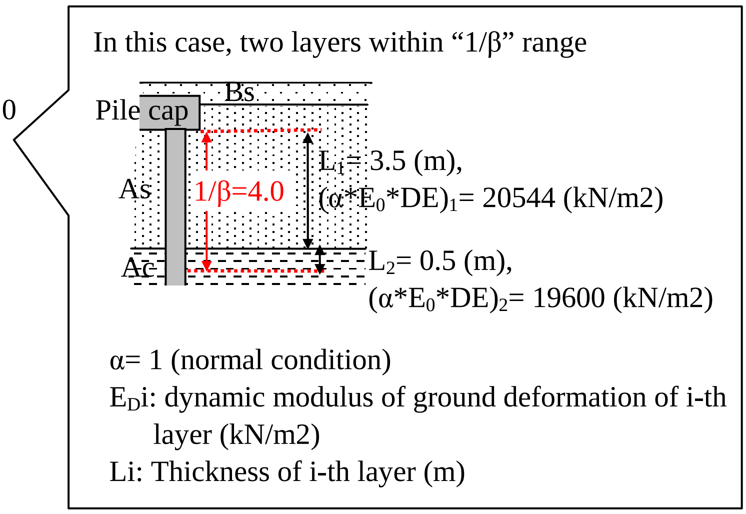
$= 68086$ (kN/m3)

Therefore,

$K_H= K_{H0}*(B_H/0.3)^{-3/4}$

$= 68086*(2.0/0.3)^{-3/4}$

$= 16411$(kN/m3)



[Step-3. Initial calculation of "β"]

The pile specific parameter, "β", is initially calculated as follows.

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

$$= \sqrt[4]{\frac{16411 * 1.0}{4 * 25000000 * 0.049}}$$

$$= 0.241\ (m^{-1})$$

Then,

(1/β)= 1/0.241

= 4.157 (m)

Where,

E: Young's modulus of elasticity; E=25000000 (kN/m2)

I: moment of inertia of a pile;

$I = \pi*(D/2)^4/4$

$= 3.14*(1/2)^4/4$

= 0.049 (m4)

[Step-4. Comparison of "1/β" and "$1/\beta_1$"]

The value of initially assumed "$1/\beta_1$" must be compared with value of "1/β". In this case, "$1/\beta_1$" is 4.0 while "1/β" is 4.157. If the two values are different, process of step-1-4 shall be iterated using resulted "1/β" replaced with "$1/\beta_1$" until "$1/\beta_1$" becomes equal to "1/β". The value of "1/β" is finalized as follows.

[Step-1: Assumption of "1/β" and calculation of "B_H"]

(1) $(1/\beta_1)$= 4.176

(2) $B_H = \sqrt{D/\beta_1}$ =2.044 (m)

[Step-2: Calculation of "K_H"]

(1) $(\alpha * E_0)_\beta = \dfrac{\sum(\alpha * E_0 * DE)i * Li}{1/\beta}$ = 20390 (kN/m2)

(2) K_{H0}= $(1/0.3)*(\alpha*E_0)_\beta$ = 67970

(3) $K_H = K_{H0}*(B_H/0.3)^{-3/4}$ = 16117 (kN/m3)

[Step-3: Calculation of "β"]

(1) $\beta = \sqrt[4]{\dfrac{K_H * D}{4 * E * I}}$ = 0.239 (m^{-1})

(2) (1/β) = 4.176 (m) = $(1/\beta_1)$ (Finalized)

[α= 4; Service Limit State]

K_{H0}= $(1/0.3)*(\alpha*E_0)_\beta$ = 67970 (kN/m3)

$K_H = K_{H0}*(B_H/0.3)^{-3/4}$ = 16117 (kN/m3)

[Step-4: Calculation of **β'** for Seismic Design]

(1) $K_H' = 2*K_H$= 2*16117= 32235 (kN/m3)

(2) $\beta' = \sqrt[4]{\dfrac{K_H' * D}{4 * E * I}}$ = 0.285 (m^{-1})

[α= 8; Extreme Event Limit State]

$K_H' = 2*K_H$

[Step-5. Calculation of spring properties of single pile]

Without pile projection over the ground surface, spring properties of single pile can be determined as follows.

- Spring property of single pile in horizontal/rotational direction

$$
\begin{aligned}
K1 &= 4*E*I*\beta'^3 \\
&= 4*25000000*0.049*0.285^3 \\
&= 113431 \text{ (kN/m)} \\
K2 = K3 &= 2*E*I*\beta'^2 \\
&= 2*25000000*0.049*0.285^2 \\
&= 1999001 \text{ (kN/rad)} \\
K4 &= 2*E*I*\beta' \\
&= 2*25000000*0.049*0.285 \\
&= 698250 \text{ (kN*m/rad)}
\end{aligned}
$$

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.

- Spring property of single pile in vertical direction

$$
\begin{aligned}
Kv &= a*Ap*E/L \\
&= 0.160*0.785*25000000/10.0 \\
&= 314000 \text{ (kN/m)}
\end{aligned}
$$

Where,

Kv: axial spring constant of a pile

a: modification factor; for CIP pile,

$$
\begin{aligned}
a &= 0.031*(L/D)-0.15 \\
&= 0.031*(10.0/1.0)-0.15 \\
&= 0.160
\end{aligned}
$$

L: pile length (m); L= 10.0

D: pile diameter (m); D= 1.0

Ap: net cross-sectional area of a pile (m2);

$$
\begin{aligned}
Ap &= \pi*(D/2)^2 \\
&= 3.14*(1.0/2)^2 \\
&= 0.785 \text{ (m2)}
\end{aligned}
$$

[Step-6. Calculation of the entire pile foundation structure]

If the effect of foundation on analyses is focused on "1/β" range, which is the effective range of "K_H", foundation structure can be modeled as group of springs in one node as shown in the following figure.

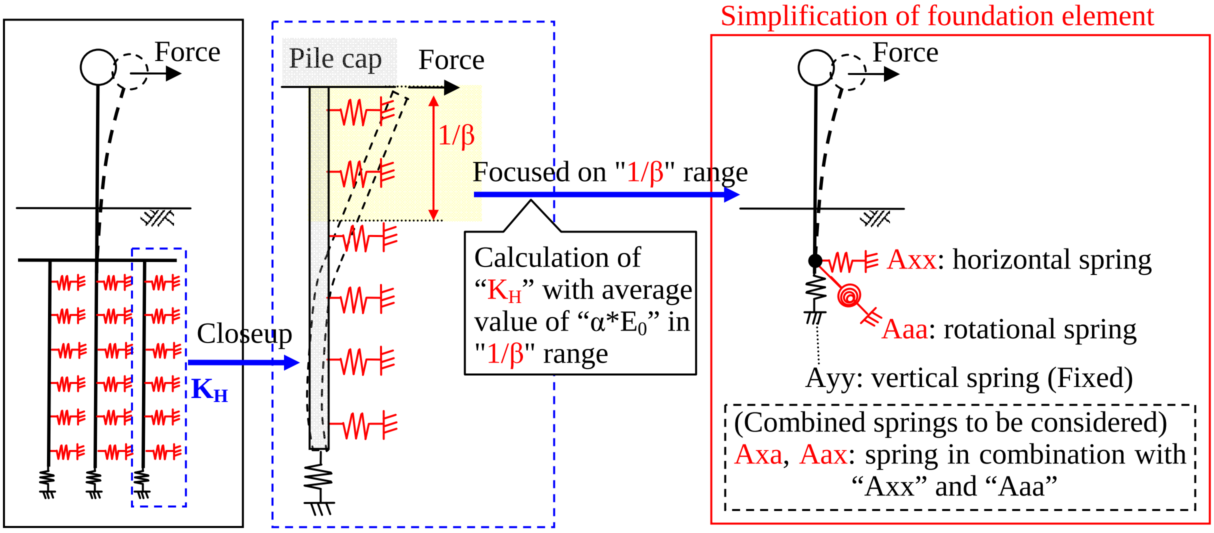
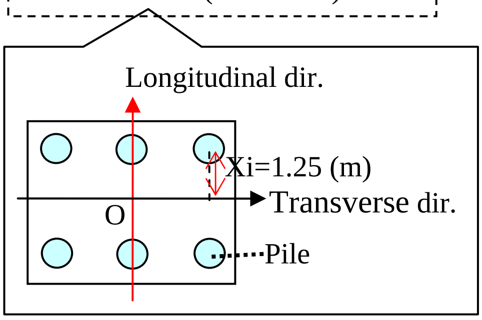


Figure 2.7.1-2 Modeling of Pile Foundation

Therefore, the spring properties of the entire pile foundation are calculated as follows.

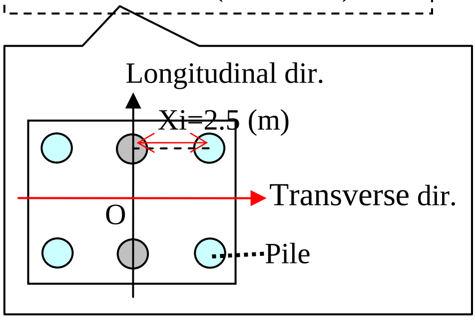
- For longitudinal direction

Axx= n*K1
= 6*113431
= 680586 (kN/m)
Axa=Aax= -n*K2
= -6*199001
= -1194008 (kN/rad)
Aaa= n*K4+KvΣXi^2
= n*K4+Kv*Xi^2*n
= 6*698250+314000*1.25^2*6
= 7133250 (kN*m/rad)



- For transverse direction

Axx= n*K1
= 6*113431
= 680586 (kN/m)
Axa=Aax= -n*K2
= -6*199001
= -1194008 (kN/rad)
Aaa= n*K4+KvΣXi^2
= n*K4+Kv*Xi^2*n
= 6*698250+314000*2.5^2*4
= 12039500 (kN*m/rad)



Where,

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

Axa, Aax: spring properties of the foundation structure in combination with "A_{xx}" and "A_{aa}" (kN/rad)

Aaa: 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)

2.7.2 Foundation Design Force

[List of forces]

Rd+0.5Rl= 3800 (kN)

As=Fpga*PGA= 0.44 (g)

Wp= 1269 (kN); weight of the column

Wf= 1436 (kN); weight of the pile cap

Mu= 19456 (kN*m): flexural resistance of the column

Design force for the foundation design is calculated as follows.

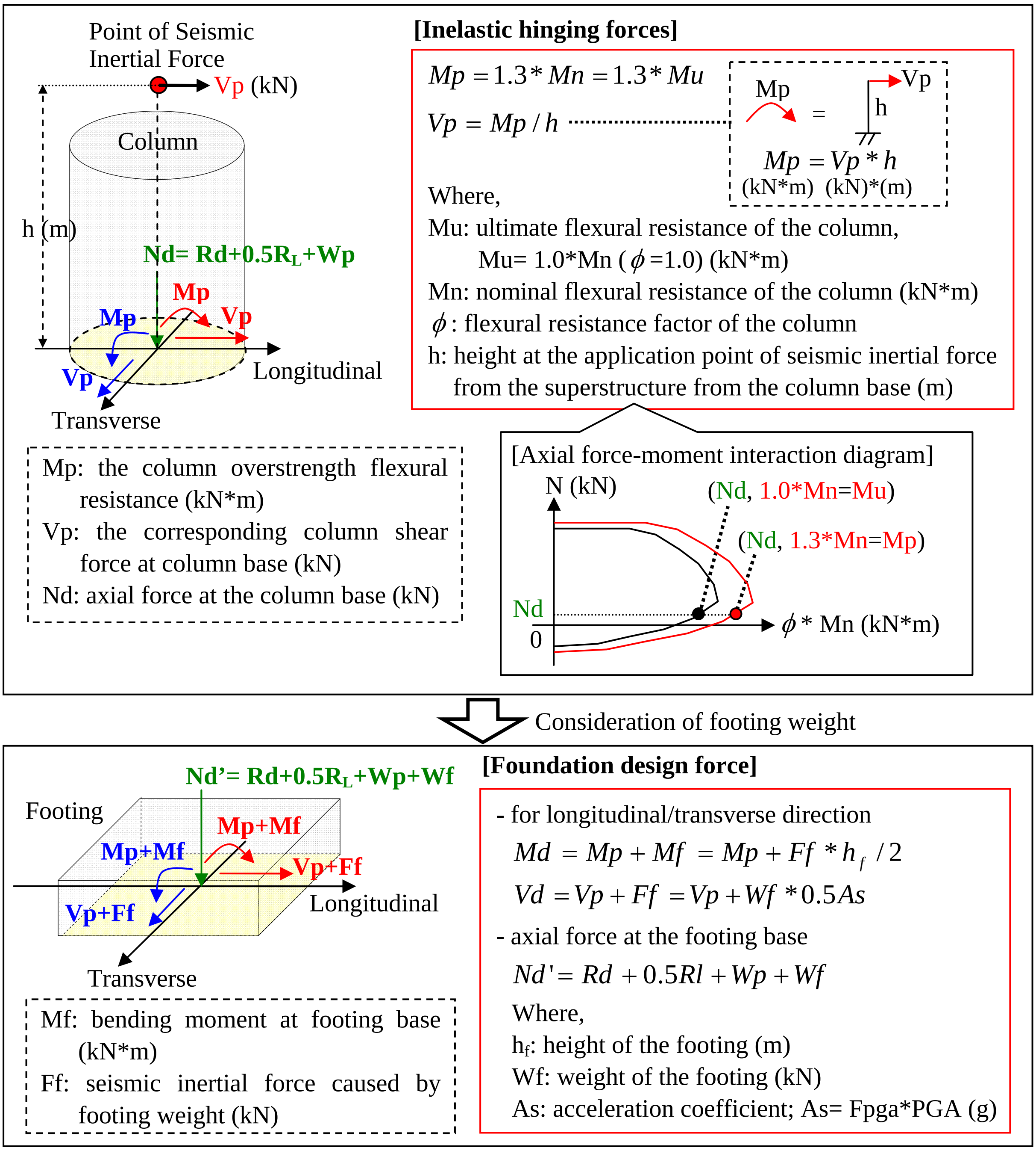


Figure 2.7.2-1 Determination of Foundation Design Force

[Inelastic hinging forces]

Mp: the column overstrength moment resistance (kN*m);

Mp= 1.3*Mu

= 1.3*19456

= 25293 (kN*m)

Vp: the column overstrength shear resistance (kN);

(for longitudinal direction)

Vp= Mp/h

= 25293/10.0

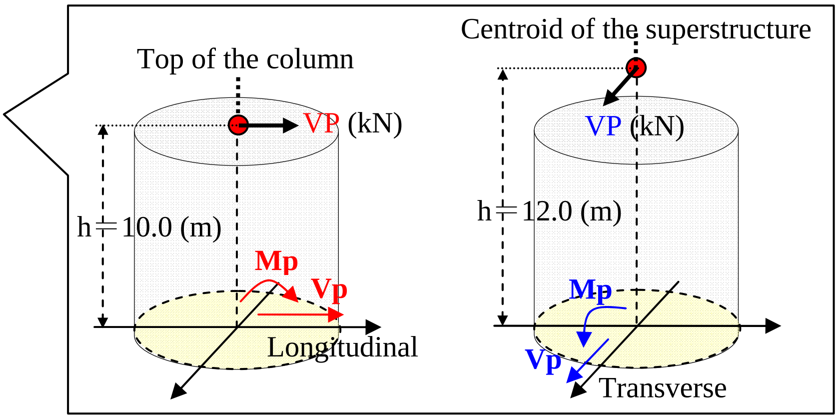
= 2529 (kN)

(for transverse direction)

Vp= Mp/h

= 25293/12.0

= 2108 (kN)



[Seismic inertial force of the footing]

Ff= Wf*(0.5*As)

= 1436*(0.5*0.44)

= 316 (kN)

Mf= Ff*hf/2

= 316*0.95

= 300 (kN*m)

Where,

hf: height of the footing (m); hf= 1.9

[Design force for longitudinal direction]

- Nd: design axial force (kN);

Nd= Rd+0.5*Rl+Wp+Wf+Wso

= 2900+0.5*1800+1269+1436+267

= 6772 (kN)

- Vd: design shear force (kN);

Vd= Vp+Ff

= 2529+316

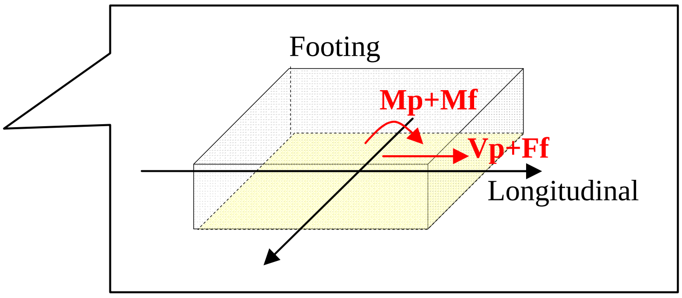
= 2845 (kN)

- Md: design bending moment (kN*m)

Md= Mp+Mf

= 25293+300

= 25593 (kN*m)



[Design force for longitudinal direction]

- Nd: design axial force (kN);

  Nd= Rd+0.5*Rl+Wp+Wf+Wso

  = 2900+0.5*1800+1269+1436+267

  = 6772 (kN)

- Vd: design shear force (kN);

  Vd= Vp+Ff

  = 2108+316

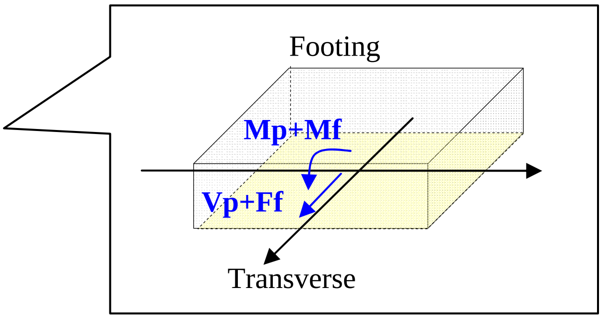
  = 2424 (kN)

- Md: design bending moment (kN*m)

  Md= Mp+Mf

  = 25293+300

  = 25593 (kN*m)



2.7.3 Reaction Force and Displacement of Each Pile

2.7.3.1 Reaction Force and Displacement of Each Pile for Longitudinal Direction

[Step-1: Calculation of “displacement and rotation” of the pile cap]

Displacement of each pile is calculated as follows.

Axx*δx+Axy*δy+Axa*α = Vd

Ayx*δx+Ayy*δy+Aya*α = Nd

Aax*δx+Aay*δy+Aaa*α = Md

[Unknown parameters]

δx: lateral displacement (m)

δy: vertical displacement (m)

α: rotational angle (degrees)

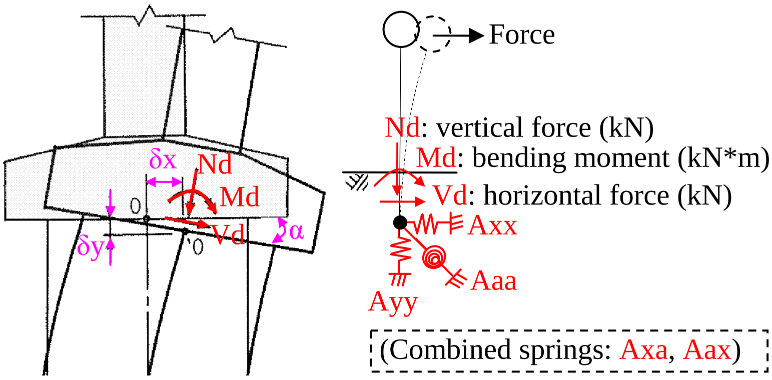


Figure 2.7.3-1 Reaction Force and Displacement of Each Pile

- δx: lateral displacement at the origin “O” (m);

$$\delta_x = \frac{V_d * A_{aa} - M_d * A_{xa}}{A_{xx} * A_{aa} - A_{xa} * A_{ax}}$$

$$= \frac{2845 * 7133250 - 25593 * (-1194008)}{680586 * 7133250 - (-1194008) * (-1194008)}$$

$$= 0.0148 \text{ (m)}$$

- δy: vertical displacement at the origin “O” (m);

$$\delta_y = \frac{N_d}{A_{yy}}$$

$$= \frac{6772}{1884000}$$

$$= 0.00359 \text{ (m)}$$

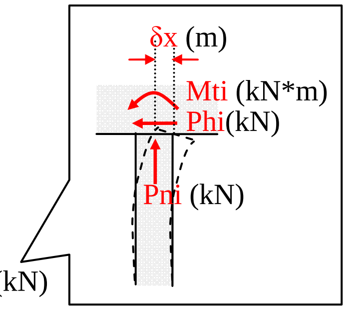
- α: rotational angle of the footing (degrees)

$$\alpha = \frac{-V_d * A_{ax} + M_d * A_{xx}}{A_{xx} * A_{aa} - A_{xa} * A_{ax}}$$
$$= \frac{-2845 * (-1194008) + 25593 * 680586}{680586 * 7133250 - (-1194008) * (-1194008)}$$
$$= 0.00607 \text{ (rad)}$$

[Step-2: Calculation of reaction force at each pile head]

Reaction forces at each pile head

Row	Xi (m)	Pni (kN)	Phi (kN)	Mti (kN*m)
1	-1.250	-1255.2	470.8	1293.2
2	1.250	3509.7	470.8	1293.2



Where,

Pni= Kv*(δy+α*Xi): axial force acting on "pile heads in i-th row" (kN)

Phi= K1*δx - K2*α: horizontal force acting on "pile heads in i-th row" (kN)

Mti= -K3*δx + K4*α: bending moment acting on "pile heads in i-th row" (kN*m)

(Calculation example; piles in the 2nd row)

- Pn_2: axial force acting on "pile head in the 2nd row" (kN);

  Pn_2 = Kv*(δy+α*X_2)

  = 314000*(0.00359+0.00607*1.25)

  = 3509.7 (kN)

- Ph_2: horizontal force acting on "pile head in the 2nd row" (kN);

  Ph_2 = K1*δx - K2*α

  = 113431*0.0148-199001*0.00607

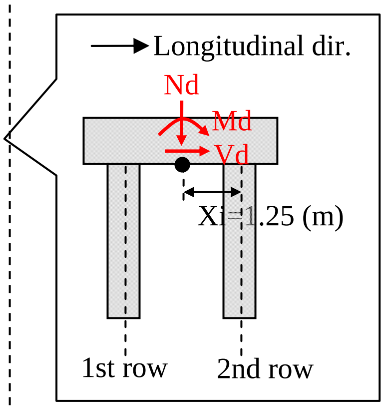
  = 470.8 (kN)

- Mt_2: bending moment acting on "pile head in the 2nd row" (kN*m);

  Mt_2 = -K3*δx + K4*α

  = -199001*0.0148+698250*0.00607

  = 1293.2 (kN*m)



[Step-3: Graphing of “reaction force and displacement” of each pile]

Reaction force and displacement of each pile is graphed as follows.

1) Rigid-pile-head connection

$$y = \frac{P_{hi}}{2 * E * I * \beta^3} e^{-\beta * X} \{(1 + \beta * h_0) * \cos(\beta * X) - \beta * h_0 * \sin(\beta * X)\}$$

: horizontal displacement of a pile (m)

$$M = \frac{-P_{hi}}{\beta} e^{-\beta * X} \{\beta * h_0 * \cos(\beta * X) - (1 + \beta * h_0) * \sin(\beta * X)\}$$

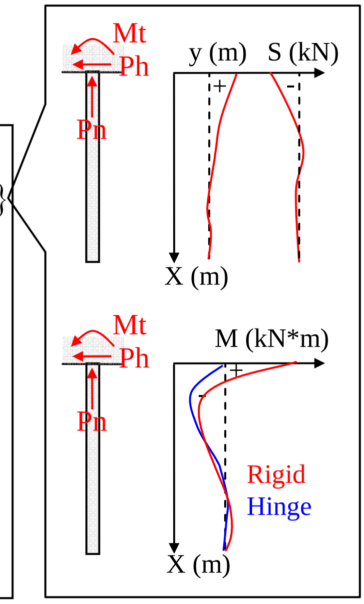
: bending moment of a pile (kN*m)

$$S = -P_{hi} * e^{-\beta * X} \{\cos(\beta * X) - (1 + 2 * \beta * h_0) * \sin(\beta * X)\}$$

: shear force of a pile (kN)

2) Hinge-pile-head connection

$$M = \frac{P_{hi}}{\beta} e^{-\beta * X} \sin(\beta * X)$$: bending moment of a pile (kN*m)



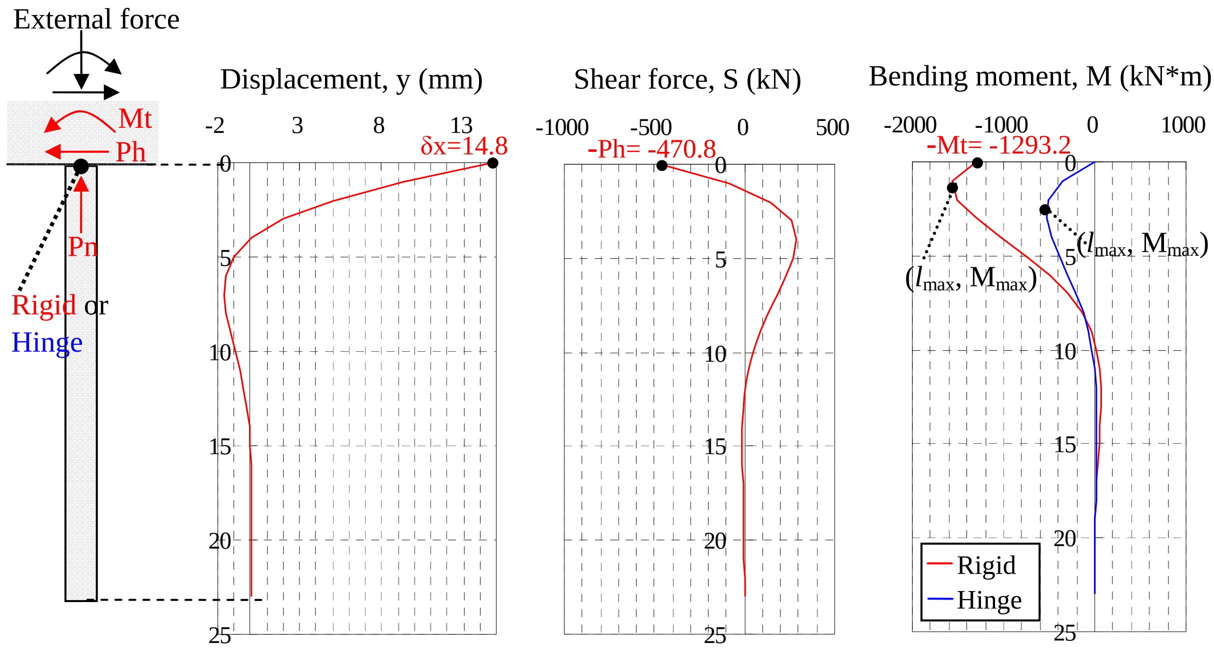


Figure 2.7.3-2 Reaction Force and Displacement of the Single Pile

(Note)

- Maximum moment under the ground for rigid-pile-head connection, M_{max} (kN*m);

$$M_{\max} = \frac{-P_{hi}}{2*\beta}\sqrt{(1+2*\beta*h_0)^2+1}*e^{-\beta*lm}$$

$$= \frac{-470.8}{2*0.285}\sqrt{(1+2*0.285*2.746)^2+1}*e^{-0.285*1.304}$$

$$= -1568.4\ (kN*m)$$

$$l_m = \frac{1}{\beta}\tan^{-1}\left(\frac{1}{1+2*\beta*h_0}\right)$$

$$= \frac{1}{0.285}\tan^{-1}\left(\frac{1}{1+2*0.285*2.746}\right)$$

$$= 1.304\ (m)$$

$h_0 = M_t / P_{hi}$: converted pile projection length (m)

= 1293.2/470.8

= 2.746 (m)

- Maximum moment under the ground for hinge-pile-head connection, M_{max} (kN*m);

$$M_{\max} = \frac{-P_{hi}}{\beta}e^{-\beta*lm}*\sin(\beta*l_m)$$

$$= \frac{-470.8}{0.285}e^{-0.285*2.756}*\sin(0.285*2.756)$$

$$= -532.6\ (kN*m)$$

$$l_m = \frac{\pi}{4*\beta}$$

$$= \frac{3.14}{4*0.285}$$

$$= 2.756\ (m)$$

2.7.3.2 Reaction Force and Displacement of Each Pile for Transverse Direction

[Step-1: Calculation of "displacement and rotation" of the pile cap]

- δx: lateral displacement at the origin "O" (m);

$$\delta_x = \frac{V_d * A_{aa} - M_d * A_{xa}}{A_{xx} * A_{aa} - A_{xa} * A_{ax}}$$

$$= \frac{2424 * 7133250 - 25593 * (-1194008)}{680586 * 12039500 - (-1194008) * (-1194008)}$$

$$= 0.0088 \text{ (m)}$$

- δy: vertical displacement at the origin "O" (m);

$$\delta_y = \frac{N_d}{A_{yy}}$$

$$= \frac{6772}{1884000}$$

$$= 0.00359 \text{ (m)}$$

- α: rotational angle of the footing (degrees)

$$\alpha = \frac{-V_d * A_{ax} + M_d * A_{xx}}{A_{xx} * A_{aa} - A_{xa} * A_{ax}}$$

$$= \frac{-2424 * (-1194008) + 25593 * 680586}{680586 * 12039500 - (-1194008) * (-1194008)}$$

$$= 0.003 \text{ (rad)}$$

[Step-2: Calculation of reaction force at each pile head]

Reaction forces at each pile head

Row	Xi (m)	Pni (kN)	Phi (kN)	Mti (kN*m)
1	-2.500	-1227.7	401.2	343.5
3	2.500	3482.3	401.2	343.5

Where,

Pni= Kv*(δy+α*Xi): axial force acting on "pile heads in i-th row" (kN)

Phi= K1*δx - K2*α: horizontal force acting on "pile heads in i-th row" (kN)

Mti= -K3*δx + K4*α: bending moment acting on "pile heads in i-th row" (kN*m)

[Step-3: Graphing of "reaction force and displacement" of each pile]

Reaction force and displacement of each pile is graphed as follows.

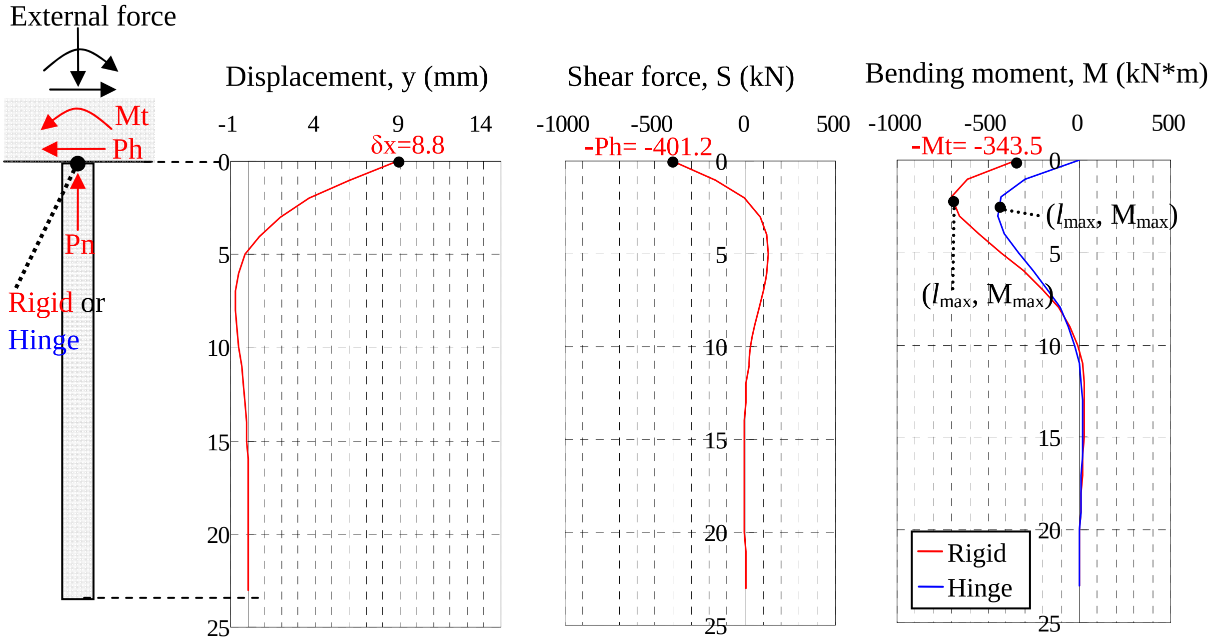


Figure 2.7.3-3 Reaction Force and Displacement of the Single Pile

(Note)

- Maximum moment under the ground for rigid-pile-head connection, M_{max} (kN*m);

$$M_{max} = \frac{-P_{hi}}{2*\beta}\sqrt{(1+2*\beta*h_0)^2+1}*e^{-\beta*lm}$$

$$= \frac{-343.5}{2*0.285}\sqrt{(1+2*0.285*0.856)^2+1}*e^{-0.285*2.076}$$

$$= -698.3 \text{ (kN * m)}$$

$$l_m = \frac{1}{\beta}\tan^{-1}\left(\frac{1}{1+2*\beta*h_0}\right)$$

$$= \frac{1}{0.285}\tan^{-1}\left(\frac{1}{1+2*0.285*0.856}\right)$$

$$= 2.076 \text{ (m)}$$

$h_0 = M_t / P_{hi}$: converted pile projection length (m)

$$= 343.5 / 401.2$$

$$= 0.856 \text{ (m)}$$

- Maximum moment under the ground for hinge-pile-head connection, M_{max} (kN*m);

$$M_{max} = \frac{-P_{hi}}{\beta}e^{-\beta*lm}*\sin(\beta*l_m)$$

$$= \frac{-401.2}{0.285}e^{-0.285*2.756}*\sin(0.285*2.756)$$

$$= -453.8 \text{ (kN * m)}$$

$$l_m = \frac{\pi}{4*\beta}$$

$$= \frac{3.14}{4*0.285}$$

$$= 2.756 \text{ (m)}$$

2.7.4 Stability Verification

2.7.4.1 Parameters for Stability Verification

[Verification parameters for longitudinal direction]

- Lateral displacement; δx= 14.8 (mm)
- Push-in force; $(P_{ni})_{max}$= 3509.7 (kN)
- Pull-out force; $(P_{ni})_{min}$= -1255.2 (kN)

[Verification force for transverse direction]

- Lateral displacement; δx= 8.8 (mm)
- Push-in force; $(P_{ni})_{max}$= 3482.3 (kN)
- Pull-out force; $(P_{ni})_{min}$= -1227.7 (kN)

2.7.4.2 Bearing Capacity and Capacity against Axial Pull-out Force

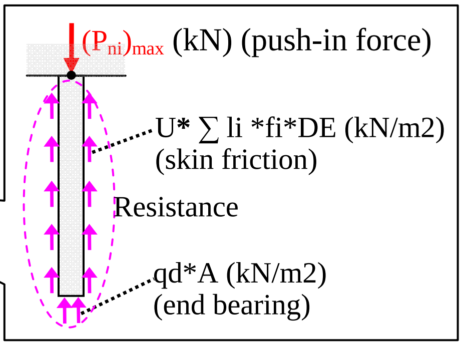
[Bearing capacity]

R_R: bearing capacity against push-in force of single pile (kN);

R_R= γ*(ϕ *Rn-Ws)+Ws-W

= 0.65*(1.0*6337.8- 68.1)+ 68.1- 109.9

= 4009.6 (kN)



Where,

γ: modification coefficient depending on nominal bearing resistance estimation method; γ= 1.0

ϕ : resistance factor for bearing capacity under extreme event limit state; ϕ = 0.65

Rn: nominal resistance of pile (kN);

Rn= qd*A+U* ∑ li *fi*DE

= 5000*0.785+2412.8

= 6337.8 (kN)

qd: ultimate end bearing capacity intensity (kN/m2); qd= 5000

A: sectional area of a pile (m2); A= $(\pi * D^2)/4$ = $(3.14*1.0^2)/4$= 0.785

D: pile diameter (m); D= 1.0 (m)

U: perimeter of the pile (m); U= π*D= 3.14*1.0= 3.14

li: thickness of i-th layer considered for the bearing capacity (m)

fi: skin friction of i-th layer (kN/m2)

DE: reduction factor of i-th layer's soil parameters considering liquefaction effects

From the following table,

- U*∑ li *fi*DE = 2412.8 (kN); nominal skin friction
- Ws= A*∑ li*γ' = 68.1 (kN); weight of soil inside the pile

Calculation of friction resistance

Layer	Layer type	li (m)	N-value	γ' (kN/m3)	li*γ' *A (kN)	fi (kN/m2)	DE	U*Li*fi*DE (kN/m2)
1	Sand	3.50	11	9.0	24.7	55	0.7	403.2
2	Cray	4.00	7	7.0	22.0	70	-	879.2
3	Rock	1.00	28	9.0	7.1	150	-	471.0
4	Rock	1.40	50	13.0	14.3	150	-	659.4
5	0.000	0.00	0	0.0	0.0	0	-	0.0
6	0.000	0.00	0	0.0	0.0	0	-	0.0
7	0.000	0.00	0	0.0	0.0	0	-	0.0
Sum				Ws=	68.1			2412.8

W: effective weight of the pile and soil inside it; W= (γc - γw)*A*L= (24.0-10.0)*0.785*10.0= 109.9

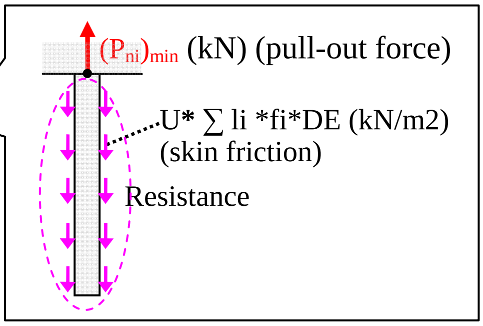
γ_c: unit weight of pile (kN/m3); γ_c= 24.0

γ_w: unit weight of water (kN/m3); γ_w= 10.0

L: length of the pile (m); L= 10.0

[Capacity against axial pull-out force]

P_R: factored axial pull-out resistance of single pile (kN);



P_R= -(ϕ *Pn+W)

= -(0.50*2412.8+109.9)

= -1316.3 (kN)

Where,

ϕ : resistance factor for capacity against axial pull-out force under extreme event limit state; ϕ =0.50

Pn: nominal axial pull-out resistance of single pile (kN); Pn= U* ∑ li *fi*DE= 2412.8

W: effective weight of the pile and soil inside it; W= (γc - γw)*A*L= 109.9

2.7.4.3 Stability Verification for Longitudinal Direction

- Displacement verification

  δx= 14.8 < 15.0 (mm) (OK)

- Stability verification against push-in force

  $(P_{ni})_{max}$= 3509.7 < 4009.6 (kN) (OK)

- Stability verification against pull-out force

  $(P_{ni})_{min}$= -1255.2 < -1316.3 (kN) (OK)

2.7.4.4 Stability Verification for Transverse Direction

- Displacement verification

  δx= 8.8 < 15.0 (mm) (OK)

- Stability verification against push-in force

  $(P_{ni})_{max}$= 3482.3 < 4009.6 (kN) (OK)

- Stability verification against pull-out force

  $(P_{ni})_{min}$= -1227.7 < -1316.3 (kN) (OK)

2.7.5 Verification of Pile Resistance against Seismic Force

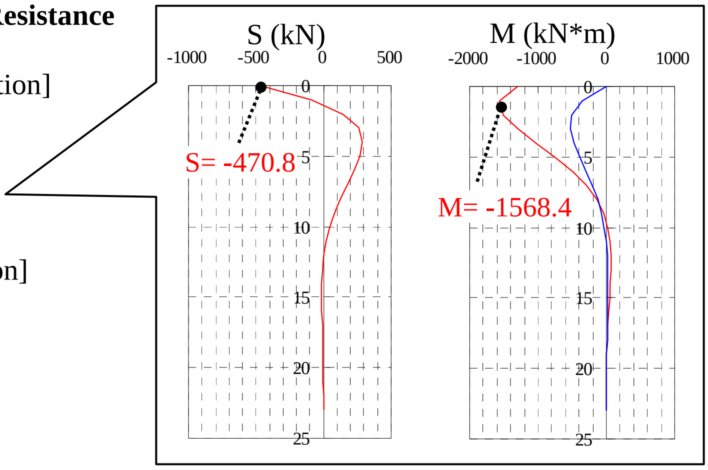
2.7.5.1 Forces for Verification of Pile Resistance

[Verification forces for longitudinal direction]

- M= 1568.4 (kN*m)
- S= 470.8 (kN)

[Verification forces for transverse direction]

- M= 698.3 (kN*m)
- S= 401.2 (kN)



2.7.5.2 Pile Resistance

[Flexural resistance]

- for longitudinal direction

  Mr= ϕ *Mn

  = 0.9*2014

  = 1812.6 (kN*m)

  (Nd= $(P_{ni})_{min}$= -1255)

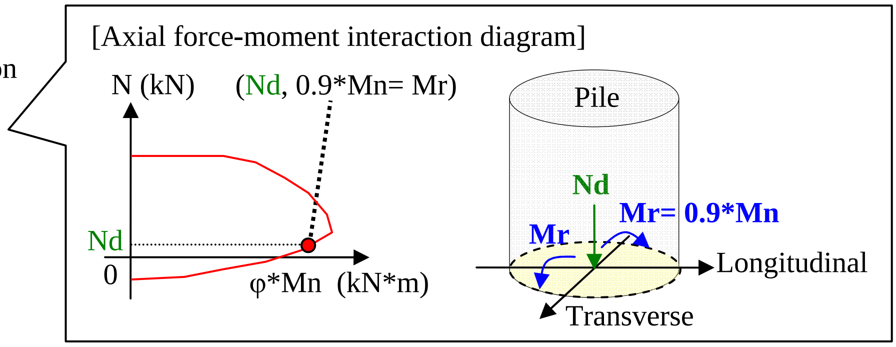
- for transverse direction

  Mr= ϕ *Mn

  = 0.9*2021

  = 1818.9 (kN*m)

(Nd= $(P_{ni})_{min}$= -1228)



Where,

Mu: flexural resistance of single pile (kN*m) with re-bars of 18-D36

ϕ : resistance factor; ϕ = 0.9

Mn: nominal flexural resistance (kN*m)

Note: ϕ *Mn is obtained from "the N-M interaction diagram" shown below

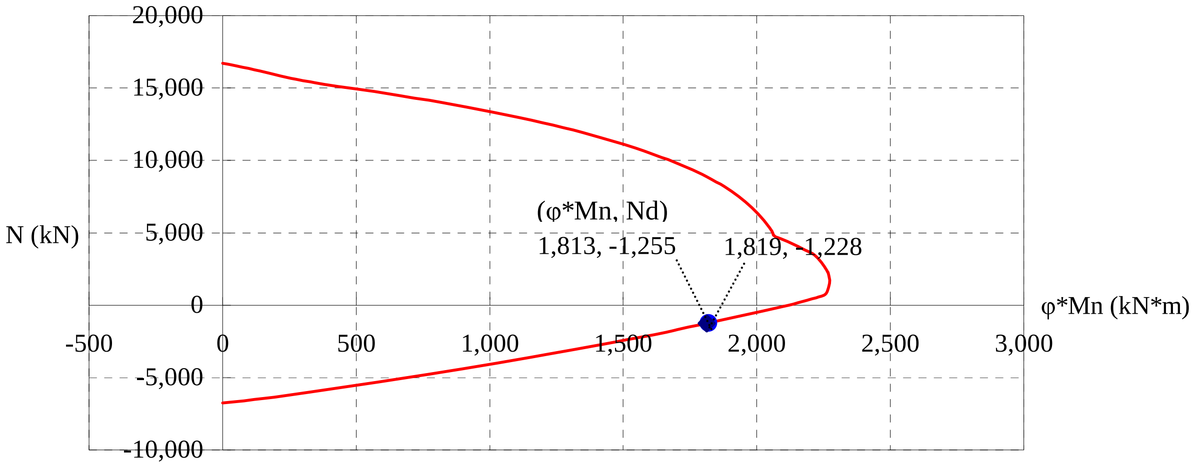


Figure 2.7.5-1 N-M Interaction Diagram of the Column

[Shear resistance]

- Step-1: calculation of "Vn"

Vn= Vc+Vs

= 0+1074

=1074 (kN)

Where,

Vn: nominal shear resistance (kN)

Vc: shear strength developed by concrete (kN);

Vc= $0.083*\beta*\sqrt{f'c'}*bv*dv$

= 0 (kN) (to be on the conservative side)

Vs: shear strength developed by re-bars (kN);

Vs= (Av*fy*dv*cotθ)/s

= (398*415*cot45)/100

= 1074 (kN)

β: factor indicating ability of diagonally cracked concrete to transmit tension and shear; β= 2.0

θ: Angle of inclination of diagonal compressive stress (degrees); θ= 45

f'c': specified compressive strength of concrete (N/mm2); fc'= 28

bv: effective web width (mm); bv= 1000

dv: effective shear span (m);

dv= 0.9*de= 0.9*(D/2+Dr/π)

= 0.9*(1000/2+700/3.14)

= 651 (mm)

D: Gross diameter of the column (mm); D= 1000

Dr: Diameter of the circle passing through the centers of longitudinal re-bars (mm); Dr= 700

fy: yield strength of transverse reinforcement (N/mm2); fy= 415

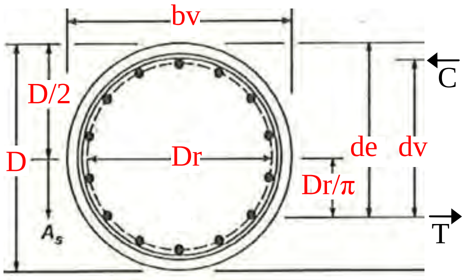
Av: area of shear reinforcement within distance "s" (mm2); Av= (199*2)*2=398 (diameter: 16)

s: spacing of transverse reinforcement (mm); s= 100

Simplified procedure is applied with the following values;

- β=2.0
- θ=45

[Definition of parameters]



C: compression

T: tension

- Step-2: determination of "Vr"

Vr= ϕ*Vn

= 0.9*1074

= 967.0 (kN)

Where,

Vr: shear resistance of the column (kN)

ϕ: resistance factor; ϕ = 0.9

2.7.5.3 Verification for Resistance of Single Pile

[Resistance verification for longitudinal direction]

- Flexural resistance of single pile

Md= 1568.4 < 1812.6 (OK)
    (0.87)   (1.00)

- Shear resistance of single pile

Vd= 470.8 < 967.0 (OK)
    (0.49)   (1.00)

[Capacity verification]

- Flexural resistance of single pile

Md= 698.3 < 1818.9 (OK)
    (0.38)   (1.00)

- Shear resistance of single pile

Vd= 401.2 < 967.0 (OK)
    (0.41) (1.00)

2.7.6 Verification of Minimum Required Reinforcement

2.7.6.1 Verification of Minimum Required Longitudinal Reinforcement

The longitudinal reinforcement ratio is verified to be not less than 0.75 as shown below.

ρs= Aa/Ag
   = 0.018/0.785
   = 0.023 > 0.0075 (OK)

Where,

ρs: ratio of longitudinal reinforcement to gross area of concrete section

Aa: total area of longitudinal reinforcement (m2); Aa= As*n = $(1006/10^6)$*18 = 0.018

As: cross-sectional area of single longitudinal reinforcement bar (mm2); As= 1006

n: number of longitudinal reinforcement bar; n= 18

Ag: gross area of concrete section (m2); Ag= $\pi*(D/2)^2 = 3.14*(1.0/2)^2 = 0.785$

D: diameter of pile (m); D= 1.0

2.7.6.2 Verification of Minimum Required Transverse Reinforcement

The transverse reinforcement ratio satisfies the requirement as shown below.

$$\rho_s \geq 0.12 * \frac{f_c'}{f_{yh}}$$

$$= 0.12 * \frac{28}{415}$$

$$= 0.0081$$

Where,

ρs: ratio of transverse reinforcement to total volume of concrete core

f'c': specified compressive strength of concrete (N/mm2); fc'=28

fyh: yield strength of transverse reinforcement (N/mm2); fy= 415

Therefore,

$$\rho_s = \frac{\pi * D_r * (A_v / 2)}{\pi * (D_r / 2)^2 * s} = \frac{4 * (A_v / 2)}{D_r * s}$$

$$= \frac{4 * (398 / 2)}{700 * 100}$$

$$= 0.0114 > 0.0081 \ (OK)$$

Where,

ρs: ratio of transverse reinforcement to total volume of concrete core

s: spacing of transverse reinforcement (mm); s= 100

Av: area of shear reinforcement within distance "s" (mm2); Av= 398 (=199*2, As= 199)

Dr: diameter of the circle passing through the centers of longitudinal re-bars (mm); Dr= 700

3. Cost Comparative Study on Determination of Return Period of Design Acceleration Response Spectrum for BSDS

3.1 Background and Objective of the Cost Comparative Study

3.1.1 Background

Development of design earthquake ground motions for use in seismic design codes in the Philippines has been largely stagnated. In the early part of this project, JICA Study Team recognized the necessity of study on development and sustainable evolution of localized design earthquake ground motions for use in seismic design of Philippine bridges. However, the Study Team found out the deficiencies regarding the localization of ground motions such as;

- adoption of AASHTO code in the past,
- difficulties to adopt the latest code,
- non-existence of strong-motion records in the Philippines, and
- limited existing data in seismology (not directly usable in seismic hazard analysis).

In view of the above, the Study Team decided to conduct the study on development of localized design earthquake ground motions using probabilistic seismic hazard analysis (PSHA) in order to generate design spectral parameters for obtaining design acceleration response spectra whose methodology was based on the latest AASHTO code. As a result, counter maps of spectral parameters (PGA, SA at 0.2 sec, and SA at 1.0 sec) corresponding to 500-year return period and 1000-year return period earthquakes were prepared. Now that the contour maps are prepared, the earthquake return period applied in the seismic design in the Philippines needs to be decided, considering the balance between structural safety and economics.

3.1.2 Objective

The objective of the comparative study here is to examine;

- the effect of earthquake return period difference (500-year or 1000-year) on structure design, and
- the effect of earthquake design code difference (NSCP or BSDS) on structure design.

3.2 Study Condition

The comparative study was conducted under the following design conditions. The conditions are totally same as those used in the seismic design example except that three different design acceleration spectra were applied in each study case.

- Bridge type: simply-supported composite steel I-shaped girder bridge
- Span length: 30.0 (m)
- Total road width: 10.5 (m)
- Skew angle: 90 degrees (non-skewed straight bridge)

- Pier type: single circular column
- Pier height: 11.9 (m) (column height: 10.0 (m))
- Foundation type: cast-in-place concrete pile foundation
- Centroid of the superstructure: 2.0m from the column top
- Reaction forces for the design
- Reaction force caused by “dead load”: Rd=2900 (kN/pier)
- Reaction force caused by “live load”: Rl=1800 (kN/pier)

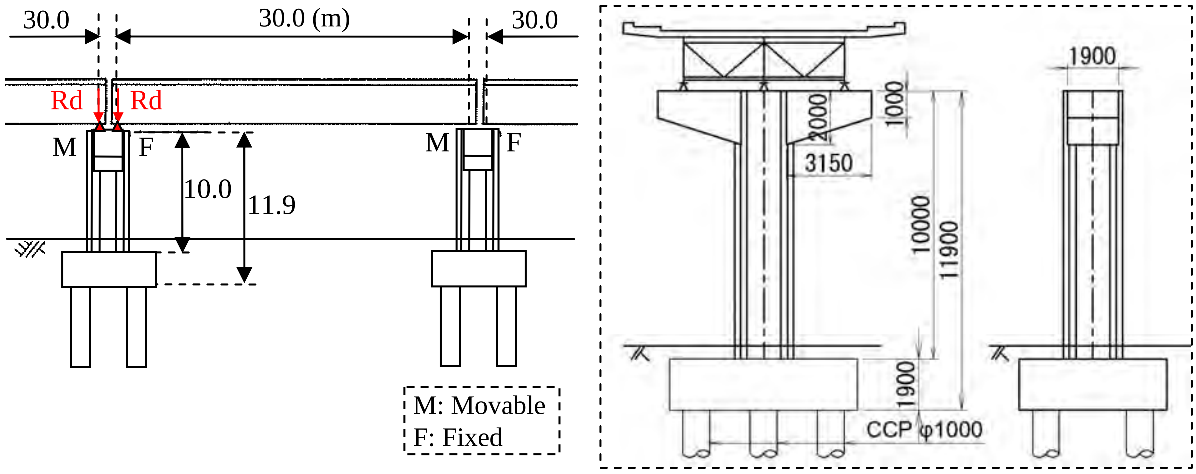
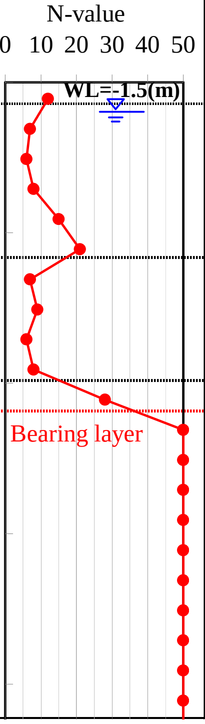


Figure 3.2-1 Structural Condition for Comparative Study

- Ground condition (geotechnical investigation result of Lambingan Bridge site; soil type-II)

α= 4, Eo= 2800*N



Layer symbol	Layer Thickness (m)	Depth (m)	N-value Blows	N-value Ave.	γt (kN/m^2)	FC (%)	D50 (mm)	C (kN/m^2)	φ (°)	α*Eo (kN/m^2)	Vsn (m/sec)
Bs	1	0	12	12	17	0.9	0.74	0	35	8,400	183
As	5		7	11	17	17.3	0.14	0	34	7,700	178
			6		17	28.0	0.12				
			8		17	12.0	0.21				
			15		17	7.3	0.42				
		5	21		17	7.3	0.20				
Ac	4		7	7	15	58.1	-	44	0	4,900	191
			9		15	77.1	-				
			6		15	66.9	-				
			8		15	51.0	-				
WGF	1	10	28	28	17	0.2	2.38	-	37	19,600	292
GF	10		50	50	21	0.5	0.60	173	21	39,530	292
			50		21	-	-				
			50		21	-	-				
		15	50		21	-	-				
			50		21	-	-				
			50		21	-	-				
			50		21	-	-				
			50		21	-	-				
		20	50		21	-	-				
			50		21	-	-				

- Design acceleration response spectra
  1) NSCP (soil type-II)
  2) BSDS (soil type-II, 500-year return period; Lambingan Bridge site)
  3) BSDS (soil type-II, 1000-year return period; Lambingan Bridge site)

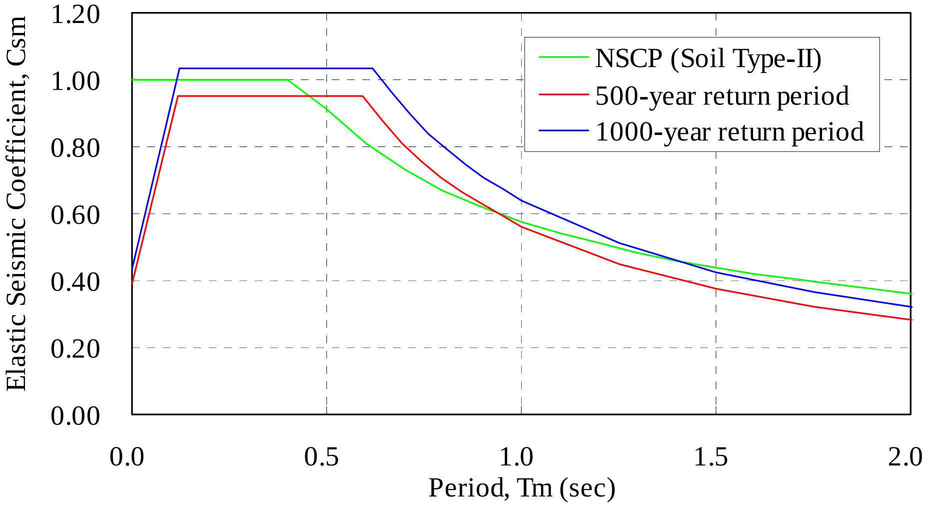


Figure 3.2-2 Design Acceleration Response Spectrum for Comparative Study

3.3 Study Cases

The following five cases were chosen for the cost comparative study.

Table 3.3-1 Study Cases

Case	Applied Spectra	Return Period (Design Spectra)	R-factor	Resistance factor φ		Column stiffness	Disp. check
				Flexural	Shear		
1	NSCP	500-year (NSCP)	3.0 (Essential/Critical)	0.75	0.85	Uncracked section	-
2	BSDS	500-year (BSDS)	2.0 (Essential)	0.75	0.85	Cracked section	Yes
3	BSDS	1000-year (BSDS)	2.0 (Essential)	0.90	0.90	Cracked section	Yes
4	BSDS	500-year (BSDS)	1.5 (Critical)	0.75	0.85	Cracked section	Yes
5	BSDS	1000-year (BSDS)	1.5 (Critical)	0.90	0.90	Cracked section	Yes

As shown in the above table, the design of five study cases was differentiated with the following six parameters.

1) Applied Spectra

Either NSCP spectra or BSDS spectrum was applied in each study case.

2) Return Period

As the return period of acceleration design spectra, either 500-year or 1000-year was in each study case.

3) Response modification factor (R-factor)

Response modification factor (R-factor) was chosen under single circular column condition for each study case, considering the difference of applied codes and bridge importance. The applied factors are 3.0 for NSCP, 2.0 for BSDS with essential bridge, and 1.5 for BSDS with critical bridge.

4) Flexural resistance factor

Either "φ= 0.75" or "φ= 0.90" was applied for flexural resistance factor in each study case. "φ= 0.75" is the factor used with 500-year return period design spectra, based on LFD. Also, "φ=

0.90” is the factor used with 1000-year return period design spectra, based on LRFD (ver. after 2008).

5) Shear resistance factor

Either “φ= 0.85” or “φ= 0.90” was applied for shear resistance factor in each study case. “φ= 0.85” is the factor used with 500-year return period design spectra, based on LFD. Also, “φ= 0.90” is the factor used with 1000-year return period design spectra, based on LRFD (ver. after 2008).

6) Column stiffness

Either “Uncracked section” or “Cracked section” was applied for column stiffness in bridge analysis. By applying “Cracked section”, natural period of the column becomes longer.

7) Displacement Check

Displacement was verified for cases designed in accordance with BSDS.

Note: Foundation spring defined in BSDS was applied to all the above cases.

3.4 Result of the Study

As shown in the following table, resulted costs ratio of studied cases range from 1.00 to 1.64. Major findings of the study are;

- cost increase by changing R-factor from 3.0 to 2.0 is about 18 % (comparison of Case-1 and Case-2).
- cost increase by changing R-factor from 3.0 to 1.5 is about 25 % (comparison of Case-1 and Case-4).
- cost difference caused by difference of earthquake return periods is 2% under the condition of R-factor 2.0 (comparison of Case-2 and Case-3).
- cost difference caused by difference of earthquake return periods is 32% under the condition of R-factor 1.5 (comparison of Case-4 and Case-5).

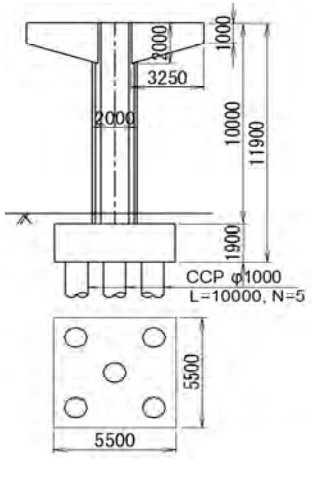
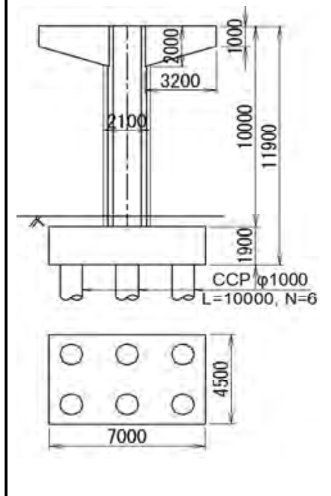
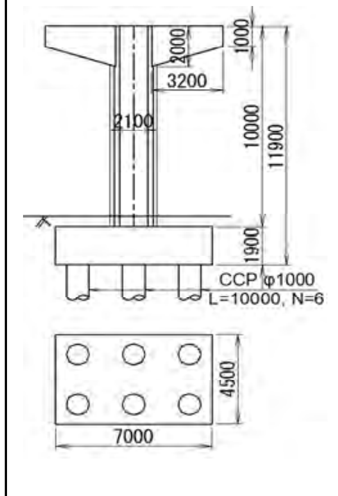
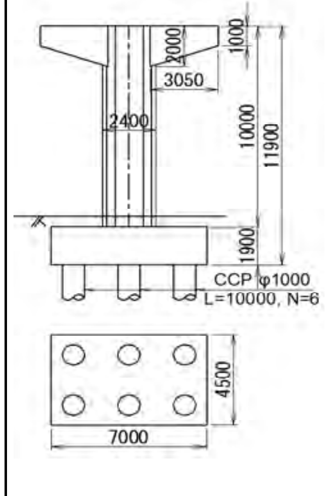
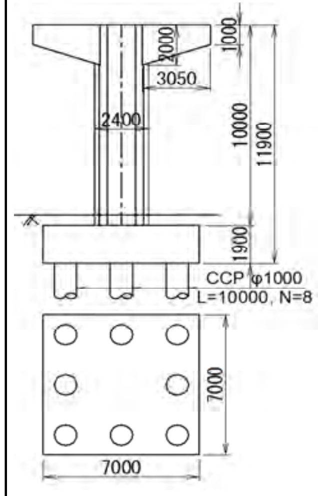
In consideration of above findings, the application of 1000-year return period design spectrum is considered to be the most realistic and practical. On the contrary, the application of R-factor 1.5 is considered to be unrealistic and impractical for the design of typical bridges. R-factor 1.5 should be applied only the special cases such as the design of very important bridges which connect islands.

Table 3.4-1 Result of the Cost Comparative Study (Summary)

Case	Applied Spectra	Return Period	R-factor	Resistance factor φ		Column stiffness	Disp. check	Column diameter (m)	No. of piles	Cost	
				Flexura	Shear					Php	Ratio
1	NSCP	500-year (NSCP)	3.0 (Essential/ Critical)	0.75	0.85	Uncracked section	-	2.0	5	3,781,279	1.00
2	BSDS	500-year (BSDS)	2.0 (Essential)	0.75	0.85	Cracked section	Yes	2.1	6	4,474,810	1.18
3	BSDS	1000-year (BSDS)	2.0 (Essential)	0.90	0.90	Cracked section	Yes	2.1	6	4,575,204	1.21
4	BSDS	500-year (BSDS)	1.5 (Critical)	0.75	0.85	Cracked section	Yes	2.4	6	4,723,966	1.25
5	BSDS	1000-year (BSDS)	1.5 (Critical)	0.90	0.90	Cracked section	Yes	2.4	8	6,242,197	1.65

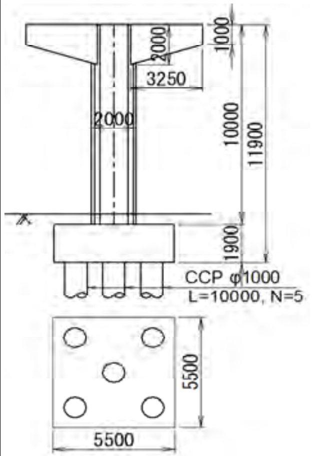
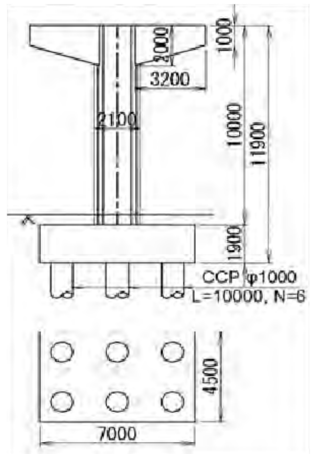
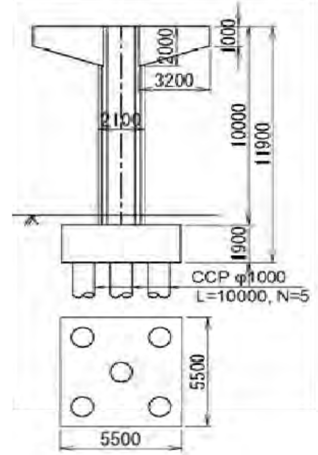
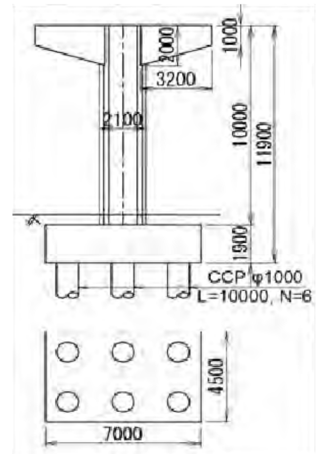
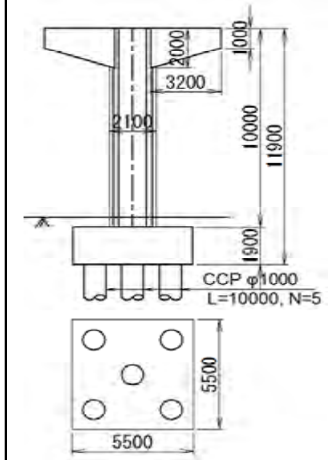
The further detail of the cost comparative study result is shown in the following table. Additionally, the effect of displacement check on the cost was shown after the main comparison table.

Table 3.4-2 Result of the Cost Comparative Study (Detail)

Study Case		Case-1		Case-2		Case-3		Case-4		Case-5	
Applied Spectra		NSCP		BSDS		BSDS		BSDS		BSDS	
Return period		500-year		500-year		1000-year		500-year		1000-year	
R-factor		3.0 (Essential/Critical)		2.0 (Essential)		2.0 (Essential)		1.5 (Critical)		1.5 (Critical)	
Resistance factor	Flexural	φ= 0.75		φ= 0.75		φ= 0.9		φ= 0.75		φ= 0.9	
	Shear	φ= 0.85		φ= 0.85		φ= 0.9		φ= 0.85		φ= 0.9	
Column stiffness		Uncracked section		Cracked section		Cracked section		Cracked section		Cracked section	
Displacement check		Not required		Required[1)]		Required[1)]		Required[1)]		Required[1)]	
Dimension											
Reinforcement (Column)		154 kg/m3		186 kg/m3		203 kg/m3		181 kg/m3		208 kg/m3	
Seismic coefficient	Long. dir.	Csm= 0.60 (T= 0.94 sec)		Csm= 0.60 (T= 0.93 sec)		Csm= 0.69 (T= 0.93 sec)		Csm= 0.72 (T= 0.78 sec)		Csm= 1.02 (T= 0.63 sec)	
	Trans. dir.	Csm= 0.57 (T= 1.03 sec)		Csm= 0.47 (T= 0.96 sec)		Csm= 0.66 (T= 0.96 sec)		Csm= 0.55 (T= 0.83 sec)		Csm= 0.72 (T= 0.78 sec)	
Column Capacity	Flexure	Md= 8499 < 10451 (= 0.75*Mn)		Md= 11825 < 14797 (= 0.75*Mn)		Md= 14939 < 194569 (= 0.9*Mn)		Md= 19212 < 22071 (= 0.75*Mn)		Md= 27395 < 30723 (= 0.9*Mn)	
	Shear	Vd= 2572 < 5121 (= 0.85*Vn)		Vd= 2599 < 5393 (= 0.85*Vn)		Vd= 3011 < 5710 (= 0.9*Vn)		Vd= 3252 < 6209 (= 0.85*Vn)		Vd= 4626 < 6574 (= 0.9*Vn)	
	Disp.	-		δ= 0.91 < 0.97		δ= 1.24 < 1.28		δ= 0.62 < 1.45		δ= 0.89 < 2.02	
Stability	Bearing	$P_{n\text{-}max}$= 2990 < 4010		$P_{n\text{-}max}$= 2905 < 4010		$P_{n\text{-}max}$= 3466 < 4010		$P_{n\text{-}max}$= 3721 < 4010		$P_{n\text{-}max}$= 3540 < 4010	
	Pull-out	$P_{n\text{-}min}$= -441 > -1678		$P_{n\text{-}min}$= -738 > -1678		$P_{n\text{-}min}$= -1299 > -1678		$P_{n\text{-}min}$= -1422 > -1678		$P_{n\text{-}min}$= -1606 > -1678	
Cost (Php)	Pier	1,681,279	1.00	1,954,810	1.16	2,055,204	1.22	2,203,966	1.31	2,882,197	1.71
	Pile	2,100,000	1.00	2,520,000	1.20	2,520,000	1.20	2,520,000	1.20	3,360,000	1.60
	Total	3,781,279	1.00	4,474,810	1.18	4,575,204	1.21	4,723,966	1.25	6,242,197	1.65

1) Verification formula: △*Pu = 0.25*φ*Mn

Table 3.4-3 Result of the Cost Comparative Study (Effect of Displacement Check on the Cost)

Study Case		Case-1		Case-2		Case-2+		Case-3		Case-3+	
Applied Spectra		NSCP		BSDS		BSDS		BSDS		BSDS	
Return period		500-year		500-year		500-year		1000-year		1000-year	
R-factor		3.0 (Essential/Critical)		2.0 (Essential)		2.0 (Essential)		2.0 (Essential)		2.0 (Essential)	
Resistance factor	Flexural	φ= 0.75		φ= 0.75		φ= 0.75		φ= 0.9		φ= 0.9	
	Shear	φ= 0.85		φ= 0.85		φ= 0.85		φ= 0.9		φ= 0.9	
Column stiffness		Uncracked section		Cracked section		Cracked section		Cracked section		Cracked section	
Displacement check		Not required		Required[1)]		Not required		Required[1)]		Not required	
Dimension											
Reinforcement (Column)		154 kg/m3		186 kg/m3		169 kg/m3		203 kg/m3		180 kg/m3	
Seismic coefficient	Long. dir.	Csm= 0.60 (T= 0.94 sec)		Csm= 0.60 (T= 0.93 sec)		Csm= 0.62 (T= 0.90 sec)		Csm= 0.69 (T= 0.93 sec)		Csm= 0.71 (T= 0.90 sec)	
	Trans. dir.	Csm= 0.57 (T= 1.03 sec)		Csm= 0.47 (T= 0.96 sec)		Csm= 0.42 (T= 1.08 sec)		Csm= 0.66 (T= 0.96 sec)		Csm= 0.59 (T= 1.08 sec)	
Column Capacity	Flexure	Md= 8499 < 10451 (= 0.75*Mn)		Md= 11825 < 14797 (= 0.75*Mn)		Md= 12072 < 13357 (= 0.75*Mn)		Md= 14939 < 194569 (= 0.9*Mn)		Md= 13993 < 17182 (= 0.9*Mn)	
	Shear	Vd= 2572 < 5121 (= 0.85*Vn)		Vd= 2599 < 5393 (= 0.85*Vn)		Vd= 2658 < 5393 (= 0.85*Vn)		Vd= 3011 < 5710 (= 0.9*Vn)		Vd= 3068 < 5710 (= 0.9*Vn)	
	Disp.	Not required		δ= 0.91 < 0.97		Not required		δ= 1.24 < 1.28		Not required	
Stability	Bearing	$P_{n\text{-}max}$= 2990 < 4010		$P_{n\text{-}max}$= 2905 < 4010		$P_{n\text{-}max}$= 3368 < 4010		$P_{n\text{-}max}$= 3466 < 4010		$P_{n\text{-}max}$= 3950 < 4010	
	Pull-out	$P_{n\text{-}min}$= -441 > -1678		$P_{n\text{-}min}$= -738 > -1678		$P_{n\text{-}min}$= -787 > -1678		$P_{n\text{-}min}$= -1299 > -1678		$P_{n\text{-}min}$= -1369 > -1678	
Cost (Php)	Pier	1,681,279	1.00	1,954,810	1.16	1,816,664	1.08	2,055,204	1.22		0.00
	Pile	2,100,000	1.00	2,520,000	1.20	2,100,000	1.00	2,520,000	1.20	2,100,000	1.00
	Total	3,781,279	1.00	4,474,810	1.18	3,916,664	1.04	4,575,204	1.21	3,982,153	1.05

1) Verification formula: △*Pu = 0.25*φ*Mn

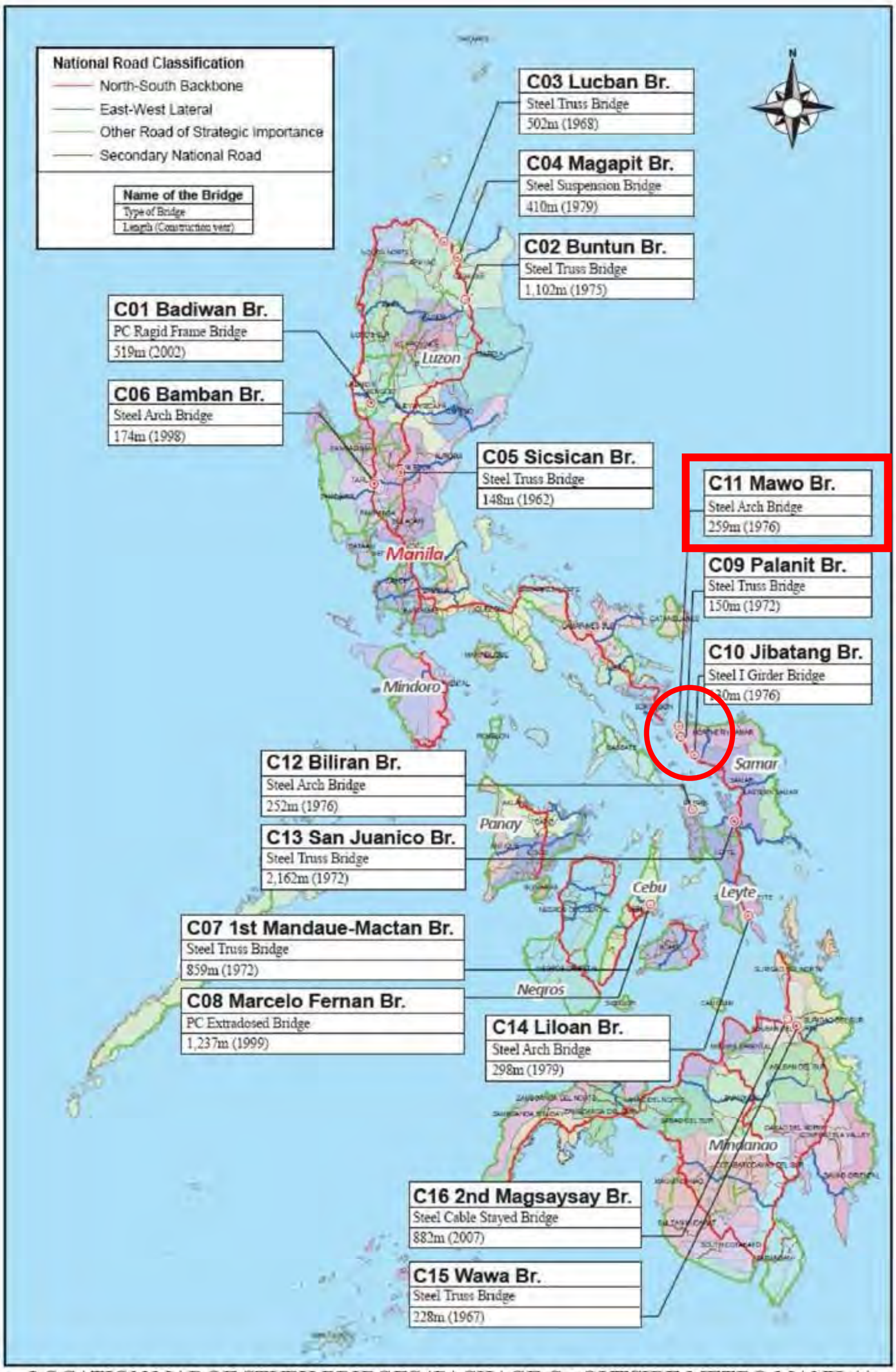
Note: There's no effect of displacement check on cost of Case-4 & 5 because displacement check doesn't dominate the designs.

PART 3

DESIGN EXAMPLE-2:
SEISMIC DESIGN EXAMPLE OF CONTINUOUS BRIDGE
(REPLACEMNT PLAN OF MAWO BRIDGE)

1. Design Condition

1.1 Location of Mawo Bridge



LOCATION MAP OF STUDY BRIDGES (PACKAGE C : OUTSIDE METRO MANILA)

Figure 1.1-1 Location of Mawo bridge

1.2 Design Condition

For the design example using new "DPWH Guide Specifications LRFD Bridge Seismic Design Specifications, 1st Edition, 2013", hereinafter referred to as "BSDS", Mawo bridge is selected because the superstructure is of girder type that does not behave with complicated modes and there are both structurally different foundation types such as pile and spread foundations; therefore, Mawo bridge may be structurally appropriate bridge for organization of the seismic design example as well as the point of view of general use of new BSDS.

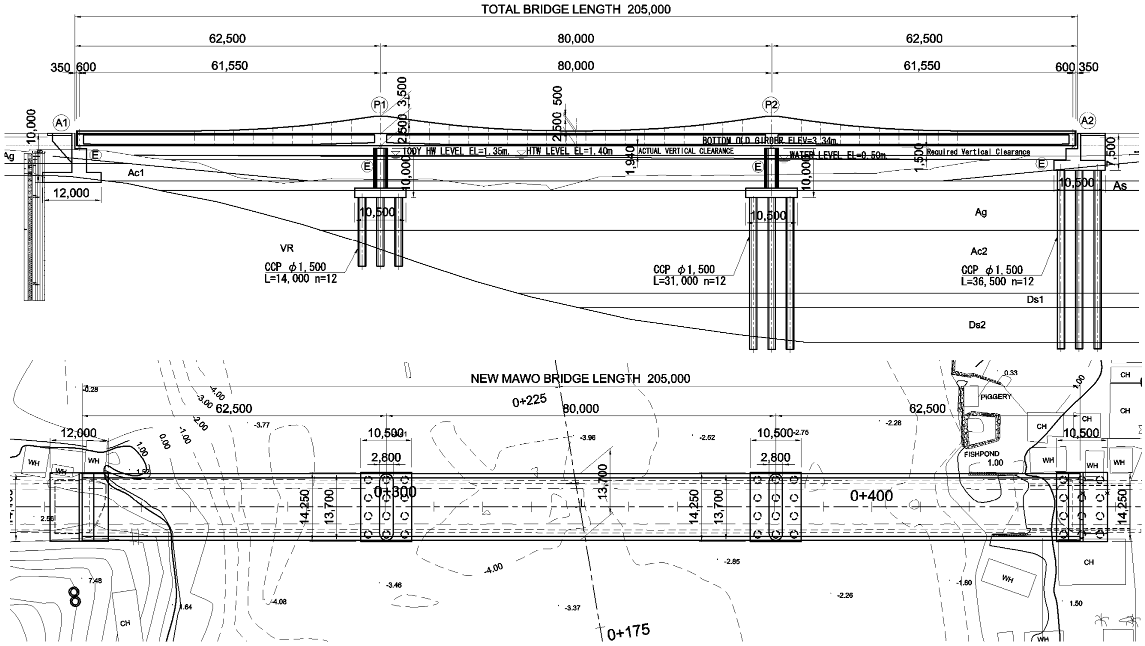


Figure 1.2-1 Mawo Bridge

(1) Superstructure

Superstructure Type	:	3-Span Continuous PC Fin-Back Box Girder
Bridge Length	:	L = 205.0m
Span Arrangement	:	62.5m + 80.0m + 62.5m
Bridge Width	:	Shown as follows:

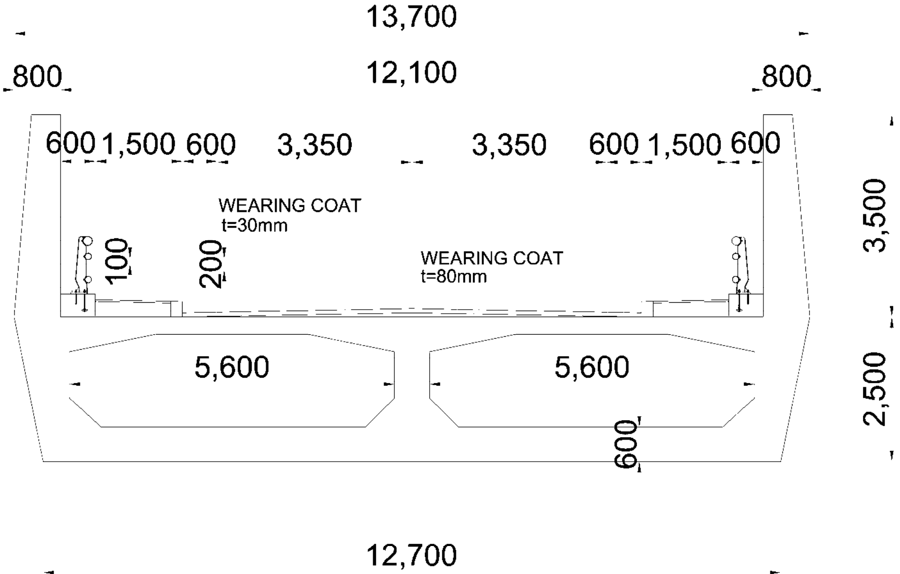


Figure 1.2-2 Cross Section/ Lane Arrangement of Mawo Bridge

Live Loads : AASHTO Live Loads HL93 and Lane Loads
Supporting Bearing : Laminated Rubber Bearing - Force Distribution Bearing
Supporting Condition : Shown as follows:

Table 1.2-1 Supporting Condition of New Bridges

	Bearing Support Abutment No.1	Bearing Support Pier No.1	Bearing Support Pier No.2	Bearing Support Abutment No.2
Longitudinal Direction	Elastic Support	Elastic Support	Elastic Support	Elastic Support
Transversal Direction	Fixed	Elastic Support	Elastic Support	Fixed

(2) Substructure

Abutment Type : Reversed T Type
Pier Type : RC Wall Type
Foundation : Spread Footing (Abutments No.1)
Cast-In-Place Pile (Piers and Abutment No.2)

(3) Materials

Table 1.2-1 Concrete Strength by Structural Member

Compressive Strength at 28 days (MPa) (Cylinder Specimen)	Structural Member
40	PC Girder and Fin-back
28	Substructure (Pier, Abutment, Pile Caps, Wing wall)
21	Approach Slab
28	Cast-in-situ Bored Pile
18	Non-reinforced Concrete Structure Lean Concrete

Table 1.2-2 Properties and Stress Limit of Reinforcing Bars

Type	Yield Strength f y (MPa)	Tensile Strength f u (MPa)	Modulus of Elasticity (MPa)	Diameter of Bar (mm)
Grade 275	275	500	200,000	D10, D12, D16,D20
Grade 415	414	620	200,000	D25,D28,D32,D36

(4) Soil Condition

The foundation is supported by assumed bearing layer shown as the following tables.

Table 1.2-3 Soil Conditions (L1)

Layer		γ(kN/m3)	N-value	C(kN/m2)	φ(deg)	E0(kN/m2)	Vs(m/sec)
Ac1	Clay	14	3	19	0	2,100	144
As	Sand	17	10	0	34	7,000	172
Ag2	Gravel	18	21	0	36	14,700	221
Ac2	Clay	18	8	50	0	5,600	200
Ds1	Sand	17	17	0	32	11,900	206
Ds2	Sand	19	31	0	34	21,700	251
VR	Rock	21	300	170	38	136,098	300

Table 1.2-4 Soil Conditions (L2)

Layer		γ(kN/m3)	N-value	C(kN/m2)	φ(deg)	E0(kN/m2)	Vs(m/sec)
Ag1	Gravel	18	7	0	32	4,900	153
Ac1	Clay	14	2	13	0	1,400	126
As	Sand	19	50	0	41	70,000	213
VR	Rock	21	300	170	38	136,098	300

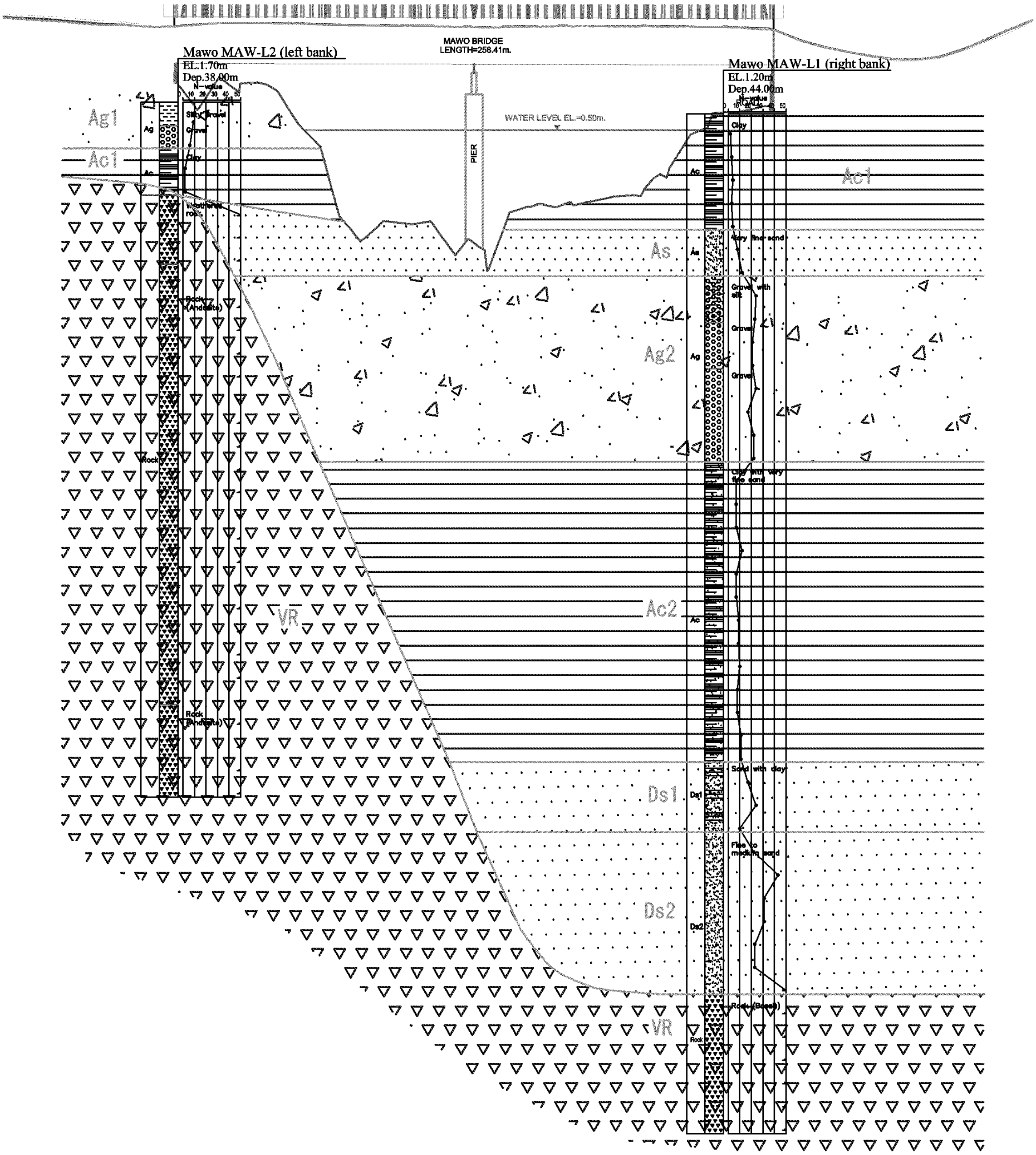


Figure 1.2-3 Geological Map at Mawo Bridge

1.3 Procedure Flow Chart of the Design Example of Mawo Bridge

Basically, the design example proceeds in accordance with “1.6 SEISMIC DESIGN FLOWCHART OF BSDS”.

The procedure flow chart of this design example Mawo Bridge is shown in the following figure based on newly specified BSDS. In this design example, only the seismic design is mentioned throughout multimode analysis; the design example of limit state design and serviceability design is not included.

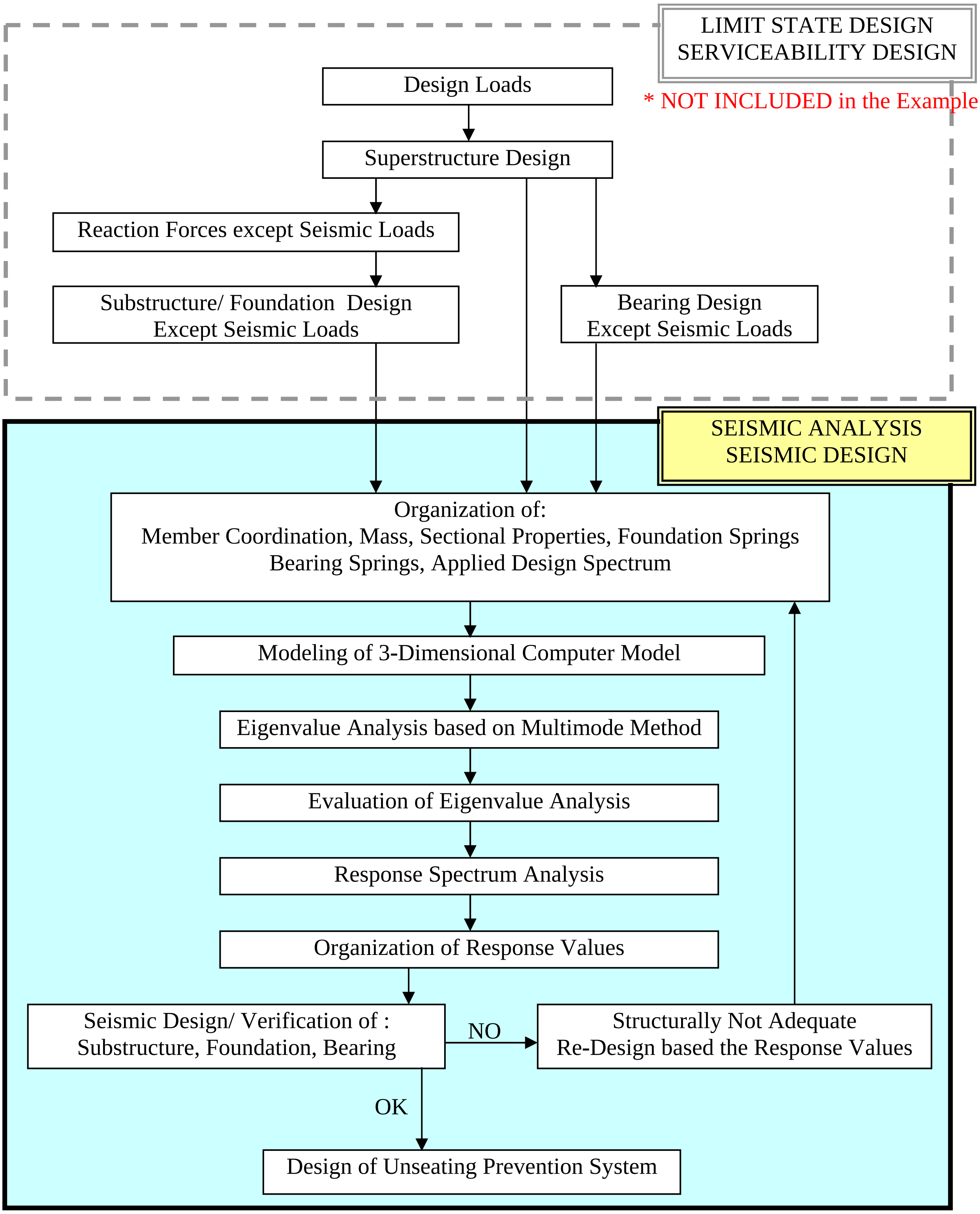


Figure 1.3-1 Procedure Flow Chart of the Design Example of Mawo Bridge

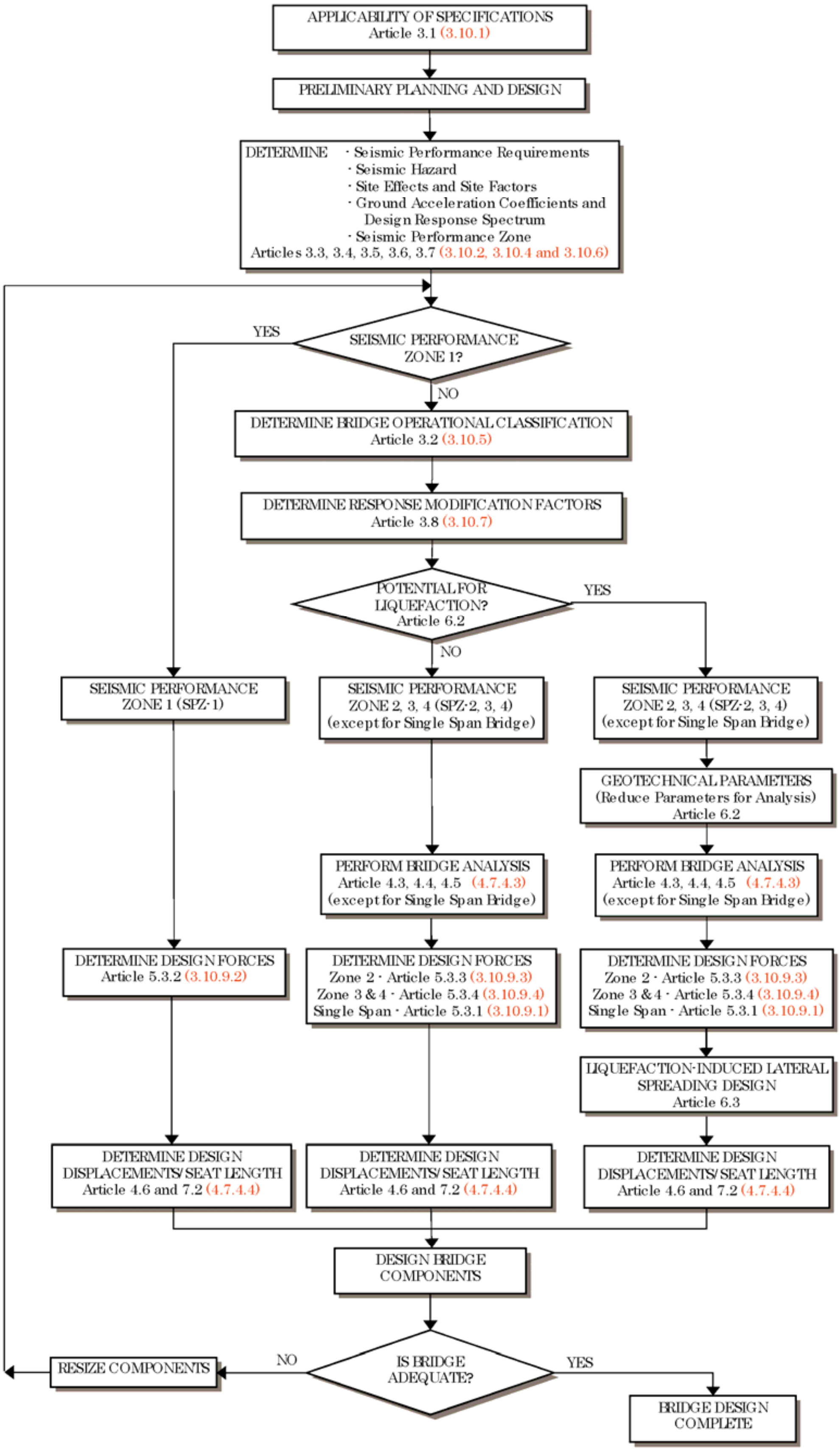


Figure 1.3-2 Seismic Design Procedure Flow Chart, Specified in BSDS

2. Eigenvalue and Response Spectrum Analysis

2.1 Seismic Analysis Methodology

2.1.1 General

For seismic design, responses of structure by given seismic forces must not be exceeded specified limitation values. As the calculation methodologies to obtain such the responses of structure, various numerical computing analytical approaches are worldwidely utilized such as static analysis, dynamic analysis, elastic analysis and elasto-plastic analysis.

Currently, familiar analytical approaches utilized in earthquake countries is categorized into static analysis and dynamic analysis, furthermore dynamic analysis can be categorized into eigenvalue analysis based on single or multimode, response spectrum analysis under elastic method and time-history response analysis under elasto-plastic method. In this sentence, the characteristic properties of such the various methodologies are organized and the seismic analysis method to be applied to the outline design example is selected.

2.1.2 Static Analysis (Uniform Load Elastic Method)

Analytical approaches of seismic performance have two methodologies such as static and dynamic analysis. The static analysis is the most simplified method because vibration characteristic has been transposed to static uniform load system under the precondition that equal energy assumption is approval. However, the load system of static analysis is commonly based on a basic vibration mode vector, what it is a basic shape of mode vector that can be transposed to mono-mass-system model and is not applicable to seismically irregular bridge. Furthermore, damping matrix as well as mass matrix do not exist naturally; therefore, the design's flexibility may be comparatively low because responses should be computed depending on only stiffness matrix and because structural damping and hysteresis damping of seismic countermeasure devices such as seismic-isolation bearing and viscosity damper cannot be considered accurately in the analysis.

< Static Analysis >

Internal Forces (Member Forces) = External Forces (Horizontal Loads):

$$K \times U = P \qquad \text{Eq.}$$

K: Stiffness matrix, U: Displacement of Nodes, P: Horizontal Forces

< Dynamic Analysis >

Internal Forces (Inertial Forces + Damping Forces + Member Forces) = Seismic Forces:

$$M \times \ddot{U} + C \times \dot{U} + K \times U = M \times \ddot{Z} \qquad \text{Eq.}$$

M: Mass matrix, C: Damping matrix, K: Stiffness Matrix, $\ddot{Z}$ *:Acceleration Vector,*

$\ddot{U}$: *Acceleration Vector of Nodes,* $\dot{U}$ *: Velocity of Nodes,* U *: Displacement of Nodes*

Therefore, the static analysis must not be applied to all of bridge types and structural conditions from the aspect of its property; firstly, a basic vibration mode shown below should be confirmed based, using eigenvalue analysis, whether the deformation shape obtained by static analysis are similar to the basic vibration mode, which can be defined as first mode, or not. Unless synchronization can be seen between them, response spectrum analysis with eigenvalue analysis based on multimode elastic analysis or time history analysis should be applied.

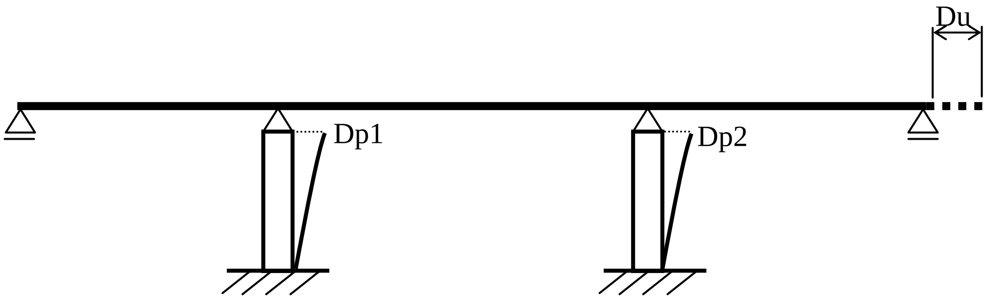


Figure 2.1.2-1 Example of Basic Vibration Mode (Longitudinal Direction)

2.1.3 Eigenvalue Analysis (Single or Multimode Elastic Method)

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.

(i) 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 above mentioned equation 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 is the inverse number of the natural frequency.

$$\left|[K] - \omega^2 [M]\right| = 0 \qquad \text{Eq.}$$

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

(ii) Participation factor and Effective mass

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

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

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" th 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 case of modal analysis, accurate analytical results are generally obtained on the basis of adoption of the vibration modes including generally 90% of total mass. Thus, the participation factor and the effective mass can present useful indicator of dominant property regarding mass of each vibration mode such as which mass, which direction, how much amount.

$$m_j = (\{\Phi_j\}^T [M]\{L\})^2 / \overline{M}_j \qquad \text{Eq.}$$

m_j : Effective mass

(iii) 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 $\{\Phi_j\}$ 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 modal analysis method.

2.1.4 Response Spectrum Analysis

Response spectrum analysis method can be defined as one of dynamic analytical approach under elastic conditions; maximum responses of structural members are easily confirmed for seismically irregular bridges.

When standard vibration mode vector can be obtained based on previously explained eigenvalue analysis, the modal analysis for the mode vector corresponding to the natural period and damping factors can be easily implemented and can compute maximum response of structural members.

Dynamic analysis consists of this response spectrum analysis and time history response analysis for which response can be computed historically by inputting wave shape historical seismic motion. However, it is not usually necessary to obtain complicated historical responses on seismic design but is frequently necessary to obtain only maximum responses of the structural members. Therefore, maximum responses for each vibration mode under a seismic motion are preliminarily prepared until a certain mode, and then the spectrum processed and organized by natural period and mode damping factor is absolutely response spectrum.

Natural modes can be called as 1st mode, 2nd mode and 3rd mode in the order corresponding to longer natural period or shorter natural vibration.

Where, the vibration modes that should be preliminarily prepared are to be adopted until the mode that over 90% of effective mass against total mass has been accumulated. For Mawo bridges, the superstructure type is categorized in girder type bridge not cable supported bridge; hence, 1st mode shape may be dominant mode. Therefore, it is not necessary to consider high modes like suspension bridges.

For superposition of maximum responses of multiple-mass system using response spectrum of each mode, SRSS, Square Rood of Sum of Square, and CQC method, Complete Quadratic Combination are worldwidely utilized.

2.1.5 Time History Analysis

Time history analysis is one of dynamic analytical approaches to obtain historical responses by inputting historical wave seismic motion. Generally, fiber elements are utilized for analytical model that may be complicated model because historical curves should be inputted into each element. However, in contrast to above mentioned response spectrum method, more advanced and high freedom dynamic behaviors can be obtained because the vibration system under material non-linearity as well as nonlinear historical properties of piers and rubber bearings can be accurately incorporated into the fiber elements.

2.1.6 Applied Methodology of Seismic Analysis

Based on the DPWH BSDS, application of dynamic analysis to obtain definite solution of seismic behavior is highly recommended, mainly specified in SECTION 4.

Because Mawo bridge is one of multi-span girder type bridges, the seismic behavior may not behave irregularly; static analysis or single-mode elastic method may be applied to the design; however, damping constants of all the structural elements as well as LRB should be appropriately incorporated into the analysis using mode-damping factor in the dynamic method; otherwise, validity and compliance of historical properties of various members and devices to be imputed into fiber model, among structural details of mainly RC piers, hysteresis curves affected significantly by the structural details and newly determined seismic motion, has not be organized completely in DPWH BSDS. Besides, both response spectrum analysis using multimode elastic method considering LRB damping constant and time history analysis under elasto-plastic method are not common methodology for seismic design; introduction of basic dynamic analysis may be highly useful for general designers and engineers in Philippines.

Consequently, for the seismic analysis for the design example of Mawo bridge, the response spectrum analysis with multimode elastic method is selected.

2.2 Analysis Model

2.2.1 Global Analysis Model

In this project, as seismic analysis, modal response spectrum analysis is conducted for seismic design. Based on the response results, various structural members can be determined such as piers, foundations, bearings and expansion joints. In the design example of Mawo bridge, for the modal response spectrum analysis, STAAD ProV8i (Bentley System. Inc, USA) is utilized because this software is commonly utilized and distributed in Philippines.
Based on the results of the outline design of superstructure such as member dimension and mass, analytical model and results of modal response spectrum analysis are explained in this item.
Besides, in this design example, abutments are not modeled in the seismic analysis because abutments may have enough strength and stiffness fixed by grounds for seismic vibration; if abutments are modeled in the analysis, excess damping efficiency would be expected to the whole of structural responses.

- Seismic Analysis : Response Spectrum Analysis based on Modal Eigenvalue Analysis
- Software : STAAD ProV8i (Bentley System. Inc, USA)
- Superstructure Type: PC Fin-Back Box Girder
- Bridge Length : L=205m
- Angle of Alignment: 90 Degrees
- Global Analysis Model and Nodes Coordination :

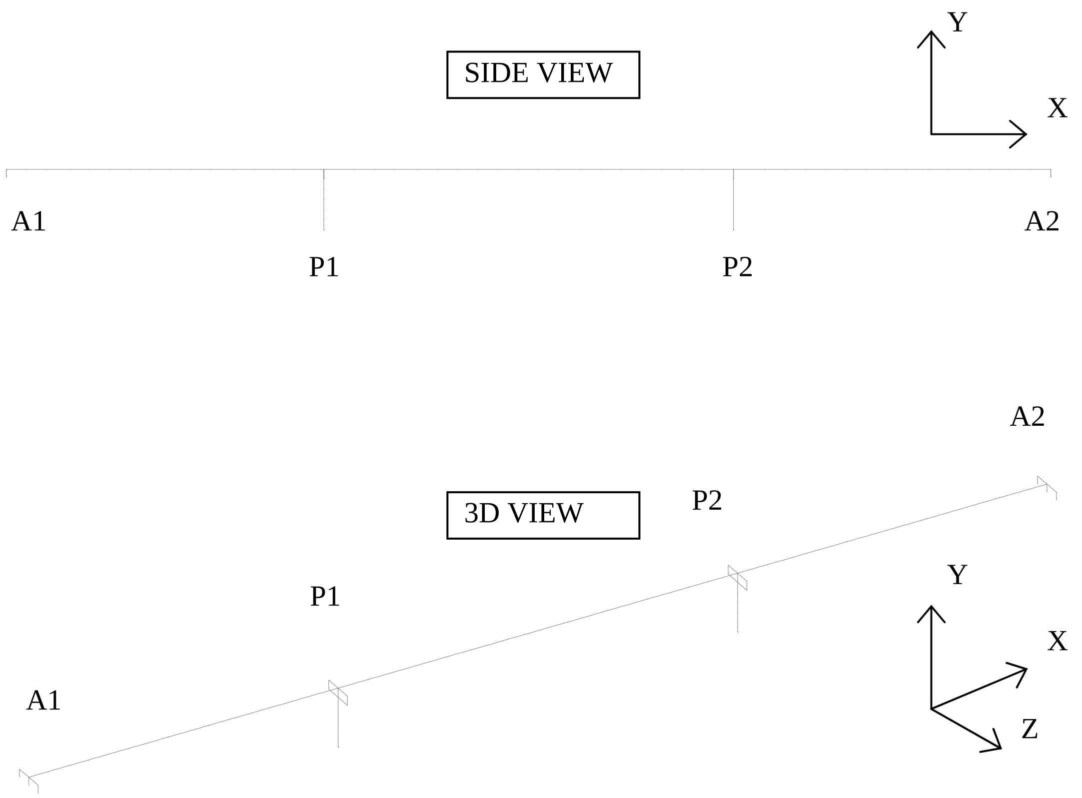


Figure 2.2.1-1 Global Analysis Model

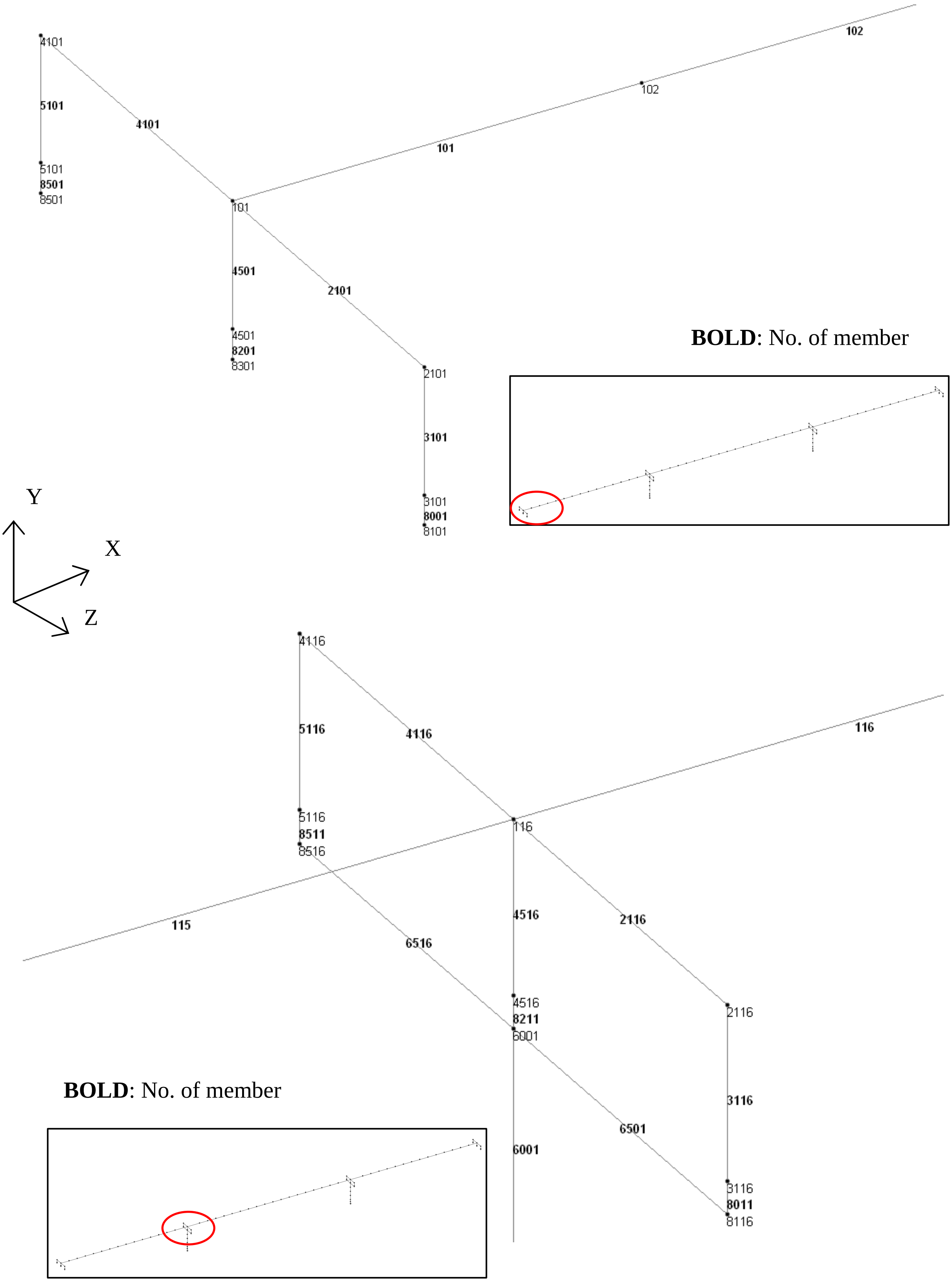
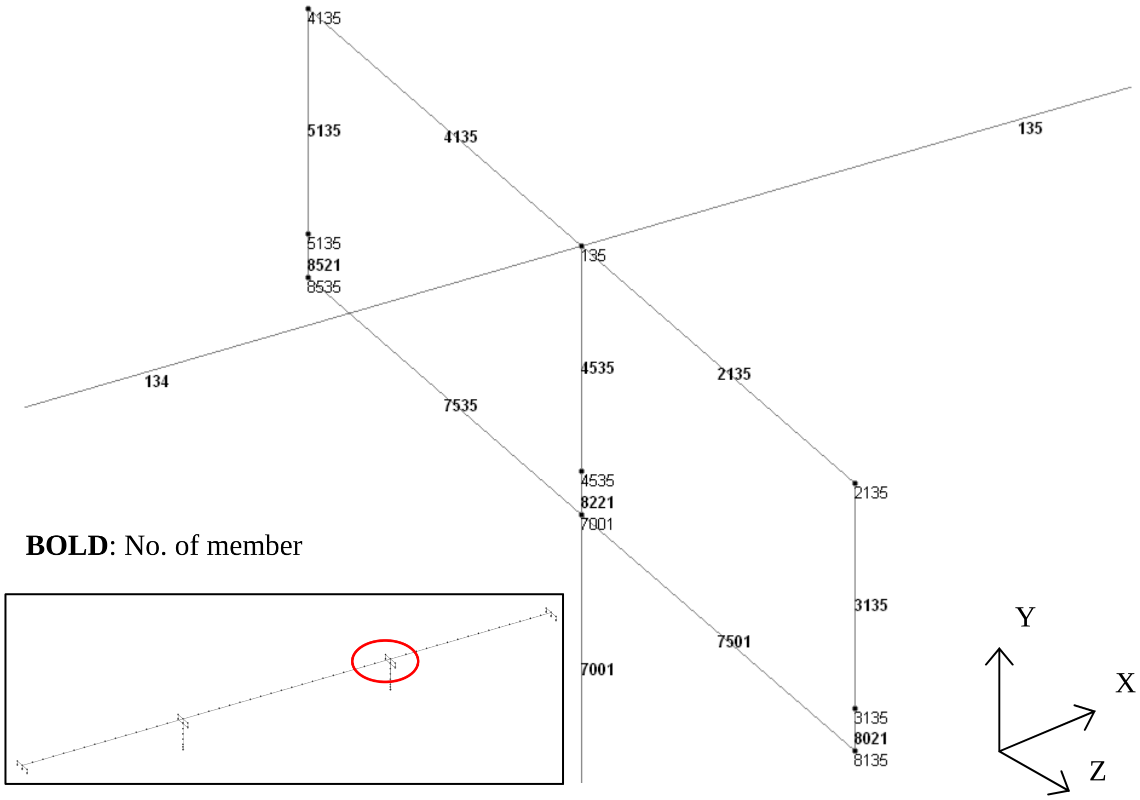


Figure 2.2.1-2 Analysis Model (Superstructure at A1 and P1)



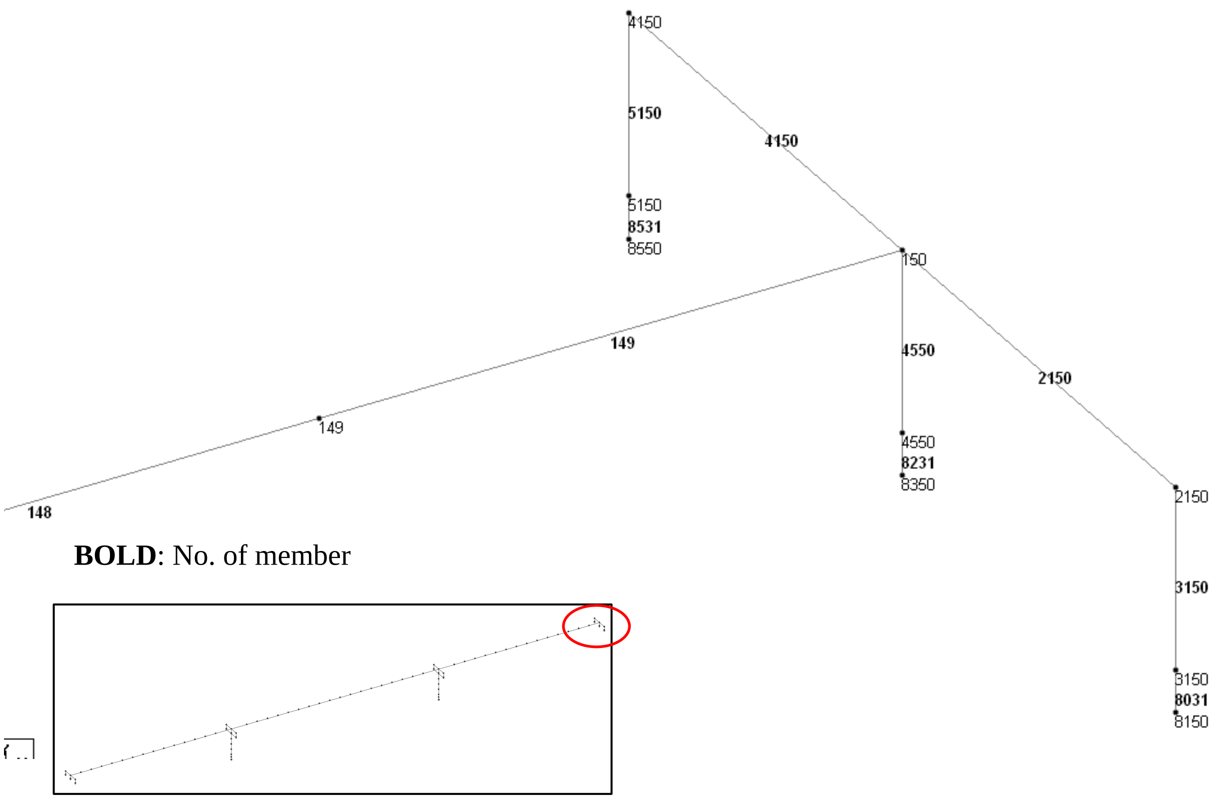


Figure 2.2.1-3 Analysis Model (Superstructure at P2 and A2)

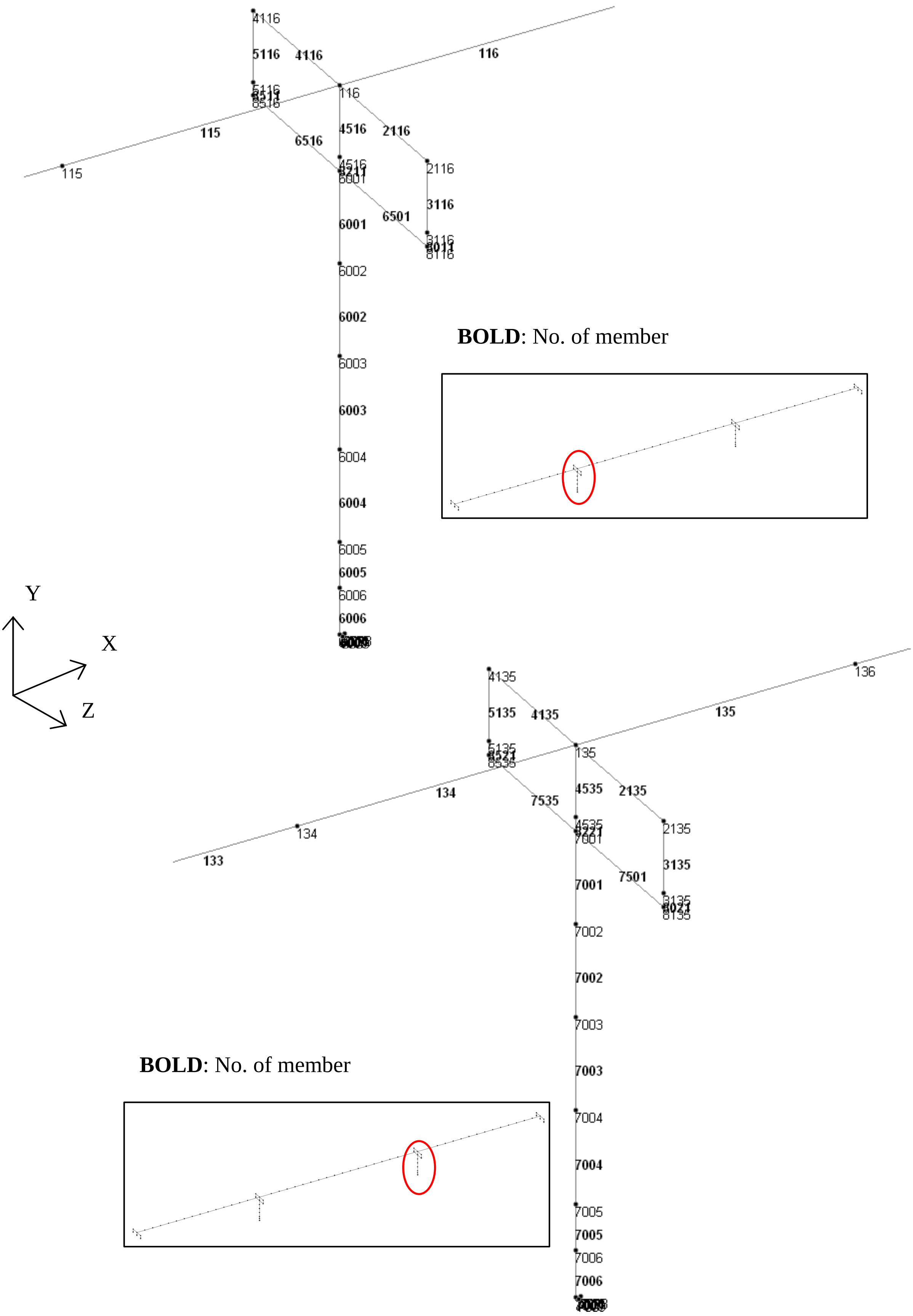


Figure 2.2.1-4 Analysis Model (Substructure P1 and P2)

2.2.2 Member Sectional Properties

Following table shows node number and their coordination inputted in the analysis.

Table 2.2.2-1 Node Coordination (Girder)

No.	X(m)	Y(m)	Z(m)
126	104.000	1.250	0.000
127	108.000	1.250	0.000
128	112.000	1.250	0.000
129	116.000	1.250	0.000
130	120.000	1.250	0.000
131	124.000	1.250	0.000
132	128.000	1.250	0.000
133	132.000	1.250	0.000
134	136.000	1.250	0.000
135	142.000	1.250	0.000
136	148.000	1.250	0.000
137	152.000	1.250	0.000
138	156.000	1.250	0.000
139	160.000	1.250	0.000
140	164.000	1.250	0.000
141	168.000	1.250	0.000
142	172.000	1.250	0.000
143	176.000	1.250	0.000
144	180.000	1.250	0.000
145	184.000	1.250	0.000
146	188.000	1.250	0.000
147	192.000	1.250	0.000
148	196.000	1.250	0.000
149	200.000	1.250	0.000
150	203.120	1.250	0.000

No.	X(m)	Y(m)	Z(m)
101	0.880	1.250	0.000
102	4.000	1.250	0.000
103	8.000	1.250	0.000
104	12.000	1.250	0.000
105	16.000	1.250	0.000
106	20.000	1.250	0.000
107	24.000	1.250	0.000
108	28.000	1.250	0.000
109	32.000	1.250	0.000
110	36.000	1.250	0.000
111	40.000	1.250	0.000
112	44.000	1.250	0.000
113	48.000	1.250	0.000
114	52.000	1.250	0.000
115	56.000	1.250	0.000
116	62.000	1.250	0.000
117	68.000	1.250	0.000
118	72.000	1.250	0.000
119	76.000	1.250	0.000
120	80.000	1.250	0.000
121	84.000	1.250	0.000
122	88.000	1.250	0.000
123	92.000	1.250	0.000
124	96.000	1.250	0.000
125	100.000	1.250	0.000

No.	X(m)	Y(m)	Z(m)
2101	0.880	1.250	4.900
2116	62.000	1.250	4.900
2135	142.000	1.250	4.900
2150	203.120	1.250	4.900
3101	0.880	0.000	4.900
3116	62.000	-0.295	4.900
3135	142.000	-0.295	4.900
3150	203.120	0.000	4.900
4101	0.880	1.250	-4.900
4116	62.000	1.250	-4.900
4135	142.000	1.250	-4.900
4150	203.120	1.250	-4.900
4501	0.880	0.000	0.000
4516	62.000	-0.295	0.000
4535	142.000	-0.295	0.000
4550	203.120	0.000	0.000
5101	0.880	0.000	-4.900
5116	62.000	-0.295	-4.900
5135	142.000	-0.295	-4.900
5150	203.120	0.000	-4.900

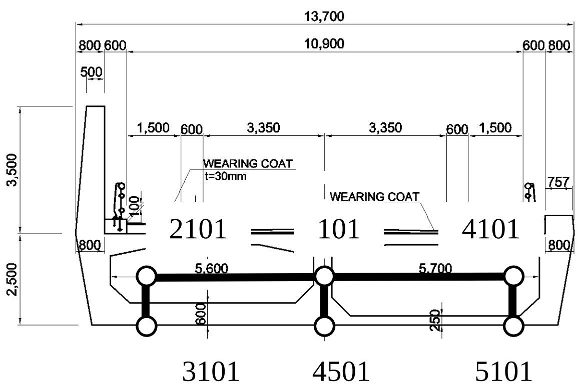


Table 2.2.2-2 Node Coordination (Pier)

P1

No.	X(m)	Y(m)	Z(m)
6001	62.000	-0.590	0.000
6002	62.000	-2.590	0.000
6003	62.000	-4.590	0.000
6004	62.000	-6.590	0.000
6005	62.000	-8.590	0.000
6006	62.000	-9.590	0.000
6007	62.000	-10.590	0.000
6008	62.000	-10.690	0.000
8116	62.000	-0.590	4.900
8516	62.000	-0.590	-4.900

P2

No.	X(m)	Y(m)	Z(m)
7001	142.000	-0.590	0.000
7002	142.000	-2.590	0.000
7003	142.000	-4.590	0.000
7004	142.000	-6.590	0.000
7005	142.000	-8.590	0.000
7006	142.000	-9.590	0.000
7007	142.000	-10.590	0.000
7008	142.000	-10.690	0.000
8135	142.000	-0.590	4.900
8535	142.000	-0.590	-4.900

Table 2.2.2-3 Node Coordination (Abutment))

A1

No.	X(m)	Y(m)	Z(m)
8101	0.880	-0.295	4.900
8301	0.880	-0.295	0.000
8501	0.880	-0.295	-4.900

A2

No.	X(m)	Y(m)	Z(m)
8150	203.120	-0.295	4.900
8350	203.120	-0.295	0.000
8550	203.120	-0.295	-4.900

Table 2.2.2-4 Structural Material

Item	Symbol	Girder	Pier
Compressive Strength at 28 days	f’c	50 MPa	24MPa
Elastic Modulus	Ec	31,000 MPa	25,000 MPa
Shear Modulus	Gc	13,480 MPa	10,870 MPa

Following the table shows the member property of superstructure of Mawo bridge. In this example of outline design, constant values are adopted based on the results of the superstructure design because the vibration behavior may be quite simple.

Member Number 101 to 149
A = 16.33 m^2
Iz = 14.02 m^4
Iy = 363.48 m^4
Jx = 44.0 m^4

Except the girder property, following rigid elements should be adequately modeled.

Table 2.2.2-5 Member Property (Rigid Elements of Superstructure)

No.	node		A	Iz	Iy	Jx	Note
	Start	End	(m2)	(m4)	(m4)	(m4)	
2101	101	2101	∞	∞	∞	∞	rigid
2116	116	2116	∞	∞	∞	∞	rigid
2135	135	2135	∞	∞	∞	∞	rigid
2150	150	2150	∞	∞	∞	∞	rigid
3101	2101	3101	∞	∞	∞	∞	rigid
3116	2116	3116	∞	∞	∞	∞	rigid
3135	2135	3135	∞	∞	∞	∞	rigid
3150	2150	3150	∞	∞	∞	∞	rigid
4101	101	4101	∞	∞	∞	∞	rigid
4116	116	4116	∞	∞	∞	∞	rigid
4135	135	4135	∞	∞	∞	∞	rigid
4150	150	4150	∞	∞	∞	∞	rigid
4501	101	4501	∞	∞	∞	∞	rigid
4516	116	4516	∞	∞	∞	∞	rigid
4535	135	4535	∞	∞	∞	∞	rigid
4550	150	4550	∞	∞	∞	∞	rigid
5101	4101	5101	∞	∞	∞	∞	rigid
5116	4116	5116	∞	∞	∞	∞	rigid
5135	4135	5135	∞	∞	∞	∞	rigid
5150	4150	5150	∞	∞	∞	∞	rigid

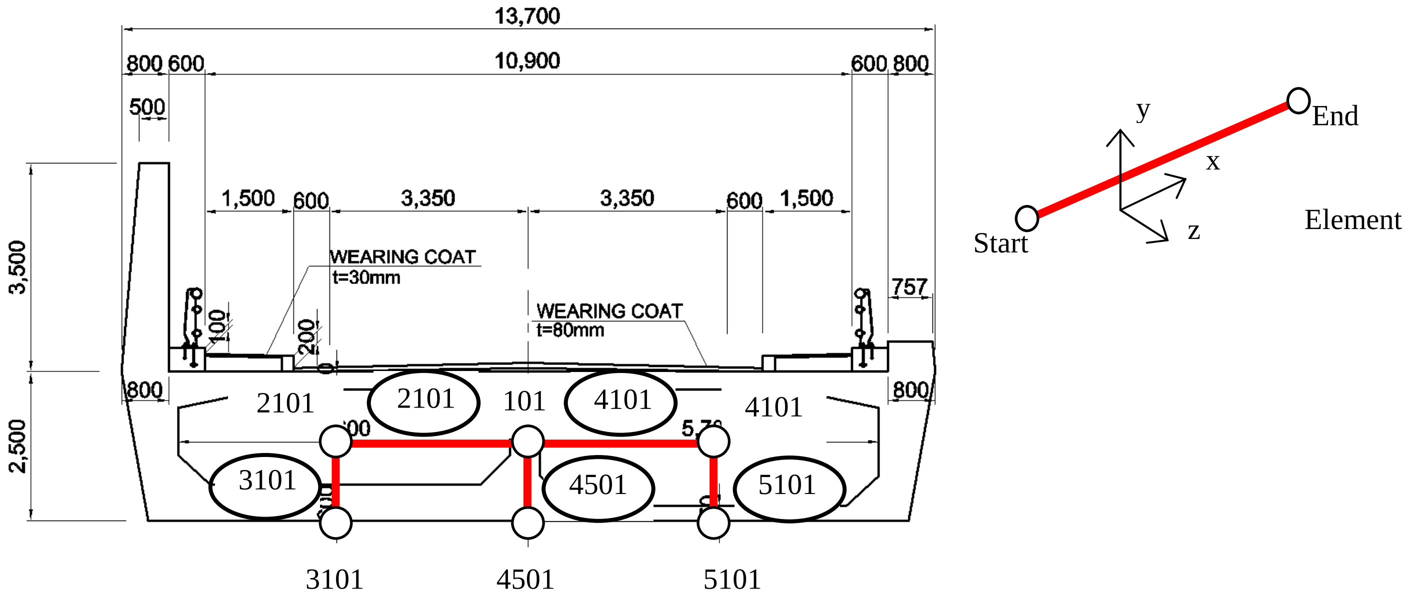


Figure 2.2.2-1 Model of Superstructure

Substructures also modeled by beam element simply. In seismic analysis, nonlinear effects of which decrease stiffness, such as inelastic deformation and cracking, should be considered. Reinforced concrete piers in Seismic Performance Zones 2, 3 and 4 should be analyzed using cracked section properties. For this purpose, a moment of inertia equal to one-half that of the uncracked section may be used, specified in 4.5.3 of SPWH BSDS.

Table 2.2.2-6 Member Property (Piers)

No.	node		A	Iz	Iy	Jx	Note
	Start	End	(m2)	(m4)	(m4)	(m4)	
6001	6001	6002	36.6	11.48	263.98	66.86	
6002	6002	6003	36.6	11.48	263.98	66.86	
6003	6003	6004	36.6	11.48	263.98	66.86	
6004	6004	6005	36.6	11.48	263.98	66.86	
6005	6005	6006	∞	∞	∞	∞	Pile Cap
6006	6006	6007	∞	∞	∞	∞	Pile Cap
6007	6007	6008	∞	∞	∞	∞	Virtual
6501	8116	6001	∞	∞	∞	∞	rigid
6516	8516	6001	∞	∞	∞	∞	rigid

No.	node		A	Iz	Iy	Jx	Note
	Start	End	(m2)	(m4)	(m4)	(m4)	
7001	7001	7002	36.6	11.48	263.98	66.86	
7002	7002	7003	36.6	11.48	263.98	66.86	
7003	7003	7004	36.6	11.48	263.98	66.86	
7004	7004	7005	36.6	11.48	263.98	66.86	
7005	7005	7006	∞	∞	∞	∞	Pile Cap
7006	7006	7007	∞	∞	∞	∞	Pile Cap
7007	7007	7008	∞	∞	∞	∞	Virtual
7501	8135	7001	∞	∞	∞	∞	rigid
7535	8535	7001	∞	∞	∞	∞	rigid

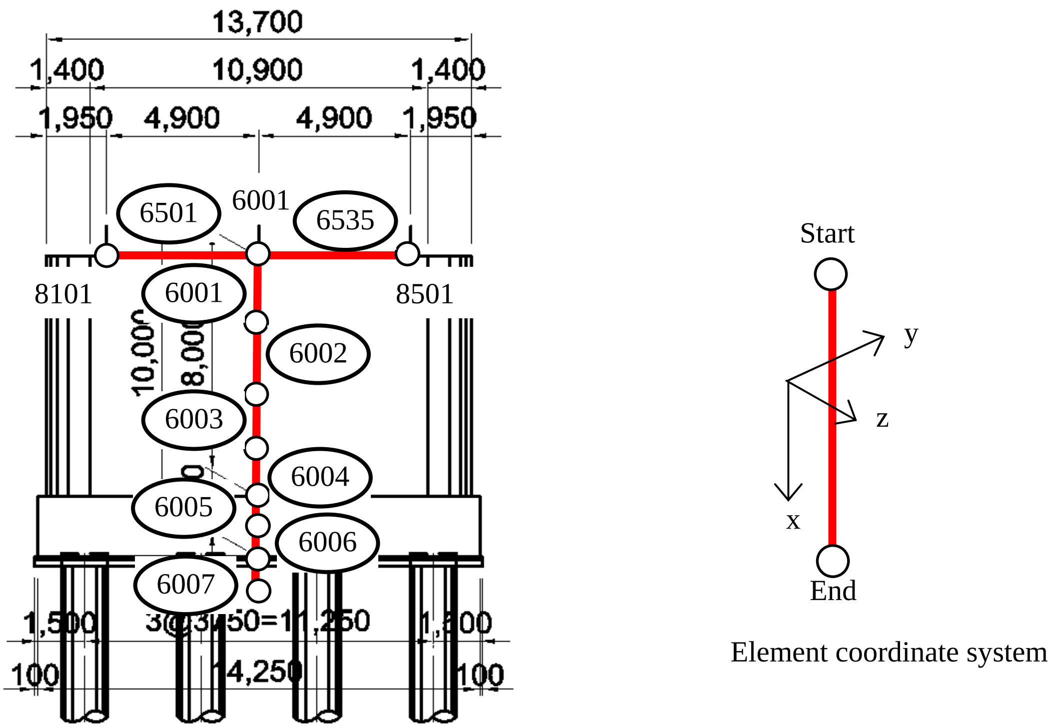


Figure 2.2.2-2 Model of Piers

The structural mass are inputted for the following sections.

i) Superstructure
Distributed loads are applied to the all nodes of superstructures as: 412.195 kN/m

ii) Substructure
The mass of column per 2m height is to be following value. 1756 kN/ 2m height
Also the footing: 6905 kN/ 2m height

2.2.3 Model of Bearing

Force distribution method by laminated rubber bearings (LRB) shown in the following figure are commonly utilized in viaducts and bridges in Japan as efficient devices to achieve appropriate seismic design.

Figure 2.2.3-1 Laminated Rubber Bearing

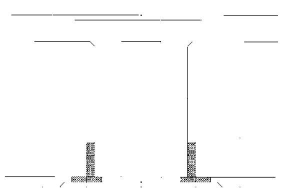
This bearing consists of rubber and steel plate layers. By changing the stiffness of the laminated rubber, such as thickness, number of layers and sizes, seismic horizontal forces can be freely and evenly adjusted to substructures. Therefore, the boundary condition between superstructure and substructure is defined as "E" that means "elastic".

Figure 2.2.3-2 Steel Bearing

Otherwise, in Philippines, commonly thin-rubber bearing with anchor bars are utilized as bearing. By this bearing, only two ways of the boundary condition such as "Fix" or "Move" can be applied, which means that controlling of horizontal seismic forces or contribution forces to substructures depends on not horizontal stiffness of bearing but just only the period of its dynamic properties.

In this outline design of Mawo bridge, a technical comparison study among laminated rubber bearing, thin-rubber bearing with anchor bars and steel bearing is studied from the point of view of seismic behavior, shown as following table.

Table2.2.3-1 Comparison Study of Bearing in Mawo Bridge

Bearing	Results of Evaluation
Laminated Rubber Bearing Under Force Distribution Method 	Boundary Condition: LD: Elastic (A1-P1-P2-A2), TD: Fix (A1,A2) Elastic (P1,P2) Time Period LD: 1.31S, TD: 1.13s Modal Dumping of 1st mode LD: 3%. TD: 2% Total Horizontal Forces of Superstructure using Modal Dumping LD: 71000kN of 84500kN, TD: 74000kN of 84500kN Seismic Force Distribution LD: A1:P1:P2:A2=1:1:1:1, TD: A1:P1:P2:A2=1:1:1:1
Pad Rubber Bearing with Dowel Under Not Force Distribution 	Boundary Condition: LD: Move (A1, A2), Fix (P1, P2), TD: Fix (A1-P1-P2-A2) Time Period LD: 0.8S, TD: 0.9s Modal Dumping of 1st mode LD: 2%. TD: 2% Total Horizontal Forces of Superstructure using Modal Dumping LD: 88000kN of 84500kN, TD: 92000kN of 84500kN Seismic Force Distribution LD: A1:P1:P2:A2=0:1:1:0,
Steel Bearing Under Not Force Distribution	Boundary Condition: LD: Move (A1, A2), Fix (P1, P2), TD: Fix (A1-P1-P2-A2) Time Period

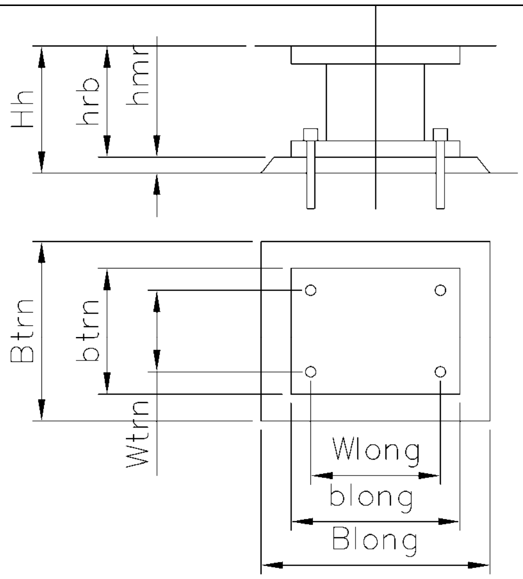
<table>
<tr><td></td><td>LD: 0.8S, TD: 0.9s
Modal Dumping of 1st mode
LD: 2%. TD: 2%
Total Horizontal Forces of Superstructure using Modal Dumping
LD: 88000kN of 84500kN, TD: 92000kN of 84500kN
Seismic Force Distribution
LD: A1:P1:P2:A2=0:1:1:0,</td></tr>
<tr><td colspan="2">By using LRB, the period of transversal 1st mode achieves beyond 1.3s, which is much longer than the structure applying Pad Rubber bearing and Steel bearing. Therefore, the seismic forces based on design spectrum are significantly reduced by extended period. And also, dominant deformation of the 1st mode is obviously longitudinal direction of superstructure, which caused by shearing deformation of the LRB; hence, the modal damping of 1st mode can achieve 3% despite just only 2%, modal damping, which is the damping factor of steel structure, of the structure using Pad Rubber bearing or Steel bearing.
Consequently the total horizontal forces of superstructure using LRB is greatly decreased comparing to the structure using common bearing due to extended period and higher structural mode dumping. Additionally, the seismic forces can be distributed evenly by LRB. Therefore, structurally, superiority of application of LRB is extremely high.</td></tr>
</table>

* LD: Longitudinal Direction, TD: Transversal Direction

In this outline design the following LRB is applied.

Table2.2.3-2 LRB of Mawo Bridge

Supports	Nos.	Rub. Dimension	Rub. Thickness	G
Abutment 1	3	1500mmx1500mm	37mmx5layers	1.4 N/mm2
Pier 1	3	1500mmx1500mm	37mmx5layers	1.4 N/mm2
Pier 2	3	1500mmx1500mm	37mmx5layers	1.4 N/mm2
Abutment 2	3	1500mmx1500mm	37mmx5layers	1.4 N/mm2

<table>
<tr><td>
</td><td>For side view:
- hrb: Height of bearing: 560mm
- hmr: Thickness of mortar: 30mm
- Hh: Total height of bearing: 590mm

For spaces of anchor bars
- Wlong: Space for longitudinal direction
1500mm
- Wtrn: Space for transversal direction
1500mm

For width of sole plates on substructure
- Blong: Including mortar: 1850mm
- blong: Bearing width: 1750mm
- Btrn: Including mortar: 2250mm
- btrn: Bearing width: 2150mm</td></tr>
</table>

Table2.2.3-3 Dimension of LRB of Mawo Bridge

In the analysis, above bearing properties are imputed as nodal spring, calculated as follows.

< Calculation of Nodal Spring of LRB >

$$K = \frac{G \times B \times B}{t \times n} \qquad \text{Eq.}$$

K: Nodal Spring of LRB (kN/m), B: Width of Rubber, t: Thickness of rubber
n: Number of layers

Where, for the LRB of Mawo bridge, the following calculation can be given by above equation.

$$K = \frac{1.4 \times 1500 \times 1500}{37 \times 5} = 17027 kN / m^2 \qquad \text{Eq.}$$

Such the nodal spring is available to be inputted into the Program "STAAD" with specified dimensions, such as the following table.

Table2.2.3-4 LRB Spring of Mawo Bridge

No.	node		Kx	Ky	Kz	KRx	KRx	KRz
	Start	End	(kN/m)	(kN/m)	(kN/m)	(kNm/rad)	(kNm/rad)	(kNm/rad)
8001	3101	8101	1.00E+09	17,030	1.00E+09	1.00E+09	1.00E-09	1.00E-09
8011	3116	8116	1.00E+09	17,030	17,030	1.00E+09	1.00E-09	1.00E-09
8021	3135	8135	1.00E+09	17,030	17,030	1.00E+09	1.00E-09	1.00E-09
8031	3150	8150	1.00E+09	17,030	1.00E+09	1.00E+09	1.00E-09	1.00E-09
8201	4501	8301	1.00E+09	17,030	1.00E+09	1.00E+09	1.00E-09	1.00E-09
8211	4516	6001	1.00E+09	17,030	17,030	1.00E+09	1.00E-09	1.00E-09
8221	4535	7001	1.00E+09	17,030	17,030	1.00E+09	1.00E-09	1.00E-09
8231	4550	8350	1.00E+09	17,030	1.00E+09	1.00E+09	1.00E-09	1.00E-09
8501	5101	8501	1.00E+09	17,030	1.00E+09	1.00E+09	1.00E-09	1.00E-09
8511	5116	8516	1.00E+09	17,030	17,030	1.00E+09	1.00E-09	1.00E-09
8521	5135	8535	1.00E+09	17,030	17,030	1.00E+09	1.00E-09	1.00E-09
8531	5150	8550	1.00E+09	17,030	1.00E+09	1.00E+09	1.00E-09	1.00E-09

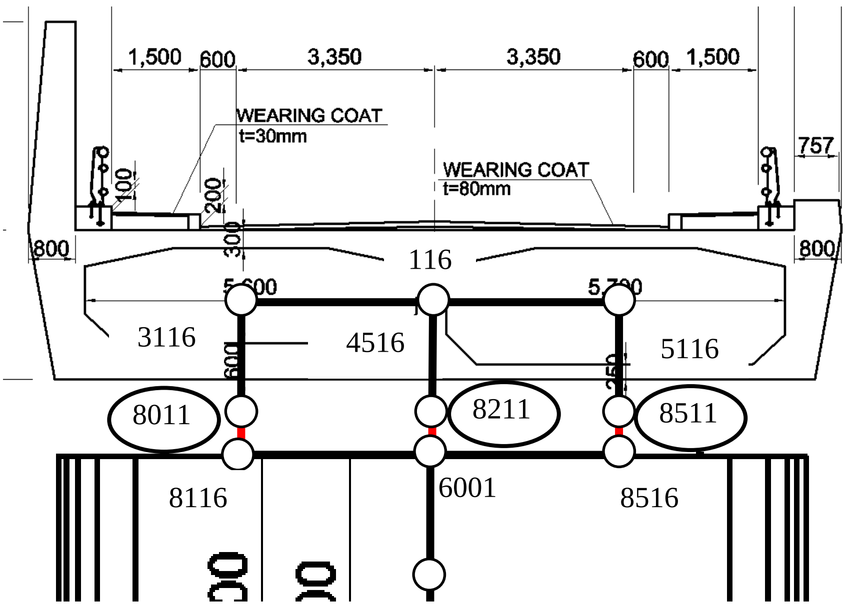
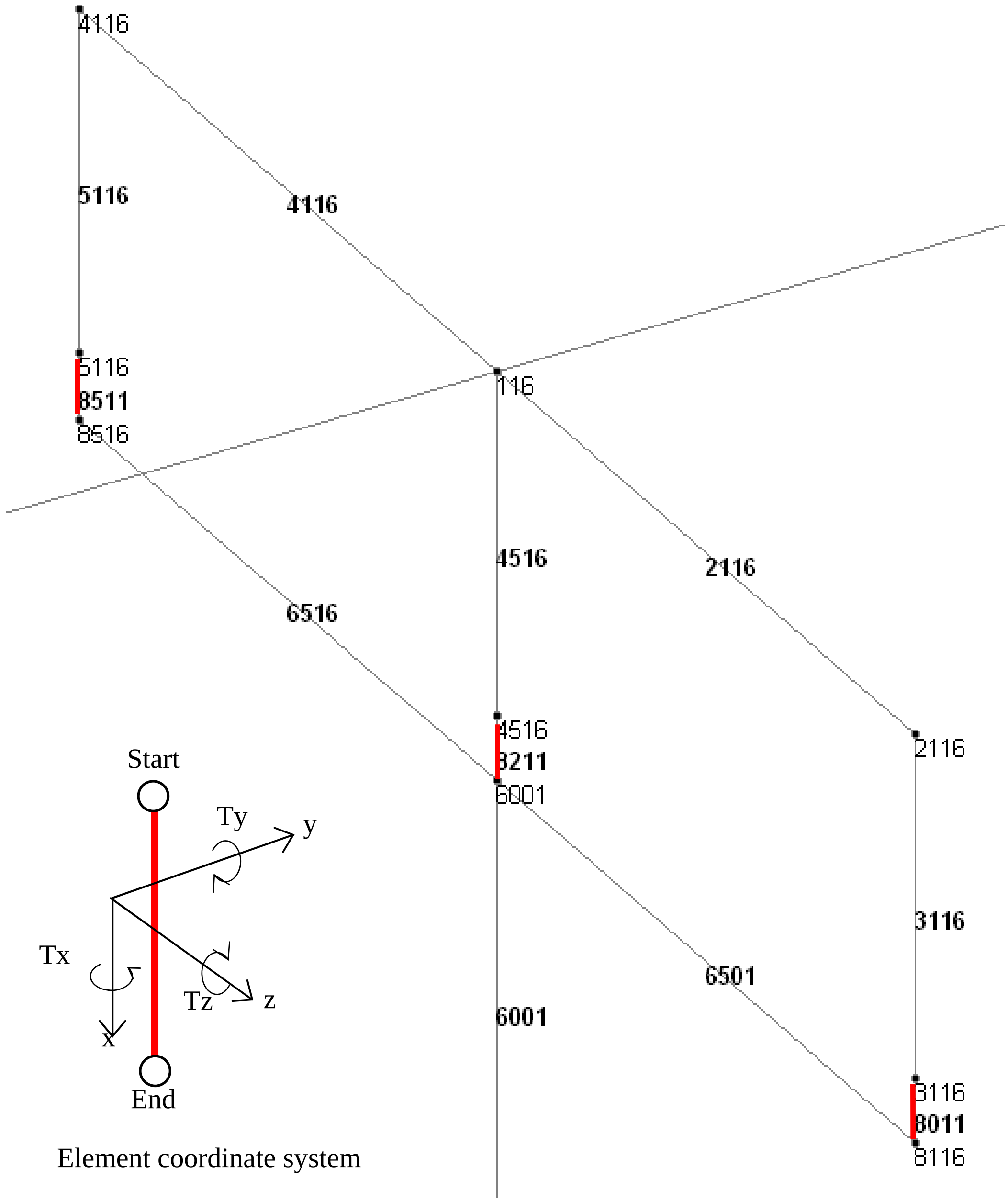


Figure 2.2.3-3 Model of LRB Bearing (1)



Node 8511, 8211 and 8011 have nodal spring

Figure 2.2.3-4 Model of LRB Bearing (2)

2.2.4 Model of Footing with Foundation Springs

Generally, piles and ground conditions are not modeled by beam elements in structural analysis but are modeled by lumped spring models consisting of piles and ground conditions. Therefore, such the lamped spring should be inputted to the structural analysis in accordance with the rule of the software. For the detail examination how to obtain the foundation springs, refer to see the Chapter 3 in this design example

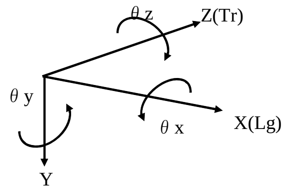
i) Composite Spring Matrix
Composite spring matrix consists of 6x6 matrix, which perfectly model the piles and ground conditions. However, for some software including STAAD, this matrix form can not easily inputted.

Table 2.2.4-1 Composite Spring Matrix

◆COMPOSITE SPRING MATRIX

Mawo P1 CHODAI's Software "NONPIER" OK/ Some Common Commercialized Sotware NOT Inputable

	δx	δy	δz	θx	θy	θz
Fx	7.669E+06					-1.290E+07
Fy		9.046E+06				
Fz			7.660E+06	1.290E+07		
Mx			1.290E+07	2.017E+08		
My					2.068E+08	
Mz	-1.290E+07					1.284E+08



Mawo P2

	δx	δy	δz	θx	θy	θz
Fx	7.012E+06					-1.290E+07
Fy		7.667E+06				
Fz			7.658E+06	1.290E+07		
Mx			1.290E+07	2.067E+08		
My					1.000E+09	
Mz	-1.290E+07					1.094E+08

ii) Simplified Spring
The composite spring matrix consists of symmetrical matrix. Therefore, in order to simplify input method, sometimes following the values extracted from the symmetrical matrix are sometimes utilized. The extracted values should be the values on the diagonal lines of the matrix. The composite spring matrix can be defined as a symmetrical matrix but can be unboundedly diagonal matrix. Therefore, the values on the diagonal lines can work as primal springs and the others may affect as supplemental springs from the point of view of structural mechanics. The results of the dynamic analysis, the natural period may be slightly shorter than the composite spring matrix.

Table 2.2.4-2 Composite Spring Matrix

◆REPRESENTATIVE SPRING

Mawo P1 CHODAI's Software "NONPIER" OK/ Common Commercialized Sotwar

	KX	KY	KZ	TX	TY	TZ
	(kN/m)	(kN/m)	(kN/m)	(kNm/rad)	(kNm/rad)	(kNm/rad)
JRA	7.410E+06	6.422E+06	7.410E+06	1.581E+08	1.000E+09	1.054E+08
FB-Pier	7.669E+06	9.046E+06	7.660E+06	2.017E+08	2.068E+08	1.284E+08

Mawo P2

	KX	KY	KZ	TX	TY	TZ
	(kN/m)	(kN/m)	(kN/m)	(kNm/rad)	(kNm/rad)	(kNm/rad)
JRA	7.403E+06	8.895E+06	7.403E+06	2.015E+08	1.000E+09	1.285E+08
FB-Pier	7.012E+06	7.667E+06	7.658E+06	2.067E+08	1.662E+08	1.094E+08

iii) Fixed supports
This method is the most simple method for the modeling of foundations. Obviously, the natural period may be shorter than any other models with foundation springs.

Here, for above three methods, following the comparison study is shown in order to determine the method to be applied for the dynamic analysis by STAAD. Where, because composite spring matrix is not allowed to be inputted by STAAD, the comparison study is conducted by "RITTAI", produced by Chodai, which have a lot of actual results from common structure to long span cable supported bridge in the world.

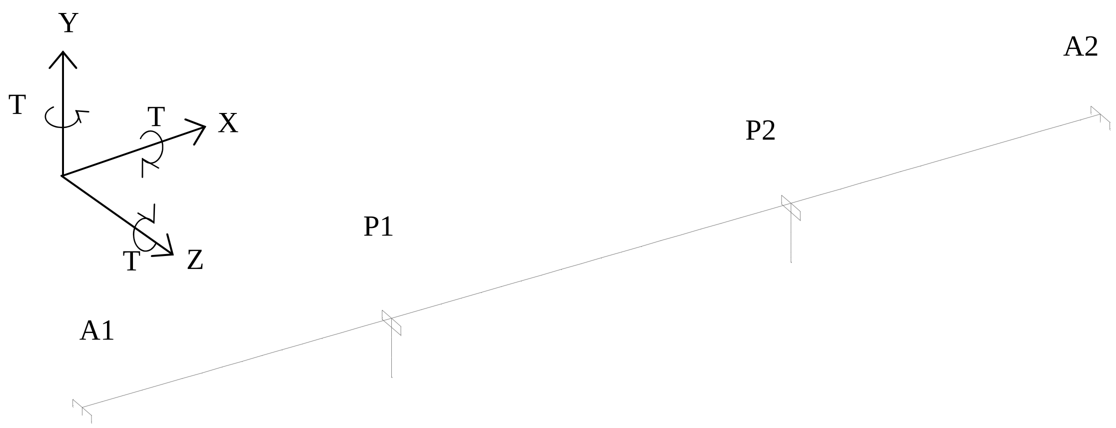
Table 2.2.4-3 Comparison Study of Foundation Springs

MODEL-1 Composite Spring Matrix	**MODEL-2** Simplified Spring	**MODEL-3** Fix
Comparison of Natural Period		
T = 1.319 s	**T = 1.310 s**	T = 1.294s
+0.68%	**1.00**	-1.22%
Comparison of Response Displacement		
Girder End: D= 36.4cm	**Girder End: D= 36.3 cm**	Girder End: D= 36.2cm
+0.03%	**1.00**	-0.03%

As shown in the comparison study, seismically the fixed model is the most conservative method of all. Naturally the natural period, which affects critically response values, is the shortest of all. Meanwhile, the differences between others such as the results of composite spring matrix and simplified springs are just only 0.68%, otherwise, the differences between simplified springs and fixed model are 1.22%. Therefore, when the modes, in which foundations are preferentially moved, are located in higher orders and if the software can not allow complicated data input, the structural modeling of simplified spring is structurally and seismically acceptable from the results, shown as follows.

Table 2.2.4-4 Supporting Condition of Piers

Pier No.	Node No.	KX (kN/m)	KY (kN/m)	KZ (kN/m)	KRX (kNm/rad)	KRY (kNm/rad)	KRZ (kNm/rad)
P1	6007	7.67E+06	FIX	7.66E+06	2.02E+08	FIX	1.28E+08
P2	7007	7.01E+06	FIX	7.66E+06	2.07E+08	FIX	1.09E+08



2.3 Results of Eigenvalue Analysis

i) Results of Eigenvalue Analysis

The following table shows the results of eigenvalue analysis until 10th modes.

Table 2.3-1 Results of Eigenvalue Analysis

Modes	Dominant Move	Frequency (Hz)	Period (s)	Ratio of Effective Mass		Mode Damping
				Longitudinal	Transversal	
1	Girder Lng.	0.763	1.310	**0.764**	0.000	0.031
2	Girder Trn.	0.885	1.130	0.000	**0.617**	0.021
3	Girder Trn.	2.619	0.382	0.000	0.088	0.012
4	Girder Trn.	5.658	0.177	0.000	0.071	0.010
5	P2 Long.	7.325	0.137	0.097	0.000	0.083
6	P1 Long.	7.712	0.130	0.099	0.000	0.082
7	Girder Lng.	8.628	0.116	0.001	0.000	0.010
8	P1 P2 Trn.	9.452	0.106	0.000	0.001	0.093
9	P1 P2 Trn.	9.584	0.104	0.000	0.001	0.093
10	All Trn.	10.059	0.099	0.000	0.000	0.011

According to the results, predominant mode for longitudinal direction shown below obviously obtained at 1st mode whose period is 1.31s and effective mass ration is 76% of modes for longitudinal direction.

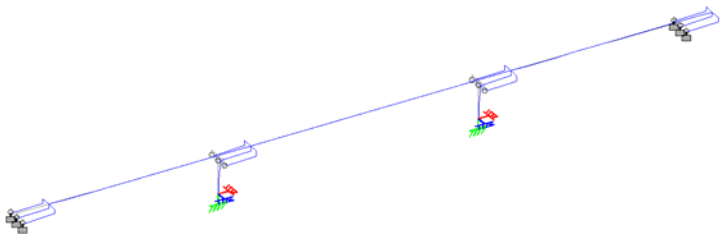


Figure 2.3-1 Predominant 1st Mode for Longitudinal Direction

Therefore, the 1st mode for longitudinal direction is so important mode shape, which have enough effective mass ratio. And for the 1st mode for transversal direction shown below is the mode with the effective mass ratio of 0.62 and with the period of 1.13s for transversal direction. Both of the behaviors of longitudinal and transversal direction are efficiently functioned against strong seismic forces, using Force Distribution Bearings and appropriate dumping coefficient of them.

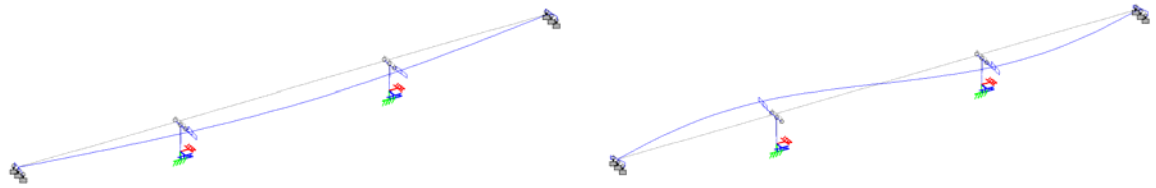


Figure 2.3-2 1st and 2nd Mode for Transversal Direction

In particular, the effective mass from 3rd mode, suddenly/ drastically decreased from the 1st and 2nd mode, the periods of which are much shorter than 0.4s caused by decrement of effective mass. Besides, the ratio of effective mass from 4th mode, there are no modes beyond 10%; hence, the contributing rate of the modes except 1st and 2nd is clearly low for response values. Therefore, combination of seven modes may compute enough accurate response value based on mode combination using CQC method.

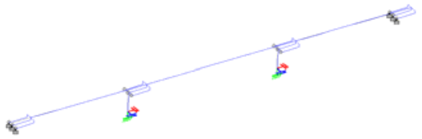
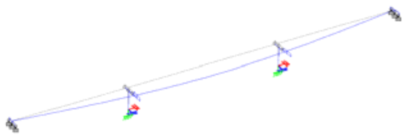
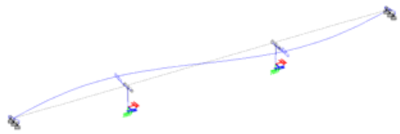
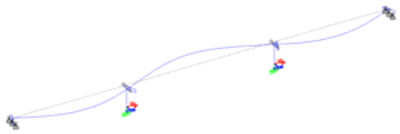
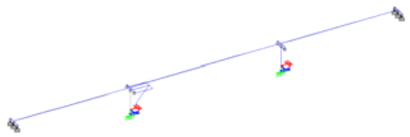
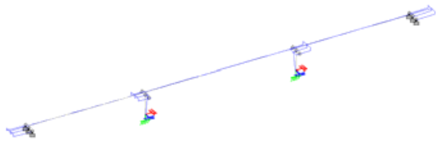
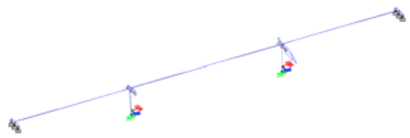
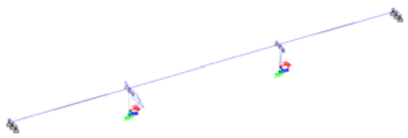
Mode No.	Mode shape		
1		Max Dir.	X
		Frequency	0.763 Hz
		Period	1.310 sec.
		Mode damp.	0.031
2		Max Dir.	Z
		Frequency	0.885 Hz
		Period	1.130 sec.
		Mode damp.	0.021
3		Max Dir.	Z
		Frequency	2.619 Hz
		Period	0.382 sec.
		Mode damp.	0.012
4		Max Dir.	Z
		Frequency	5.658 Hz
		Period	0.177 sec.
		Mode damp.	0.010
5		Max Dir.	X
		Frequency	7.325 Hz
		Period	0.137 sec.
		Mode damp.	0.083
6		Max Dir.	X
		Frequency	7.712 Hz
		Period	0.130 sec.
		Mode damp.	0.082
7		Max Dir.	X
		Frequency	8.628 Hz
		Period	0.116 sec.
		Mode damp.	0.010
8		Max Dir.	Z
		Frequency	9.452 Hz
		Period	0.106 sec.
		Mode damp.	0.093
9		Max Dir.	Z
		Frequency	9.584 Hz
		Period	0.104 sec.
		Mode damp.	0.093
10		Max Dir.	Z
		Frequency	10.059 Hz
		Period	0.099 sec.
		Mode damp.	0.011

Figure 2.3-3 Results of Eigenvalue Analysis

2.4 Damping Constant

Structural damping usually strongly affects the results of dynamic analysis; appropriate examined damping coefficient must be incorporated into the model regardless linear, non-linear, modal analysis or time history response analysis.
For superstructures of general bridge types, viscous damping material internal damping, friction damping at bearing supports and aero dynamical damping can be considered. Also, for piers, material internal damping and friction damping as well as fugacity damping and friction damping between ground and footing can be considered.

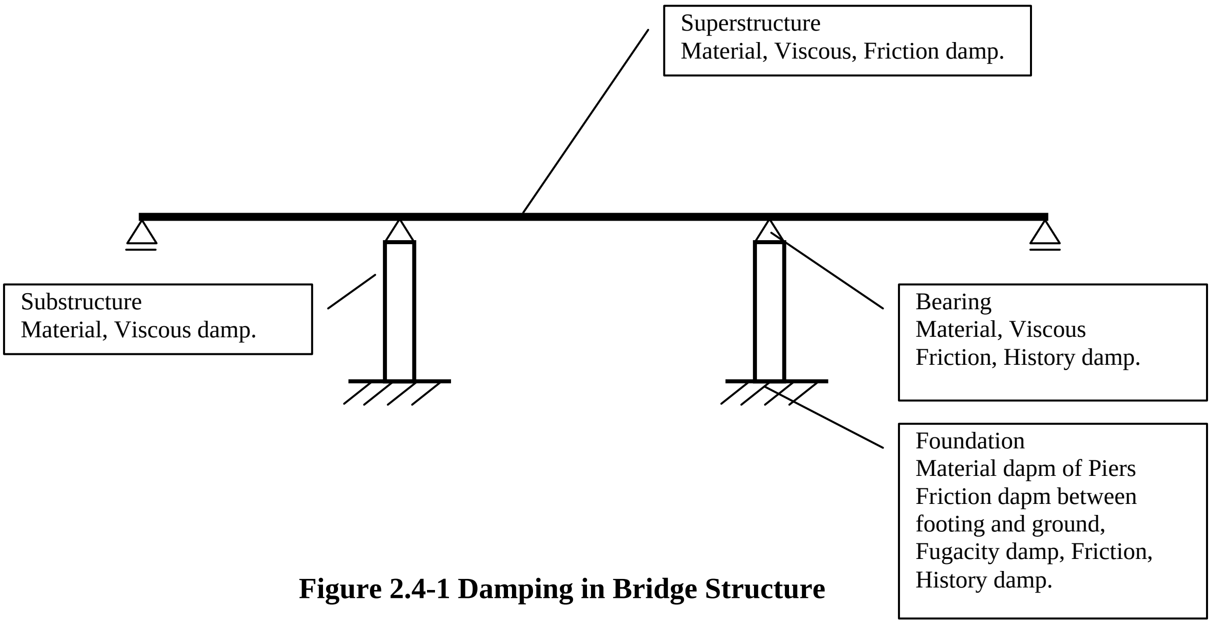


Figure 2.4-1 Damping in Bridge Structure

However, the specific mechanism of each damping factors are absolutely complicated, for execution of dynamic analysis, such the specific mechanism is not necessary to be understood. Generally damping forces are treated as equivalent damping forces in proportional to mass and strain energy.
Generally, because equivalent damping factor of each structural member can not directly be incorporated into dynamic equation, for response spectrum analysis, damping forces should be transformed to mode damping factors in order to be considered in the analysis.

<Dynamic Equation>

$$M \times \ddot{U} + \overset{\text{Damping}}{\boxed{C \times \dot{U}}} + K \times U = M \times \ddot{Z} \qquad \text{Eq.}$$

Where, generally for girder type bridge, strain energy proportional method, shown in the following equation, are utilized because this method can be incorporated into the dynamic in proportional to the amount of strain of the members and structural springs that do not have any mass.

<Mode damping hi: Strain Energy Proportional Method>

$$h_i = \frac{\sum_{j=1}^{n} c_j x_i^t k_j x_i}{x_i^t K x_i} \qquad \text{Eq.}$$

c_j : Structure damping factor of each element, x_i : Mode at i, k_j : Stiffness matrix of each element,
K : Stiffness matrix of all structure

For the bridges in this project, as the Cj in the above equation, following values are adopted.

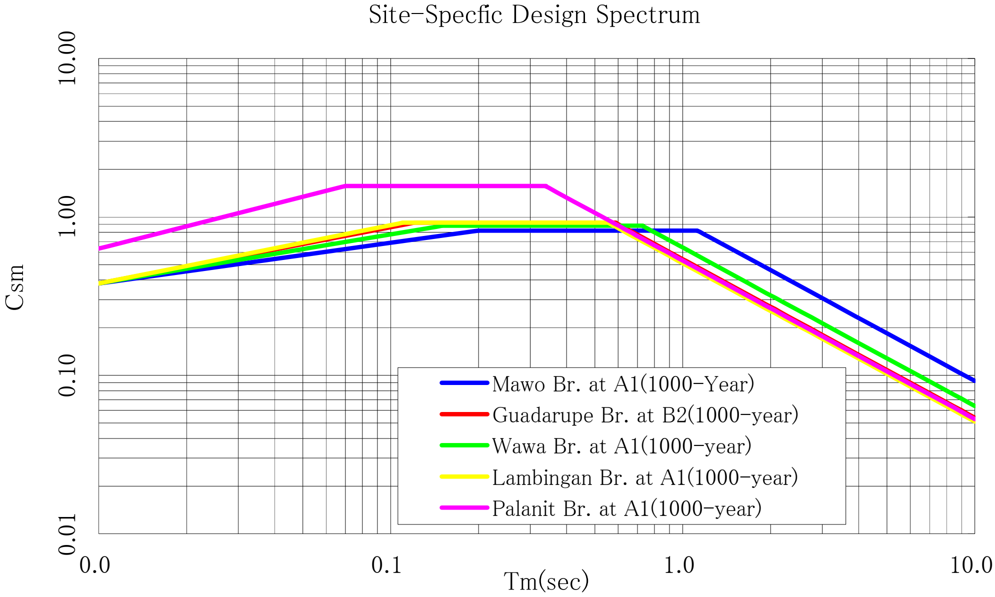
Table 2.4-1 Damping Constant

Structure	Damping ratio
Girder	2%
Bearing (elastic type)	3%
Pier	2%
Foundation	10%

2.5 Design Spectrum

The design spectrum utilized for modal and response spectrum analysis shall be as following figure and table, specified for each bridge to be replaced.

5% Damped



Soil Type & Response Coefficement

Soil Profile Type									
Guadarupe Br. at B2		Mawo Br. at A1 at A1		Wawa Br. at A1 at A1		Lambingan Br. at A1 & B1		Palanit Br. at A1	
T(sec)	Cs(g)	T(sec)	Cs(g)	T(sec)	Cs(g)	T(sec)	Cs(g)	T(sec)	Cs(g)
0.010	0.380	0.010	0.380	0.010	0.380	0.010	0.380	0.010	0.630
0.120	0.920	0.200	0.820	0.150	0.880	0.110	0.920	0.070	1.570
0.120	0.920	0.200	0.820	0.150	0.880	0.110	0.920	0.070	1.570
0.590	0.920	1.120	0.820	0.730	0.880	0.560	0.920	0.340	1.570
0.590	0.915	1.120	0.820	0.730	0.877	0.560	0.911	0.340	1.559
0.610	0.885	1.200	0.767	0.750	0.853	0.600	0.850	0.600	0.883
0.700	0.771	3.000	0.307	0.800	0.800	0.700	0.729	0.700	0.757
0.810	0.667	4.000	0.230	0.850	0.753	0.800	0.638	0.800	0.663
0.900	0.600	5.000	0.184	0.900	0.711	0.900	0.567	0.900	0.589
1.000	0.540	6.000	0.153	1.000	0.640	1.000	0.510	1.000	0.530
2.000	0.270	7.000	0.131	2.000	0.320	2.000	0.255	2.000	0.265
3.000	0.180	8.000	0.115	3.000	0.213	3.000	0.170	3.000	0.177
4.000	0.135	9.000	0.102	4.000	0.160	4.000	0.128	4.000	0.133
5.000	0.108	10.000	0.092	5.000	0.128	5.000	0.102	5.000	0.106
6.000	0.090	0.000	0.000	6.000	0.107	6.000	0.085	6.000	0.088
7.000	0.077	0.000	0.000	7.000	0.091	7.000	0.073	7.000	0.076
8.000	0.068	0.000	0.000	8.000	0.080	8.000	0.064	8.000	0.066
9.000	0.060	0.000	0.000	9.000	0.071	9.000	0.057	9.000	0.059
10.000	0.054	0.000	0.000	10.000	0.064	10.000	0.051	10.000	0.053

Figure 2.5-1 Design Spectrum for New Bridge Design

2.6 Results of Response Spectrum Analysis

From the Following table, seismic response values computed by response spectrum values are shown. For superposition of maximum responses of multiple-mass system using response spectrum of each mode, SRSS, Square Rood of Sum of Square, and CQC method, Complete Quadratic Combination, are worldwidely utilized for the mode combination method of applied modes in the response spectrum analysis. In this outline design, CQC method is applied because accuracy of the combination is better than any other method for adjacent values such as the modes of 5th, 6th and 7th, the eigenvalues of which are distributed closely.

(1) Response values for Longitudinal Direction

i) Displacements

Table 2.6-1 Displacements of Girder

Picked up the end nodes of girder.

Node	DX(mm)	DY(mm)	DZ(mm)	RX(rad)	RY(rad)	RZ(rad)
101	362.6	0.0	0.0	0.0	0.0	0.0
150	362.6	0.0	0.0	0.0	0.0	0.0

Table 2.6-2 Displacement of P1

Node	DX(mm)	DY(mm)	DZ(mm)	RX(rad)	RY(rad)	RZ(rad)
6001	27.4	0.0	0.0	0.0	0.0	0.0
6002	20.5	0.0	0.0	0.0	0.0	0.0
6003	14.2	0.0	0.0	0.0	0.0	0.0
6004	9.0	0.0	0.0	0.0	0.0	0.0
6005	5.2	0.0	0.0	0.0	0.0	0.0
6006	3.8	0.0	0.0	0.0	0.0	0.0
6007	2.4	0.0	0.0	0.0	0.0	0.0

Table 2.6-3 Displacement of P2

Node	DX(mm)	DY(mm)	DZ(mm)	RX(rad)	RY(rad)	RZ(rad)
7001	29.8	0.0	0.0	0.0	0.0	0.0
7002	22.6	0.0	0.0	0.0	0.0	0.0
7003	15.8	0.0	0.0	0.0	0.0	0.0
7004	10.1	0.0	0.0	0.0	0.0	0.0
7005	5.9	0.0	0.0	0.0	0.0	0.0
7006	4.3	0.0	0.0	0.0	0.0	0.0
7007	2.6	0.0	0.0	0.0	0.0	0.0

ii) Sectional force

Table 2.6-4 Sectional Forces of P1

No.	Node	Fx(kN)	Fy(kN)	Fz(kN)	Mx(kNm)	My(kNm)	Mz(kNm)
6001	6001	439.9	17187.1	0.0	0.0	0.0	5048.9
	6002	-439.9	-17187.1	0.0	0.0	0.0	-39415.5
6002	6002	439.9	17407.2	0.0	0.0	0.0	39415.5
	6003	-439.9	-17407.2	0.0	0.0	0.0	-74144.8
6003	6003	439.9	17644.0	0.0	0.0	0.0	74144.8
	6004	-439.9	-17644.0	0.0	0.0	0.0	-109278.1
6004	6004	439.9	17841.0	0.0	0.0	0.0	109278.1
	6005	-439.9	-17841.0	0.0	0.0	0.0	-144762.1
6005	6005	439.9	17913.9	0.0	0.0	0.0	144762.1
	6006	-439.9	-17913.9	0.0	0.0	0.0	-162582.8
6006	6006	439.9	18588.1	0.0	0.0	0.0	162582.8
	6007	-439.9	-18588.1	0.0	0.0	0.0	-180738.5

Table 2.6-5 Sectional Forces of P2

No.	Node	Fx(kN)	Fy(kN)	Fz(kN)	Mx(kNm)	My(kNm)	Mz(kNm)
7001	7001	437.8	17064.9	0.0	0.0	0.0	5011.8
	7002	-437.8	-17064.9	0.0	0.0	0.0	-39134.0
7002	7002	437.8	17292.2	0.0	0.0	0.0	39134.0
	7003	-437.8	-17292.2	0.0	0.0	0.0	-73632.4
7003	7003	437.8	17537.2	0.0	0.0	0.0	73632.4
	7004	-437.8	-17537.2	0.0	0.0	0.0	-108549.4
7004	7004	437.8	17742.2	0.0	0.0	0.0	108549.4
	7005	-437.8	-17742.2	0.0	0.0	0.0	-143831.3
7005	7005	437.8	17817.5	0.0	0.0	0.0	143831.3
	7006	-437.8	-17817.5	0.0	0.0	0.0	-161553.6
7006	7006	437.8	18503.1	0.0	0.0	0.0	161553.6
	7007	-437.8	-18503.1	0.0	0.0	0.0	-179618.8

iii) Supporting Reaction

Table 2.6-6 Supporting Reaction of A1

No.	Fx(kN)	Fy(kN)	Fz(kN)	Mx(kNm)	My(kNm)	Mz(kNm)
8101	6159	205	9	1	0	1817
8301	6159	253	0	0	0	1817
8501	6159	205	9	1	0	1817

Table 2.6-7 Supporting Reaction of A2

No.	Fx(kN)	Fy(kN)	Fz(kN)	Mx(kNm)	My(kNm)	Mz(kNm)
8150	6159	204	9	1	0	1817
8350	6159	253	0	0	0	1817
8550	6159	204	9	1	0	1817

Table 2.6-8 Supporting Reaction of P1 and P2

	Fx(kN)	Fy(kN)	Fz(kN)	Mx(kNm)	My(kNm)	Mz(kNm)
P1	18588	440	0	0	0	182559
P2	18503	438	0	0	0	181430

(2) Response values for Transversal Direction

i) Displacements

Table 2.6-9 Displacement of P1

Node	DX(mm)	DY(mm)	DZ(mm)	RX(rad)	RY(rad)	RZ(rad)
6001	0.0	0.0	12.8	12.8	0.0	0.0
6002	0.0	0.0	10.6	10.6	0.0	0.0
6003	0.0	0.0	8.5	8.5	0.0	0.0
6004	0.0	0.0	6.4	6.4	0.0	0.0
6005	0.0	0.0	4.4	4.4	0.0	0.0
6006	0.0	0.0	3.4	3.4	0.0	0.0
6007	0.0	0.0	2.5	2.5	0.0	0.0

Table 2.6-10 Displacement of P2

Node	DX(mm)	DY(mm)	DZ(mm)	RX(rad)	RY(rad)	RZ(rad)
7001	0.0	0.0	12.6	12.6	0.0	0.0
7002	0.0	0.0	10.5	10.5	0.0	0.0
7003	0.0	0.0	8.4	8.4	0.0	0.0
7004	0.0	0.0	6.3	6.3	0.0	0.0
7005	0.0	0.0	4.4	4.4	0.0	0.0
7006	0.0	0.0	3.4	3.4	0.0	0.0
7007	0.0	0.0	2.5	2.5	0.0	0.0

ii) Sectional force

Table 2.6-11 Sectional Forces of P1

No.	Node	Fx(kN)	Fy(kN)	Fz(kN)	Mx(kNm)	My(kNm)	Mz(kNm)
6001	6001	0.0	0.0	17322.6	3262.2	21179.0	0.0
	6002	0.0	0.0	-17322.6	-3262.2	-55734.3	0.0
6002	6002	0.0	0.0	17478.2	3262.2	55734.3	0.0
	6003	0.0	0.0	-17478.2	-3262.2	-90536.7	0.0
6003	6003	0.0	0.0	17681.6	3262.2	90536.7	0.0
	6004	0.0	0.0	-17681.6	-3262.2	-125665.3	0.0
6004	6004	0.0	0.0	17895.2	3262.2	125665.3	0.0
	6005	0.0	0.0	-17895.2	-3262.2	-161151.0	0.0
6005	6005	0.0	0.0	17990.2	3262.2	161151.0	0.0
	6006	0.0	0.0	-17990.2	-3262.2	-178993.6	0.0
6006	6006	0.0	0.0	18855.7	3262.2	178993.6	0.0
	6007	0.0	0.0	-18855.7	-3262.2	-197272.3	0.0

Table 2.6-12 Sectional Forces of P1

No.	Node	Fx(kN)	Fy(kN)	Fz(kN)	Mx(kNm)	My(kNm)	Mz(kNm)
7001	7001	0.0	0.0	17332.8	3262.4	21669.5	0.0
	7002	0.0	0.0	-17332.8	-3262.4	-56251.8	0.0
7002	7002	0.0	0.0	17487.0	3262.4	56251.8	0.0
	7003	0.0	0.0	-17487.0	-3262.4	-91075.3	0.0
7003	7003	0.0	0.0	17689.7	3262.4	91075.3	0.0
	7004	0.0	0.0	-17689.7	-3262.4	-126222.1	0.0
7004	7004	0.0	0.0	17903.3	3262.4	126222.1	0.0
	7005	0.0	0.0	-17903.3	-3262.4	-161724.4	0.0
7005	7005	0.0	0.0	17998.8	3262.4	161724.4	0.0
	7006	0.0	0.0	-17998.8	-3262.4	-179575.4	0.0
7006	7006	0.0	0.0	18872.3	3262.4	179575.4	0.0
	7007	0.0	0.0	-18872.3	-3262.4	-197865.9	0.0

iii) Supporting Reaction

Table 2.6-13 Supporting Reaction of A1

No.	Fx(kN)	Fy(kN)	Fz(kN)	Mx(kNm)	My(kNm)	Mz(kNm)
8101	561	3845	6524	1010	0	166
8301	0	0	6608	1023	0	0
8501	561	3845	6524	1010	0	166

Table 2.6-14 Supporting Reaction of A2

No.	Fx(kN)	Fy(kN)	Fz(kN)	Mx(kNm)	My(kNm)	Mz(kNm)
8150	561	3825	6522	1010	0	166
8350	0	0	6607	1023	0	0
8550	561	3825	6522	1010	0	166

Table 2.6-15 Supporting Reaction of P1 and P2

	Fx(kN)	Fy(kN)	Fz(kN)	Mx(kNm)	My(kNm)	Mz(kNm)
P1	0	0	18856	199106	0	0
P2	0	0	18872	199701	0	0

3. Design Example of Substructure and Foundation

3.1 Design Concept

3.1.1 General

Based on that the following dimensions are obtained as the outline design for substructure & foundation of Mawo bridge.
The design of foundation is conducted base on two models for the seismic analysis model & foundation design model. When determining the natural periods of the bridge structure, the foundation may be modeled using a series of vertical, horizontal shear and rotational springs. In this example, the seismic analysis is modeled lumped springs at the bottom of pile cap as Figure 3.1.1-2 (b), and the pile foundation design is modeled discrete element modeled & spring at nodal points as Figure 3.1.1-2 (c).

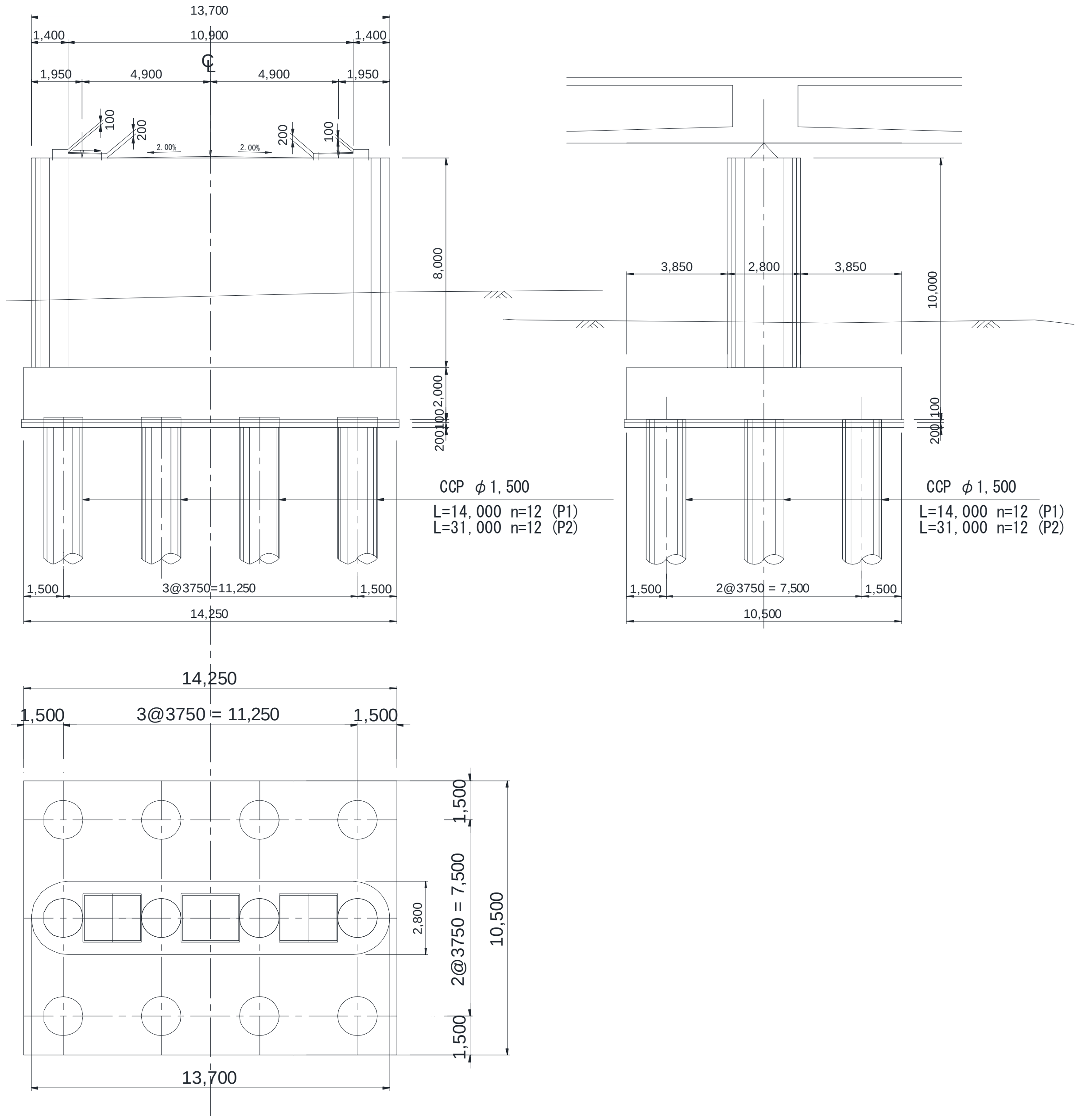
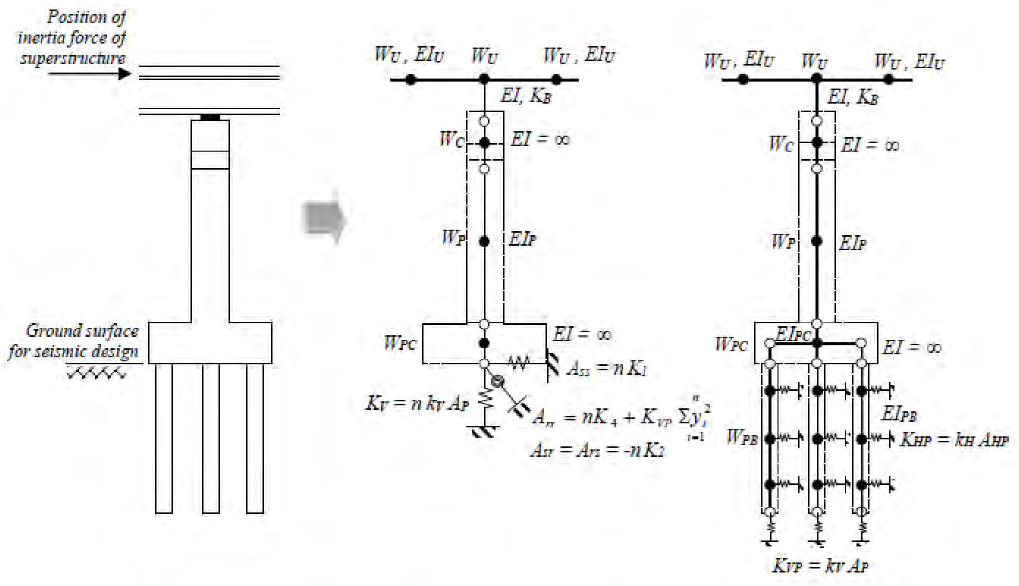


Figure 3.1.1-1 Dimension for Substructure & foundation of Mawo bridge.



(a) Bridge Pier

(b) Footing Model using Lumped Springs at the End Support Node

(c) Footing Model Using Discrete Elements and Spring at Nodal points

Figure 3.1.1-2 Natural Period Calculation Model and Foundation Design Model for Pile Foundation.

3.1.2 Design Specifications

Basically, the design example proceeds in accordance with "LRFD Bridge Seismic Design Specifications" (BSDS). In the "LRFD Bridge Seismic Design Specifications" was consisted in the following table based on AASHTO LRFD 2012, JRA 2012 and DPWT.

Table 3.1.2-1 Design Specifications for Substructure & Foundation Design

Items		BSDS	DPWH	LRFD	JRA
1. Design Conditions & Analysis					
1)	**Materials Characteristics**		○		
	Loads (Live load)		○		
	Load factor			○	
	Seismic Force	○			
	Liquefaction	○			(○)
	Load combination			○	
	Seismic Analysis (except foundation spring)			○	
2. Foundation Design					
	Foundation Spring (for Analysis & Design)	○			(○)
	Foundation Design Force			○	
	Stability of Pile Foundation	○			(○)
	Stability of Spread Sheet Foundation	○		(○)	
3. Structural Section Design (column & pile)					
	Section check items			○	
	Section Strength			○	
	Bar-Arrangement			○	

3.1.3 General Outline of Design Flows

The procedure general outline flow chart of Substructure & Foundation design example Mawo Bridge is shown in the following figure based on newly specified BSDS. In this design example, only the seismic design is mentioned throughout the substructure & foundation design.

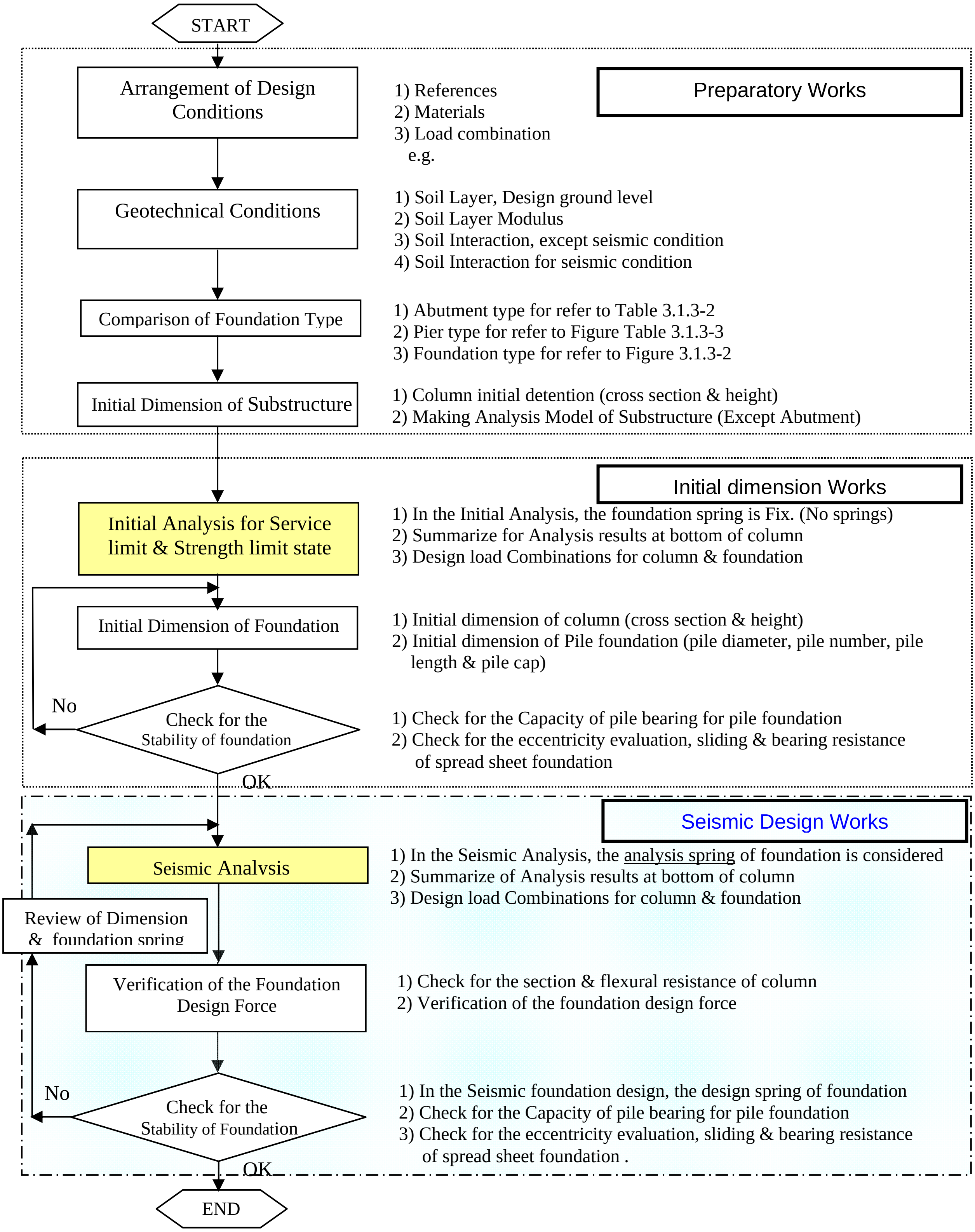


Figure 3.1.3-1 Outline Design Flow for Pile Foundation of Pier

Table 3.1.3-2 Relationship between Abutment Type & Height

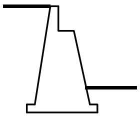
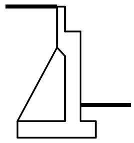
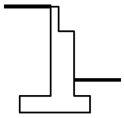
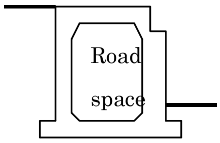
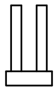
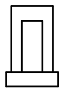
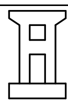
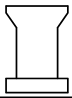
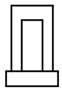
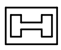
Type of Abutment	Applicable height; H	Type of Abutment	Applicable height; H
a. Gravity Type 	H≦ 3.0～6.0m	c. Buttress Type 	H＞ 12. 0m
b. Cantilever Type 	H≦ 5.0～12.0m	d. Box Type 	H＞ 12. 0m

Table 3.1.3-3 Relationship between Abutment Type & Height

Pier type	Height(m) 10 20 over30	Remarks
Double column type	15	
Frame type(1 layer)	5 15	
Frame type(2 layer)	15 25	
Wall or Column type		Including Tubular pier 
I type	25	  Side view plan

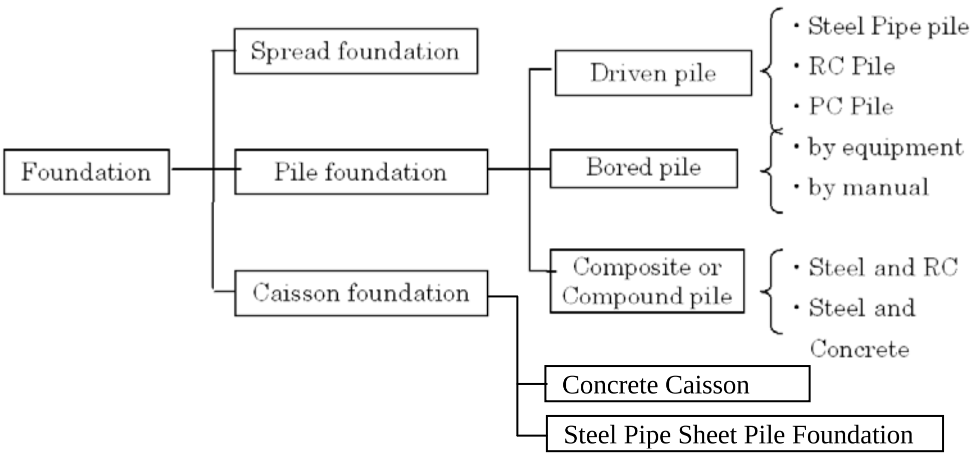


Figure 3.1.3-4 Design Flow for Pile Foundation

3.1.4 Outline Design Items of Foundation Seismic Design

The principal design items of substructure & foundation outline design example Mawo Bridge is shown in the following table based on newly specified BSDS. In this outline design example, only the seismic design is mentioned throughout the substructure & foundation design; the design items of limit state design and serviceability design is not included.

Table 3.1.4-1 Outline Design Items of Foundation Seismic Design and design specifications

Items		Specification	Design Example
1. General Requirement			
	Ground surface in seismic design	BSDS A.3.5.2	3.3.1. (2)
	Response modification factors (R)	BSDS A.3.8.1	A.3.3.1. (2)
			A.3.3.1. (4)
2. Analysis Requirement			
	Coefficients of subgrade reaction for foundation design(k)	BSDS A.4.4.2	A.3.3.1.(4)
	Coefficients of subgrade reaction for seismic analysis(ko)	BSDS A.4.4.3	A.3.2.2.(1)
3. Design Requirement			
	Single column and pier	BSDS A.5.3.4.3.b	A.3.3.2.
	Foundation design forces	BSDS A.5.3.4.3.f	A.3.3.3(2)
	Pile Foundation	BSDS A.5.4.3.	A.3.3.3.
	Pile Arrangement	BSDS A.5.4.3.2	A.3.3.3.(1)
	Nominal Axial Compression Resistance of a Single Pile (Bearing Capacity)	BSDS A.5.4.3.3	A.3.3.3.(3).3)
	Section Design	LRFD Section5	A3.3.2(4),(5) A3.3.3.(4) 2),3)
	Re-bar Arrangement	LRFD Section5	A3.3.2(3) A3.3.3.(4) 1)

3.1.5 Design Software

The outline design of Substructure and Foundation adopt the following design software.

Table 3.1.5-1 Design Software for Substructure & Foundation Design

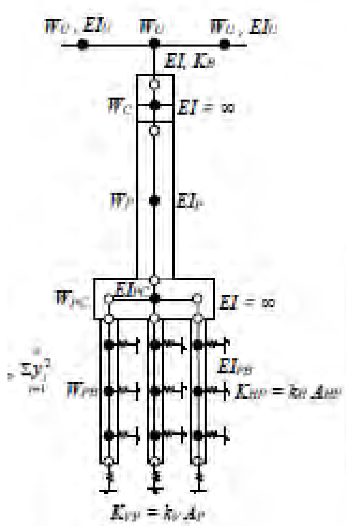
Design Items	Name of Software (Developed)
Bridge Structural Analysis	STAAD Pro. (Bentley)
Pile Foundation Design (including Analysis spring) Biaxial Interaction Curve for Column & Pile Pile reaction, Section force, Displacement and analysis spring for Pile Foundation	FB-Multi Pier (Bridge Software Institute)

Note) BSI ; Bridge Software Institute(http://bsi.ce.ufl.edu/)

3.2 Analysis

3.2.1 General

The ground condition & pile foundation at P2 is shown in the following figure. In this outline design example, only the P2 is shown throughout the coefficient of subgrade reaction & analysis results. And the foundation spring constants for seismic, the FB-Multi pier is conducted by JRA spring.



where;

kh = Coefficient of subgrade reaction in the horizontal direction at the pile section corresponding to area A_{HP} derived

kv = Coefficient of subgrade reaction in the vertical direction at the bottom of foundation derived

● : Nodal point to model structure weight	W_C : Weight of coping beam at nodes
○ : Nodal point for section property change	W_P : Weight of pier body at nodes
EI_U : Superstructure flexural rigidity	W_{PC} : Weight pile cap at nodes
EI : Substructure flexural rigidity	W_U : Weight of superstructure at nodes
EI_{PC} : Pile cap flexural rigidity	W_{PB} : Weight of pile body at nodes
EI_{PB} : Pile body flexural rigidity	K_B : Spring constant of elastic bearing

Figure 3.2.1-1 Natural Period Calculation Model for Pile Foundation

- A2 Side - Mawo MAW-L1 (right ban			Param	Layer Thickness (m)
Depth Upper	SPT Blows	Soil Classification Observation	Nd	
0.55	2	Clay	3	5
1.55	3	Clay		
2.55	4	Silty clay		
3.55	3	Claye silt		
4.55	4	Silty v.f. sand		
5.55	8	Silty fine sand	10	2
6.55	12	Fine sand with silt		
7.55	24	Gravel w/ silt	21	8
8.55	23	Gravel w/ silt		
9.55	21	Gravel		
10.55	21	Sandy gravel		
11.55	24	Gravel with sand		
12.55	17	Gravel with silt		
13.55	22	Gravel with silt		
14.55	22	Corse sand with gravel		
15.55	7	Silty clay	8	13
16.55	7	Sandy silt		
17.55	7	Clayey silt		
18.55	12	Sandy clay		
19.55	7	Clay with sand		
20.55	7	Clay with sand		
21.55	9	Clayey sand		
22.55	9	Clay with sand		
23.55	10	Clay with sand		
24.55	8	Sandy silt		
25.55	8	Clay with sand		
26.55	11	Clay with sand		
27.55	11	Sandy clay		
28.55	17	Sand with clay	17	3
29.55	24	Fine to med. sand		
30.55	10			
31.55	22	Fine to med. sand	31	7
32.55	43	Fine to med. sand		
33.55	31	Sand with gravel		
34.55	31	Gravelly sand		
35.55	23	Silty grravel		
36.55	23	Silty sand with gravel		
37.55	50	Med. sand with gravel		
38.55	50	Rock	300	6
39.55	50	Rock		
40.55	50	Rock		
41.55	50	Rock		
42.55	50	Rock		
43.55	50	Rock		

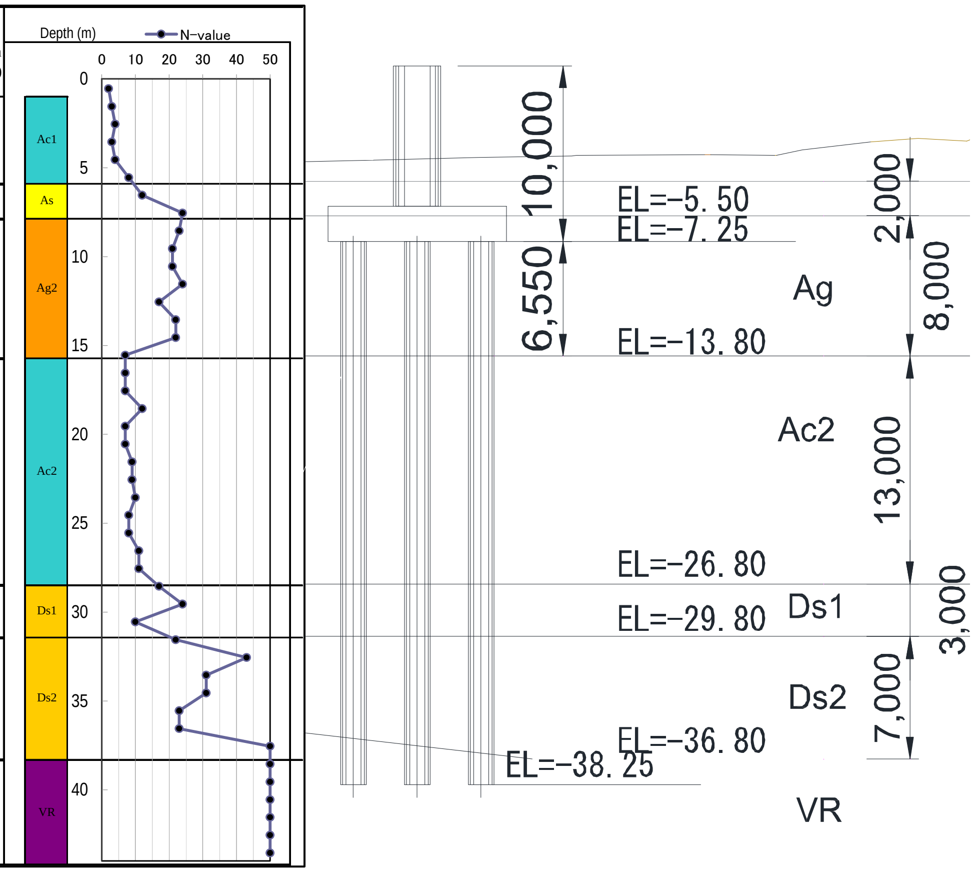


Figure 3.2.1-2 Soil condition & Pile foundation at P2

3.2.2 Foundation Spring for Seismic Analysis

(1) Coefficients of Subgrade Reaction

When calculating the natural periods, the deformation effects of the structural members and the foundations shall be considered. The coefficients of subgrade reaction, for verification of seismic performance and calculation of natural period shall be obtained based on the stiffness of the ground which is equivalent to the deformation of the ground during an earthquake.
In this outline design, the analysis model of soil springs would be suggested using the p-y curve method based on the coefficient of subgrade reaction. The values of the coefficient of subgrade reaction shall be obtained as followings.

Table 3.2.2-1 Coefficients of Subgrade Reaction for horizontal (kh)

D	(m)	1.500	
Ac	(m²)	1.76715	
Ic	(m⁴)	0.24850	
Es	(kN/m²)	2.00E+08	
Ec	(kN/m²)	2.50E+07	
Es/Ec	(-)	8.000	
A	(m²)	1.7671	
I	(m⁴)	0.2485	
1/β(①)	(m)	3.3848	estimate value
$aE_{0(ave)}$	(kN/m²)	171,788	
B_H	(m)	2.2533	
k_{H0}	(kN/m³)	572,625	
k_H	(kN/m³)	126,212	
b	(1/m)	0.2954	
1/β(②)	(m)	3.3848	0.0000

(The check of convergence calculation ①−②)

where;

$$k_H = k_{H0}\left(\frac{B_H}{0.3}\right)^{-3/4} \quad (\text{kN/m}^3)$$

$$k_{H0} = \frac{1}{0.3}\alpha E_0 \quad (\text{kN/m}^3)$$

$$B_H = \sqrt{D/\beta} \quad (\text{m})$$

$$\beta = \sqrt[4]{\frac{k_H D}{4EI}} \quad (\text{m}^{-1})$$

$$k_{HE} = \eta_k \alpha_k k_H$$

No.	Strata name	Thickness (m)	Depth (m)	EL.(m) (Top of Layer)	N-values	Vsi (m/s)	VsD (m/s)	νd	γt (kN/m3)	E_D (kN/m² (kN/m2))	k_H (kN/m3)
1	Ag2 Gravel	6.55	6.55	-7.25	21.0	220.71	176.57	0.5	18.0	171,788	126,212
2	Ac2 Clay	13.00	19.55	-13.80	8.0	200.00	160.00	0.5	18.0	141,061	103,638
3	Ds1 Sand	3.00	22.55	-26.80	17.0	205.70	164.56	0.5	17.0	140,926	103,539
4	Ds2 Sand	7.00	29.55	-29.80	31.0	251.31	201.05	0.5	19.0	235,098	172,726
5	VR Rock	1.45	31.00	-36.80	50.0	294.72	235.78	0.5	21.0	357,366	262,557
			Tip	-38.25							

※ED=2(1+νd)γ_t/g・V_{si}^2

VsD={Vsi<300, 0.8xVsi, Vsi>=300, 1.0xVsi)

Table 3.2.2-2 Seismic Analysis Springs at P2 Foundation

P2 Pile Length in the grou L : 31.000 (m)
Diameter D : 1.500 (m)

EL= -7.25

	Soil Layers	Thickness	Depth	N-Vaiues	Angle of shear resistance	Chesion	Unit weight (underwater)	k_H (kN/m^3)	k_{HE} (kN/m³) [=a x k_H]	K_{HE} (kN/m/m) [=D x k_{HE}]	P −y Curve P(kN/m)	Y(m) [D x 1%]
	—	(m)	(m)	—	φ(°)	c (kN/m²)	g(kN/m³)					
① Ag2 Grav	As	6.55	6.55	21	36.0	0.0	9.0	126,212	126,212	189,318	2,840	0.015
② Ac2 Clay	Ac	13.00	19.55	8	0.0	50.0	9.0	103,638	103,638	155,456	2,332	0.015
③ Ds1 Sand	As	3.00	22.55	17	32.0	0.0	8.0	103,539	103,539	155,308	2,330	0.015
④ Ds2 Sand	As	7.00	29.55	31	34.0	0.0	10.0	172,726	172,726	259,089	3,886	0.015
⑤ VR Rock	As	1.45	31.00	50	38.0	170.0	12.0	262,557	262,557	393,835	5,908	0.015

L

※1) Sandy: As, Cohesive: Ac ※g=Total Unit Weight − 9kN/m³

(2) Foundation Spring Constants for Seismic Analysis

The Pier Foundation spring for seismic analysis is calculated by "FB-Multi Pier" (refer to A 2.2). The results of foundation spring are shown as followings. In this design, abutments are not modeled in the seismic analysis because abutments may have enough strength and stiffness fixed by grounds for seismic vibration.

Table 3.2.2-3 Foundation Springs constants of Seismic Analysis for P1 & P2

Mawo P1

	KX	KY	KZ	TX	TY	TZ
	(kN/m)	(-)	(kN/m)	(kNm/rad)	(-)	(kNm/rad)
FB-Pier	7.669E+06	Fix	7.660E+06	2.017E+08	Fix	1.284E+08

Mawo P2

	KX	KY	KZ	TX	TY	TZ
	(kN/m)	(-)	(kN/m)	(kNm/rad)	(-)	(kNm/rad)
FB-Pier	7.012E+06	Fix	7.658E+06	2.067E+08	Fix	1.094E+08

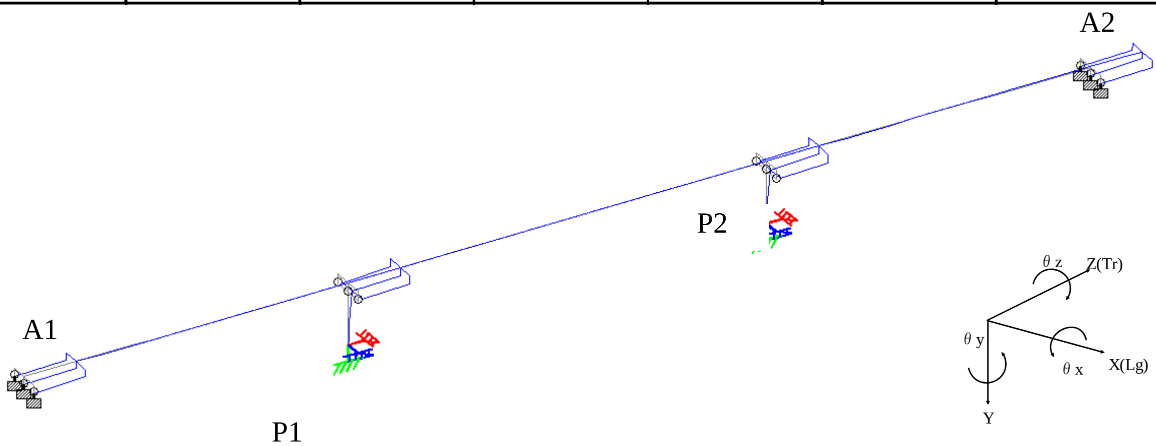


Figure 3.2.2-4 Analytical Mode of Seismic Analysis by Staad Pro.

3.2.3 Results of Seismic Analysis

From the Following table, seismic response values computed by response spectrum values are shown.

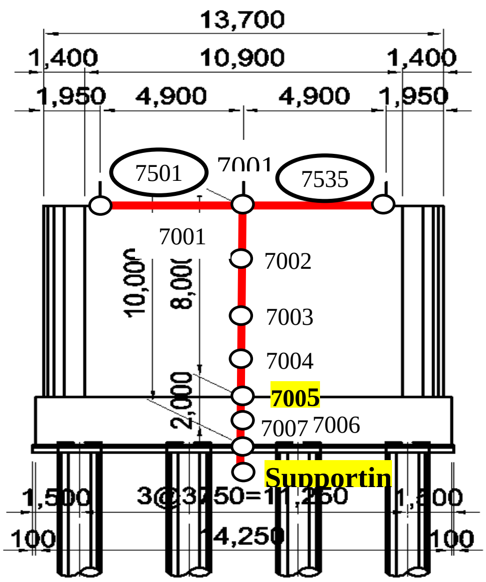


Figure 3.2.3-2 Model of P2 pier

Table 3.2.3-1 Results of Seismic Analysis for Foundation Design

- LONGITUDINAL DIRECTION

<TOP OF SUBSTRUCTURES>

	Longitudinal Direction		
	H(KN)	M(KNm)	V(KN)
A1 (E)	18,477	5,451	663
A2 (E)	18,477	5,451	661

Note:

<BOTTOM OF PIERS>

	H(KN)	M(KNm)	V(KN)
P1 (E)	17,841	144,762	440
P2 (E)	17,742	143,831	438

Note: Node No 6005, 7005

<BOTTOM OF FOOTING>

	H(KN)	M(KNm)	V(KN)
P1 (E)	18,588	182,559	440
P2 (E)	18,503	181,430	438

Note: Node Supporting

- TRANSVERSAL DIRECTION

<BOTTOM OF PIERS>

	H(KN)	M(KNm)	V(KN)
P1 (E)	17,895	161,151	0
P2 (E)	17,903	161,724	0

Note: Node No 6005, 7005

<BOTTOM OF FOOTING>

	H(KN)	M(KNm)	V(KN)
P1 (E)	18,856	199,106	0
P2 (E)	18,872	199,701	0

Note: Node Supporting

3.3 Pier Outline Design

3.3.1 Design Condition

(1) Design Force

From the Following table, the Substructure & Pile foundation design values computed by results of seismic analysis values are shown.

Table 3.3.1-1 Outline Design Items of Pile Foundation Seismic Design

- Results of Eigenvalue Analysis

VERTICAL REACTIONS FOR SUBSTRUCTURE STABLE CALCULATION (KN)

	Dead load	Live load	Sum.
P1	33,900	3,800	37,700
P2	33,900	3,800	37,700

Note: Impact factor exclusive

-Section Forces for Column design (at bottom of Column)

	Longitudinal Direction			Transversal Direction		
	H(KN)	M(KNm)	V(KN)	H(KN)	M(KNm)	V(KN)
P1 (F)	17,841	144,762	440	17,895	161,151	0
P2 (F)	17,742	143,831	438	17,903	161,724	0

-Section Forces for foundation design (at bottom of Pile Cap)

	Longitudinal Direction			Transversal Direction		
	H(KN)	M(KNm)	V(KN)	H(KN)	M(KNm)	V(KN)
P1 (F)	18,588	182,559	440	18,856	199,106	0
P2 (F)	18,503	181,430	438	18,872	199,701	0

- Dead Loas (for Pier column)

	Pier Column					
	h	B	Aria	Height	Unit Weight	Self Weight
	(m)	(m)	(m2)	(m)	(kN/m3)	(kN)
P1	2.8	13.7	36.7	8.0	24.5	7,189
P2	2.8	13.7	36.7	8.0	24.5	7,189

- Combination Loads for Column Design

Longitudinal Direction at Section A-A (bottom of Column)

	DL		LL		EQ			SUM of LONGITUDINAL				
	N (kN)	φ	N (kN)	φ	H (kN)	M (kNm)	φ	N (kN)	Hl (kN)	Ml (kNm)	Ht (kN)	Mt (kNm)
P2(Nmax)	41,089	1.25	3,800	0.50	17,742	143,831	1.00	**53,270**	**17,742**	**143,831**	5,371	48,517
(Nmin)	41,089	0.90	3,800	0.50	17,742	143,831	1.00	**38,890**	**17,742**	**143,831**	5,371	48,517
								Z	Y	Mx	X	My

Transverse Direction at Section A-A (bottom of column)

	DL		LL		EQ			SUM of TRANSVERSAL				
	N (kN)	φ	N (kN)	φ	H (kN)	M (kNm)	φ	N (kN)	Hl (kN)	Ml (kNm)	**Ht (kN)**	**Mt (kNm)**
P2(Nmax)	41,089	1.25	3,800	0.50	17,903	161,724	1.00	**53,270**	5,323	43,149	**17,903**	**161,724**
(Nmin)	41,089	0.90	3,800	0.50	17,903	161,724	1.00	**38,890**	5,323	43,149	**17,903**	**161,724**

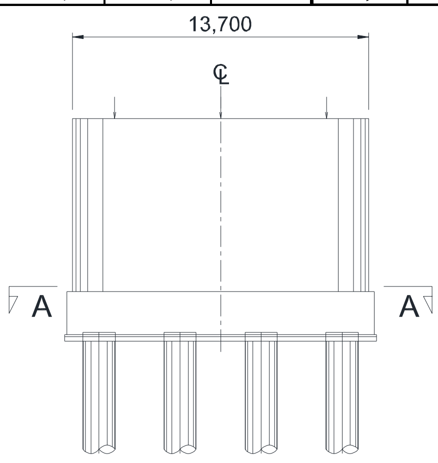


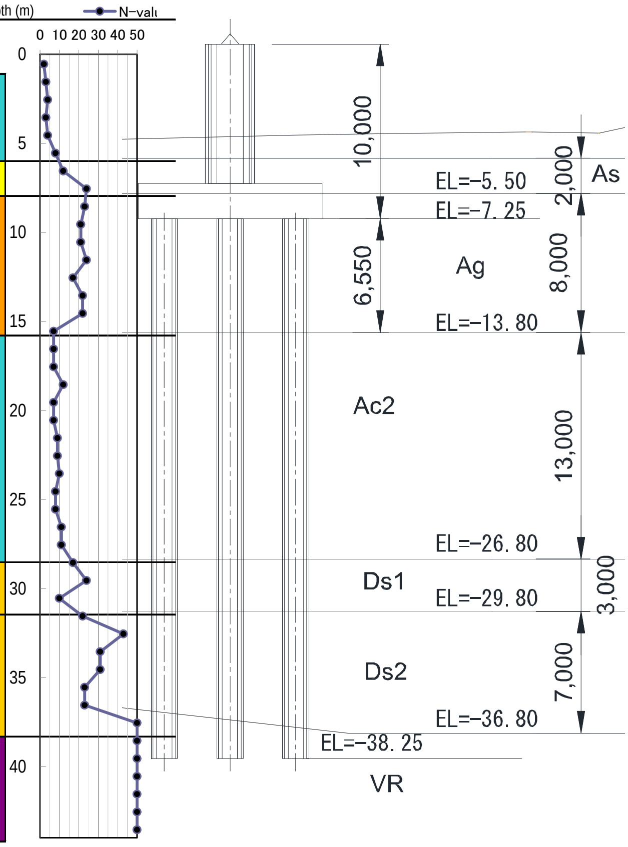
Figure 3.3.1-1 Design Section of Column (Section A-A)

(2) Soil Condition

The results of ground investigation are shown in below illustrations and following table. The weathered rock layer that can be regarded as the bearing layer is distributed E.L. -2.0m to E.L.-6.5m depth, and has a thick surface layer predominant with gravely sand on top. Specialty, liquefiable sand (Dsg) is thickly deposited from ground surface to GL-2.0m, of which N-value is 15 to 29, will be affected by liquefaction occurs with reduction of geotechnical parameter. .

Table 3.3.1-2 Soil Condition & P2 Pile Foundation

- A2 Side - Mawo M			Soil Parameters							Layer Thickness	
Number	Depth Upper	SPT Blows	Nd	γt (tf/m³)	C (kN/m²)	φ (°)	E0 (kN/m²)	Vsn (m/sec)		(m)	
1	0.55	2		14							
2	1.55	3		14							
3	2.55	4	3	14	19	0	2,100	144		5	Ac1
4	3.55	3		14							
5	4.55	4		14							
6	5.55	8	10	17	0	34	7,000	172		2	As
7	6.55	12		17							
8	7.55	24		18							
9	8.55	23		18							
10	9.55	21		18							
11	10.6	21	21	18	0	36	14,700	221		8	Ag2
12	11.6	24		18							
13	12.6	17		18							
14	13.6	22		18							
15	14.6	22		18							
16	15.6	7		18							
17	16.6	7		18							
18	17.6	7		18							
19	18.6	12		18							
20	19.6	7		18							
21	20.6	7		18							
22	21.6	9	8	18	50	0	5,600	200		13	Ac2
23	22.6	9		18							
24	23.6	10		18							
25	24.6	8		18							
26	25.6	8		18							
27	26.6	11		18							
28	27.6	11		18							
29	28.6	17		17							
30	29.6	24	17	17	0	32	11,900	206		3	Ds1
31	30.6	10		17							
32	31.6	22		19							
33	32.6	43		19							
34	33.6	31		19							
35	34.6	31	31	19	0	34	21,700	251		7	Ds2
36	35.6	23		19							
37	36.6	23		19							
38	37.6	50		19							
39	38.6	50		21							
40	39.6	50		21							
41	40.6	50	300	21	170	38	136,098	300		6	VR
42	41.6	50		21							
43	42.6	50		21							
44	43.6	50		21							

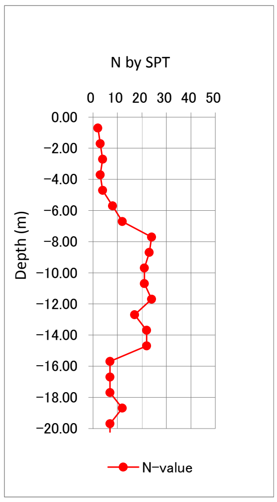


(3) Assessment of Soil Liquefaction

According to the design specifications, sandy layer requiring liquefaction Assessment is obviously obtained as following table.

Table 3.3.1-3 Assessment of Soil Liquefaction

Assessment of Liquefaction Potential								
GL-(m)	Soil Layers	N by SPT	Ground Water Level (-m)	Fc (%)	PI	D50 (mm)	D10 (mm)	Liquefiable
		<30		<35%	<15	<10mm	<1mm	
0.70	Ac1	2	0.50	62.3	16			
1.70	Ac1	3	0.50	56.3	2			
2.70	Ac1	4	0.50	56.3	2			
3.70	Ac1	3	0.50	79.7	8			
4.70	Ac1	4	0.50	2.2		0.30	0.10	
5.70	As	8	0.50	1.5		0.24	0.11	○
6.70	As	12	0.50	1.5		0.63	0.13	○
7.70	Ag	24	0.50	0.2		12.61	0.49	○
8.70	Ag	23	0.50	0.6		2.75	0.21	○
9.70	Ag	21	0.50	0.2		2.57	0.76	○
10.70	Ag	21	0.50	1.1		4.57	0.20	○
11.70	Ag	24	0.50	1.3		3.24	0.23	○
12.70	Ag	17	0.50	3.1		2.85	0.14	○
13.70	Ag	22	0.50	0.2		10.30	0.47	○
14.70	Ag	22	0.50	1.6		0.86	0.16	○
15.70	Ac2	7	0.50	61.8	12.0			
16.70	Ac2	7	0.50	68.0	9.0			
17.70	Ac2	7	0.50	72.1	4.0			
18.70	Ac2	12	0.50	56.6	2.0			
19.70	Ac2	7	0.50	75.7	4.0			
20.70	Ac2	7	0.50	56.6	5.0			
21.70	Ac2	9	0.50	35.6	4.0	0.34		
22.70	Ac2	9	0.50	43.4	4.0	0.09		
23.70	Ac2	10	0.50	60.5	4.0			
24.70	Ac2	8	0.50	35.3		0.14		



Based on the results of liquefaction assessment, reduction of geotechnical parameters shall be conducted in accordance with the following tables. In this the outline design, the liquefaction have no effect on the foundation design due to which is upper layer than the pile cap.

Table 3.3.1-4 Assessment of Soil Liquefaction Parameters

Basic Soil Profile Information										
GL-(m)	Soil Layers	N by SPT	Fc (%)	S=1 G=2 C=3	γt γt1	Water unit weight	Ground Water Level (-m)	σU (Kpa)	σv (Kpa)	σv' (Kpa)
5.70	As	8	1.5	1	18	10.00	0.50	52.00	87.80	35.80
6.70	As	12	1.5	1	18	10.00	0.50	62.00	105.80	43.80
7.70	Ag	24	0.2	1	19	10.00	0.50	72.00	124.80	52.80
8.70	Ag	23	0.6	1	19	10.00	0.50	82.00	143.80	61.80
9.70	Ag	21	0.2	1	19	10.00	0.50	92.00	162.80	70.80
10.70	Ag	21	1.1	1	19	10.00	0.50	102.00	181.80	79.80
11.70	Ag	24	1.3	1	19	10.00	0.50	112.00	200.80	88.80
12.70	Ag	17	3.1	1	19	10.00	0.50	122.00	219.80	97.80
13.70	Ag	22	0.2	1	19	10.00	0.50	132.00	238.80	106.80
14.70	Ag	22	1.6	1	19	10.00	0.50	142.00	257.80	115.80

Table 3.3.1-5 Results on Liquefaction Resistance Factor (FL) & Reduction Factor (D_E)

Calculation for FL								Reduction Factor D_E		
Depth	N1	C1	C2	Na	R	L	FL	**R(Ave.)**	**FL(Ave.)**	**D_E**
-5.70	12.85	1.000	0.000	12.854	0.243	0.852	0.285	**0.265**	**0.316**	**0.00**
-6.70	17.93	1.000	0.000	17.926	0.287	0.826	0.348			
-7.70	33.22	1.000	0.000	33.225	1.348	0.794	1.697	**0.533**	**0.719**	**1.00**
-8.70	29.67	1.000	0.000	29.666	0.750	0.769	0.975			
-9.70	25.36	1.000	0.000	25.355	0.430	0.747	0.576			
-10.70	23.83	1.000	0.000	23.832	0.377	0.727	0.519			
-11.70	25.69	1.000	0.000	25.693	0.445	0.708	0.628			
-12.70	17.22	1.000	0.000	17.223	0.281	0.691	0.407			
-13.70	21.15	1.000	0.000	21.154	0.322	0.675	0.477			
-14.70	20.13	1.000	0.000	20.129	0.309	0.659	0.469			

(4) Coefficients of Subgrade Reaction

When design of the pile foundation, the coefficients of subgrade reaction, for verification of the design of pile foundation performance shall be obtained based on the seismic design coefficients of the ground which is equivalent to the deformation of the ground during an earthquake. In this outline design, the design model of soil springs would be suggested using the p-y curve method based on the coefficient of subgrade reaction as seismic analysis model. The values of the coefficient of subgrade reaction shall be obtained as followings.

Table 3.3.1-6 Coefficients of Subgrade Reaction for horizontal (kh) & Vertical (kv)

D	(m)	1.500		
t	(m)	0.000	casing	
Dc	(m)	1.500		
As	(m^2)	0.00000		
Ac	(m^2)	1.76715		
Is	(m^4)	0.00000		
Ic	(m^4)	0.24850		
Es	(kN/m^2)	2.00E+08		
Ec	(kN/m^2)	2.50E+07		
Es/Ec	(-)	8.000		
A	(m^2)	1.7671		
I	(m^4)	0.2485		
1/β(①)	(m)	4.5497		
$aE_{0(ave)}$	(kN/m^2)	58,800		
B_H	(m)	2.6124		
k_{H0}	(kN/m^3)	196,000		
k_H	(kN/m^3)	38,665		
b	(1/m)	0.2198		
1/β(②)	(m)	4.5497	0.0000	(The check of convergence calculation ①−②)

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$$k_H = k_{H0}\left(\frac{B_H}{0.3}\right)^{-3/4} \quad (\mathrm{kN/m^3})$$

$$k_{H0} = \frac{1}{0.3}\alpha E_0 \quad (\mathrm{kN/m^3})$$

$$B_H = \sqrt{D/\beta} \quad (\mathrm{m})$$

$$\beta = \sqrt[4]{\frac{k_H D}{4EI}} \quad (\mathrm{m^{-1}})$$

$$k_{HE} = \eta_k \alpha_k k_H$$

No.	Strata name	Thickness (m)	Depth (m)	EL.(m) (Top of Layer)	N-values	γt (kN/m3)	Deformation Modulus α	E_0(kN/m^2)	aE_0(kN/m^2)	k_H (kN/m^3) Normal	EQ
1	Ag2 Gravel	6.55	6.55	-7.25	21.0	18.0	1	58,800	58,800	38,665	77,330
2	Ac2 Clay	13.00	19.55	-13.80	8.0	18.0	1	22,400	22,400	14,730	29,459
3	Ds1 Sand	3.00	22.55	-26.80	17.0	17.0	1	47,600	47,600	31,300	62,601
4	Ds2 Sand	7.00	29.55	-29.80	31.0	19.0	1	86,800	86,800	57,077	114,154
5	VR Rock	1.45	31.00	-36.80	300.0	21.0	2	136,098	272,196	178,988	357,976
			Tip	-38.25				*Eo=2800N*	*except for the rock		

Table 3.3.1-7 Coefficient of vertical ground reaction k_V

D	(m)	1.500
A	(m^2)	1.767
E_0	(kN/m^2)	272,196
α	(-)	2
$aE0$	(kN/m^2)	544,392
k_{V0}	(kN/m^3)	1,814,640
k_V	(kN/m^3)	542,704
K_V	(kN/m)	959,037
K_{VE}	(kN/m)	1,918,074
P	(kN)	11,012
y	(m)	0.00574

$$k_V = k_{V0}\left(\frac{B_V}{0.3}\right)^{-3/4} \quad (\mathrm{kN/m^3})$$

$$k_{V0} = \frac{1}{0.3}\alpha E_0 \quad (\mathrm{kN/m^3})$$

$$B_V = \sqrt{A_H} = D \quad (\mathrm{m})$$

$$K_V = k_V A \quad (\mathrm{kN/m})$$

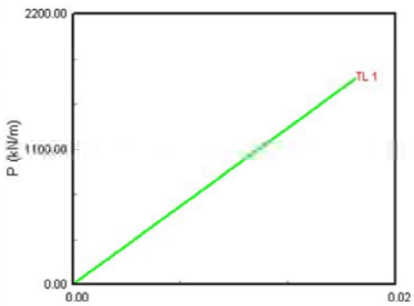
* P : Bearing Resistance

Table 3.3.1-8 Seismic Design Springs at P2 Foundation

P2 Pile Length in the grc L : 31.000 (m)
Diameter D : 1.500 (m)

	Soil Layers	Thickness	Depth	N-Vaiues	Angle of shear resistance	Chesion	Unit weight (underwat	k_H (kN/m^3)	K_H (kN/m/m) [=D x k_{HE}]	$P-y$ Curve for Static P(kN/m)	$P-y$ Curve for Static Y(m) [D x 1%]	$P-y$ Curve for EQ P_{EQ}(kN/m)	$P-y$ Curve for EQ Y_{EQ}(m) [D x 1%]
	—	(m)	(Hx)	—	φ(°)	c (kN/m^2)	γ(kN/m^3)						
① Ag2 Grave	As	6.55	6.55	21	36.0	0.0	9.0	38,665	57,998	870	0.015	1,740	0.015
② Ac2 Clay	Ac	13.00	19.55	8	0.0	50.0	9.0	14,730	22,094	331	0.015	663	0.015
③ Ds1 Sand	As	3.00	22.55	17	32.0	0.0	8.0	31,300	46,951	704	0.015	1,409	0.015
④ Ds2 Sand	As	7.00	29.55	31	34.0	0.0	10.0	57,077	85,616	1,284	0.015	2,568	0.015
⑤ VR Rock	As	1.45	31.00	300	38.0	170.0	12.0	178,988	268,482	4,027	0.015	8,054	0.015

※1）Sandy：As，Cohesive：Ac ※γ=Total Unit Weight － 9kN/m^3



p-y curve of 1st As Layer

3.3.2 Column Design

(1) Column Dimension

Based on that the following dimensions are conducted as the outline design for column of Mawo bridge, which are determined due to the dimension of rubber bearing (for column plane) and design ground surface (for column height).

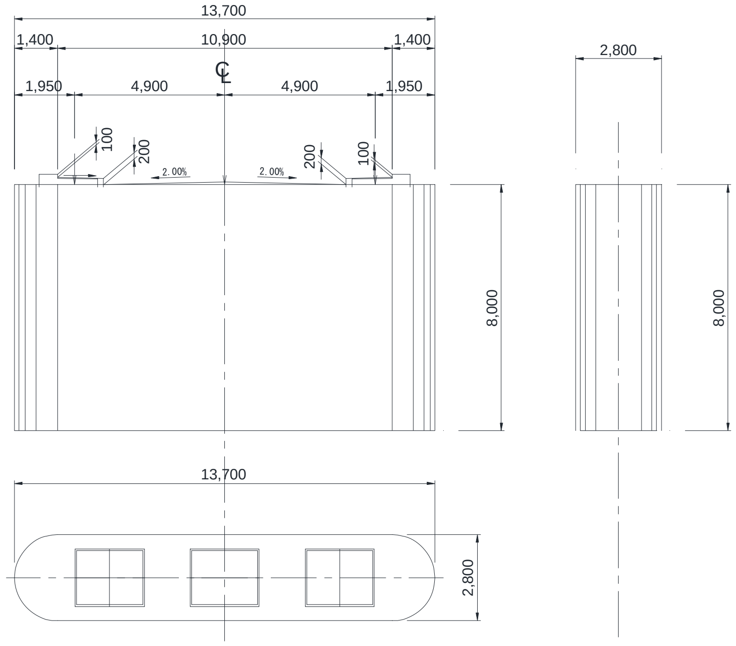


Figure 3.3.2-1 Dimension of P2 column

(2) Design Load

Based on the results of seismic analysis, summarize of design force of column is conducted for the following table, which are considered for load combination and following combination.

-100 percent of the absolute value of the force effects in one of the perpendicular directions combined with 30 percent of the absolute value of the force effects in the second perpendicular direction, and
- 100 percent of the absolute value of the force effects in the second perpendicular direction combined with 30 percent of the absolute value of the force effects in the first perpendicular direction. (BSDS A.5.2 (1))

Table 3.3.2-1 Summarize of Design Forces at Bottom of Column

Longitudinal Direction at Section A-A (bottom of Column)

	DL		LL		EQ			SUM of LONGITUDINAL				
	N (kN)	φ	N (kN)	φ	H (kN)	M (kNm)	φ	N (kN)	Hl (kN)	Ml (kNm)	Ht (kN)	Mt (kNm)
P2(Nmax)	41,089	1.25	3,800	0.50	17,742	143,831	1.00	**53,270**	**17,742**	**143,831**	5,371	48,517
(Nmin)	41,089	0.90	3,800	0.50	17,742	143,831	1.00	**38,890**	**17,742**	**143,831**	5,371	48,517

Transverse Direction at Section A-A (bottom of column)

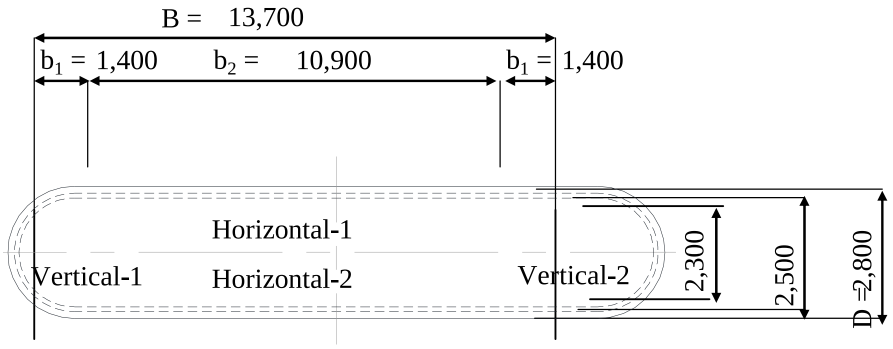
	DL		LL		EQ			SUM of TRANSVERSAL				
	N (kN)	φ	N (kN)	φ	H (kN)	M (kNm)	φ	N (kN)	Hl (kN)	Ml (kNm)	**Ht (kN)**	**Mt (kNm)**
P2(Nmax)	41,089	1.25	3,800	0.50	17,903	161,724	1.00	**53,270**	5,323	43,149	**17,903**	**161,724**
(Nmin)	41,089	0.90	3,800	0.50	17,903	161,724	1.00	**38,890**	5,323	43,149	**17,903**	**161,724**

(3) Verification of Limit for Reinforcement

According to the AASHTO LRFD specifications, the verification of limit for reinforcement is obtained as followings.

1) Cross Section of Column

B =	13,700	mm
b_1 =	1,400	mm
b_2 =	10,900	mm
h =	2,800	mm



2) Longitudinal Reinforcement

			Num.	d_{ba}				
Horizontal-1		Layer-1	88	- D32	A_{st1} =	69,890	mm^2	@= 125
		Layer-2	88	- D32	A_{st2} =	69,890	mm^2	@= 125
	n =	2.0	176.0		A_{st} =	139,779	mm^2	

			Num.	d_{ba}				
Horizontal-2		Layer-1	88.00	- D32	A_{st1} =	69,890	mm^2	@= 125
		Layer-2	88.00	- D32	A_{st2} =	69,890	mm^2	@= 125
	n =	2.0	176.0		A_{st} =	139,779	mm^2	

			Num.	d_{ba}				
Vertical-1		Layer-1	30.00	- D32	A_{sc1} =	23,826	mm^2	@= 125
		Layer-2	28.00	- D32	A_{sc2} =	22,238	mm^2	@= 125
	n =	2.0	58.0		A_{ct} =	46,064	mm^2	

			Num.	d_{ba}				
Vertical-2		Layer-1	30.00	- D32	A_{st1} =	23,826		
		Layer-2	28.00	- D32	A_{st2} =	22,238	mm^2	@= 125
	n =	2.0	58.0		A_{st} =	46,064	mm^2	

n = 468 $\boxed{\Sigma A_{st} = 371,686\ mm^2}$

3) Transverse Reinforcement

4.0	- D25	A_{s1}	=	2,027	mm^2	@=	150

Steel Strength	f_y =	414.0	MPa
Concrete Strength	f'_c =	28.0	MPa
Elastic Modulus	E_s =	200,000	MPa

4) Limits for Reinforcement (LRFD 5.7.4.2)

- Minimum area of longitudinal reinforcement

$A_s f_y / A_g f'_c > 0.135$ *(LRFD 5.7.4.2-3)*

where:

Area of longitudinal reinforcement steel	A_s =	371,686	mm^2
Yield strength of reinforcing bars	f_y =	414	MPa
Area of section	A_g =	36,674,400	mm^2
Compressive strength of concrete	f'_c =	28	MPa

Minimum area of longitudinal reinforcem	$A_s f_y / A_g f'_c$ =	0.150	>	0.135	[OK]

As / Ag > 0.01 (LRFD 5.10.11.4.1a)

Minimum area of longitudinal reinforcem	A_s / A_g =	0.0101	>	0.010	[OK]

- Maximum area of longitudinal reinforcement

As / Ag < 0.08 (LRFD 5.7.4.2-1)
As / Ag < 0.04 (LRFD 5.10.11.4.1a)

Maximum area of longitudinal reinforcen	A_s / A_g =	0.010	<	0.040	[OK]

- Minimum area of Transverse reinforcement (LRFD 5.8.2.5)

$V_c = 0.083b(\sqrt{f'c})b_v s/f_y$ *(LRFD 5.8.3.3-3)*

where:

· Minimum transverse reinforcement (LRFD 5.8.2.5)

area of shear reinforcement within a distance	Av =	2027	mm^2
spacing of stirrups	*S* =	150	mm
effective web width taken as the minimum web (LRFD A5.8.2	b_v =	12,300	mm

(assumed as; bv=B-b 1)

Min. transverse reinf (mm^2)	A_{min}=	1,957	<	Av =	2,027	mm^2	[OK]

(4) Verification of Nominal Flexural Resistance of Column

Based on the FB-Multi Pier result, the verification of Nominal Flexural Resistance of column is obtained as followings, which R factor (response modification factor) is 1.0. According to the following results and minimum reinforcement, the response modification factor isn't required for this section.

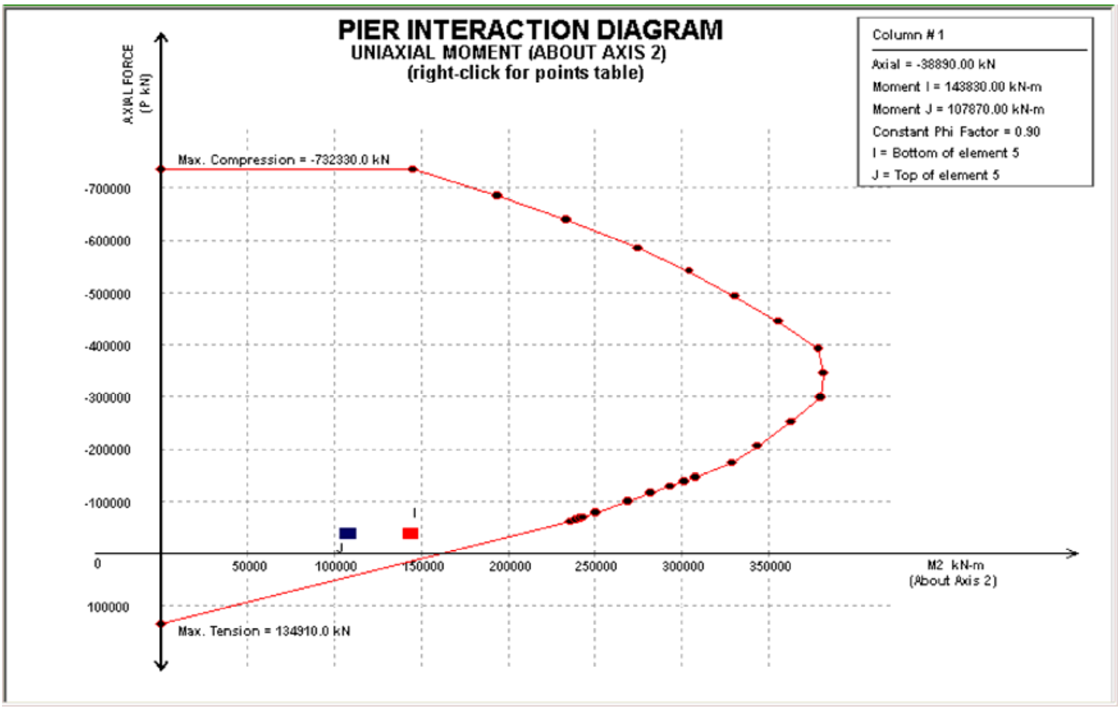


Figure 3.3.2-2 M-N Interaction Diagram & Design Reaction for longitudinal Direction (LL, Nmin, 2-D32@125)

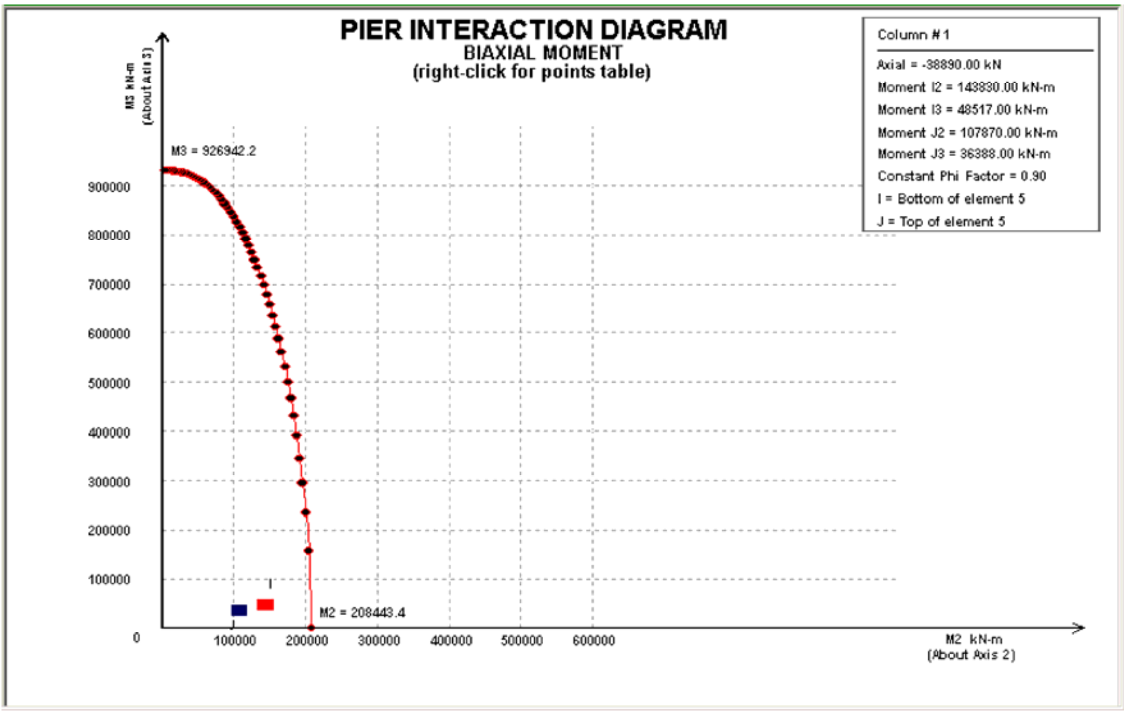
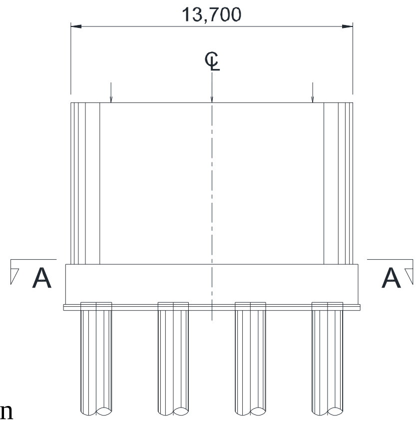


Figure 3.3.2-3 M-M Interaction Diagram & Biaxial Moment for Design Axial Force (at N=38,890kN, Nmin)

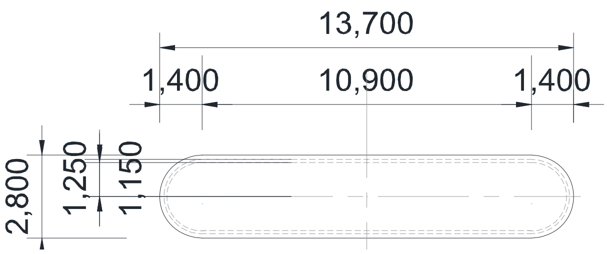
(5) Verification of Nominal Shear Resistance

1) Design Section (A-A)



2) Detail of Cross-section of Column

Conversion height with an equivalent Cross Sectional Area



A = 36.674 m^2

B = 13.700 m

H = 2.800 m

Beam/Effective Web unit Width	b =	13,700	mm
Total Beam Depth	h =	2,800	mm
Depth from compression face to Steel Centroid	d =	2,600	mm
Distance	$d's$ =	150	mm
Dist. from the base to lowest tension bar	$d1$ =	150	mm
Space for vertical direction	$d2$ =	100	mm
Dist. from extreme compression fiber to tension bar.	ds =	2,600	mm
Number of Tension bar layer	nt =	2	
Number of Compression bar layer	nc =	2	

Tension Reinforcement	1	118 - D32	A_{s1}	= 93,692 mm^2	, @ =	125
	1	116 - D32	A_{s2}	= 92,104 mm^2	, @ =	125
			ΣA_s	=185,796 mm^2		
Compression Reinforcem	2	234 - D32	A'_s	=185,796 mm^2	, @ =	125
			$\Sigma A'_s$	=185,796 mm^2		
	n=	468				

3) Characteristics of material

Steel Strength		f_y =	414	MPa
Concrete Strength		f'_c =	28.0	MPa
Elastic modulus	steel	Es =	200,000	MPa
	concrete	Ec =	25,000	MPa

4) Sectional force

Longitudinal direction of P2 at Nmin (refer to Table 3.3.1-1)

Factored Axial force:	*Nu =*	**38,890**	**kN**
Factored Moment:	*Mu =*	**143,831**	**kNm**
Factored Shear force for section	*Vu =*	**17,742**	**kN**

5) Verification of Shear Resistance

The factored shear resistance, Vr, shall be taken as;

$V_r = \phi V_n$ (LRFD 5.8.2.1-2)

where:

ϕ : resistance factor specified in A.5.10.11.4.1b

V_n : nominal shear resistance shall be taken as;

$V_u > 0.5\ \varphi\ (V_c +$ (LRFD 5.8.2.4-1)

$V_{n1} = Vc + Vs + V_p$ (LRFD 5.8.3.3-1)

$V_{n2} = 0.25f'_c b_v d_v + V_p$ (LRFD 5.8.3.3-2)

in which :

$V_c = 0.083b(\sqrt{f'c})b_v d_v$ (LRFD 5.8.3.3-3)

$V_s = A_v f_y d_v \cot\theta/s$ (LRFD C5.8.3.3-

resistance factor specified (LRFD A.5.10.11.4.1b)	ϕ	=	0.90	
	d_s	=	2,600	mm
the corresponding effective depth from the extreme compression (LRFD 5.8.2.9-2)				
de = As fy ds/As fy	d_e	=	2,600	mm
effective shear depth (A 5.8.2.9 max(0.9de, 0.72h)	d_v	=	2,340	mm
effective web width taken as the minimum web width within	b_v	=	12,300	mm
factor indicating ability of diagonally cracked concrete to transmit tension (LRFD A 5.8.3.4.1)	β	=	2.00	
angle of inclination of diagonal compressive stress (LRFD A 5.8.3.4.1)	θ	=	45.0	deg
angle of inclination of transverse reinforcement to longitudinal	α	=	90	deg
area of shear reinforcement within a distance S Av' =	D25	=	506.0	mm^2
number of shear reinforcement within S	n	=	4	
spacing of stirrups	S	=	150	mm
area of shear reinforcement within a distance S *(Av=n*s)*	Av	=	2024	mm^2

nominal shear resistance provided by tensile stress in concrete(LRFD 5.8.3. V_c = 25,282 kN

$Vc = 0.083b(\sqrt{f'c})bvdv$

shear resistance provided by shear reinforcement (LRFD C5.8.3.3-4) V_s = 13,072 kN

$Vs = A_v f_y d_v cotq/s$

nominal resistance (LRFD 5.8.3 $V_n = \min(V_{n1}, V_{n2})$ V_n = 38,354 kN

$V_{n1} = Vc + Vs + V_p$ (LRFD 5.8.3.3-1) = 38,354 kN

$V_{n2} = 0.25f'_c b_v d_v + V_p$ (LRFD 5.8.3.3-2) =201,474 kN

Regions requiring transverse reinforcement (5.8.2.4-1)					
V_u =	17,742	kN	>	$Vn = 0.5\,\varphi\,(V_c + V_p)$	
				= 11,377	kN

Transverse reinforcement is necessary

The factored resistance	V_u =	17,742	kN	<	φVn =	34,518	kN	-O.K-

3.3.3 Pile Foundation Design

(1) Pile Foundation Dimension

Based on that the following dimensions are conducted as the outline design for pile foundation of Mawo bridge, which are determined due to the following specification (for pile arrangement) and pile bearing with design forces (for pile diameter, pile length and pile arrangement).

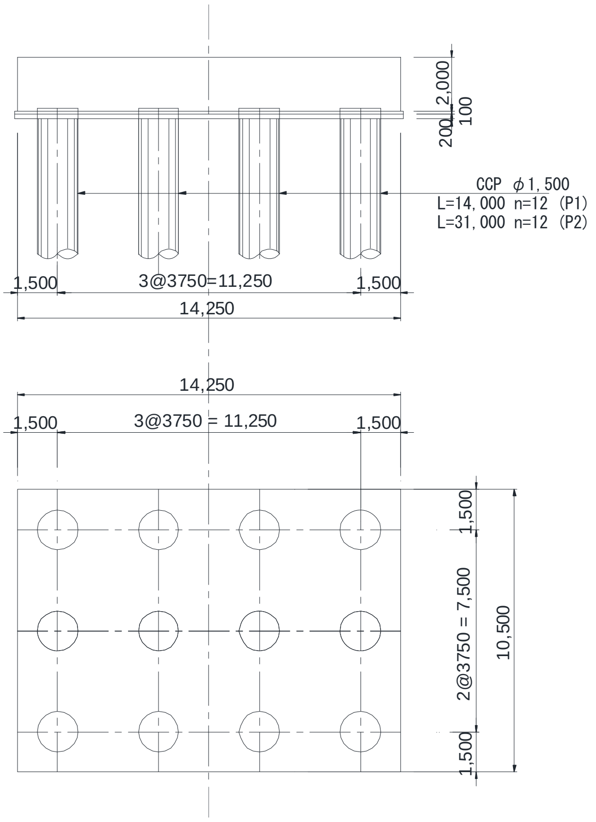


Figure 3.3.3-1 Dimension of Pile Foundation

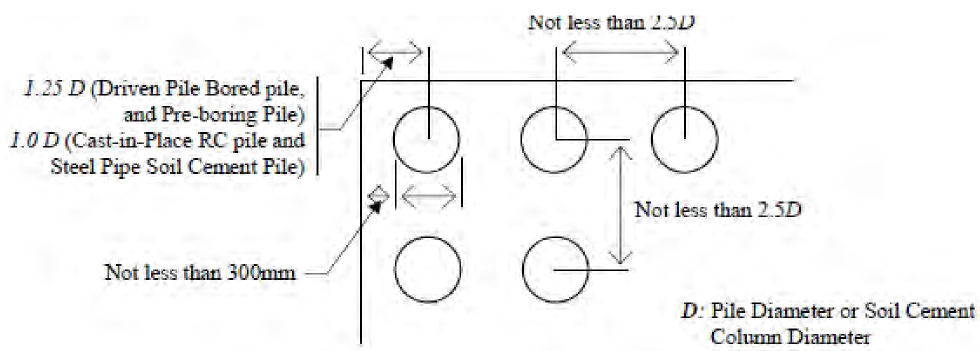


Figure 3.3.3-2 Minimum Distance between Pile Centers and Distance between Outermost Pile Center and Footing Edge (BSDS Figure C5.4.3.2-1)

(2) Design Load

Based on the results of seismic analysis and column design, summarize of design force of pile foundation is conducted for the following table, which are considered for load combination and following combination. In this design, column is not plastic hinging, thus the foundation design forces are obtained based on the analysis results.

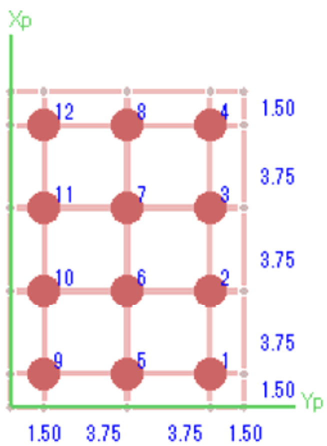
Table 3.3.3-1 Summarize of Design Forces at Bottom of Pile Cap

LOAD CASE	N (kN)	Hl (kN)	Ml (kNm)	Ht (kN)	Mt (kNm)
1) Extreme- Ⅰ LL(Nmax)	62,430	18,503	181,430	5,662	59,910
2) Extreme- Ⅰ LL(Nmin)	45,480	18,503	181,430	5,662	59,910
3) Extreme- Ⅰ TT(Nmax)	62,430	5,551	54,429	18,872	199,701
4) Extreme- Ⅰ TT(Nmin)	45,480	5,551	54,429	18,872	199,701

(3) Verification of Foundation Stability

1) Design Model

Based on that the following models are conducted for pile foundation of Mawo bridge. The values of the coefficient of subgrade reaction shall be obtained as Table 3.3-1-8.



Multipiler Value		
Line	X	Y
Space	3.75	3.75
1	1.5000	1.5000
2	3.7500	3.7500
3	3.7500	1.5000
4	1.5000	-

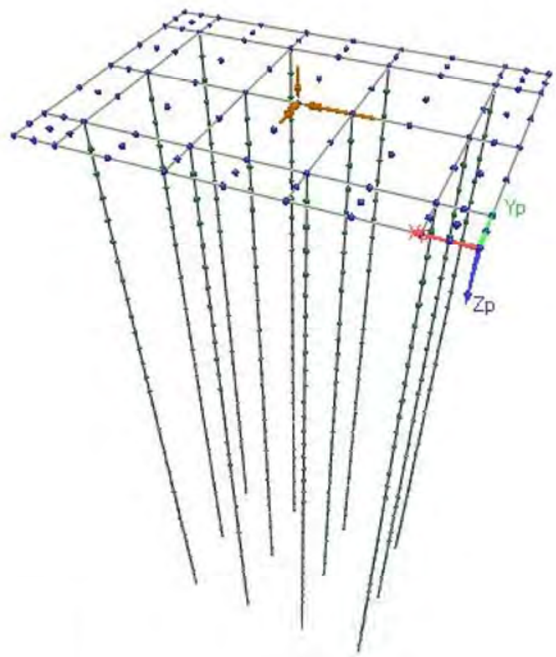


Figure 3.3.3-3 Design Model of Pile Foundation by FB-Multi Pier

2) Section Forces & Displacement at Pile Head

According to the foundation calculation by FB-Multi Pier, the section forces at pile head are obtained as followings.

-Case1. Extreme- Ⅰ Longitudinal direction (Nmax)

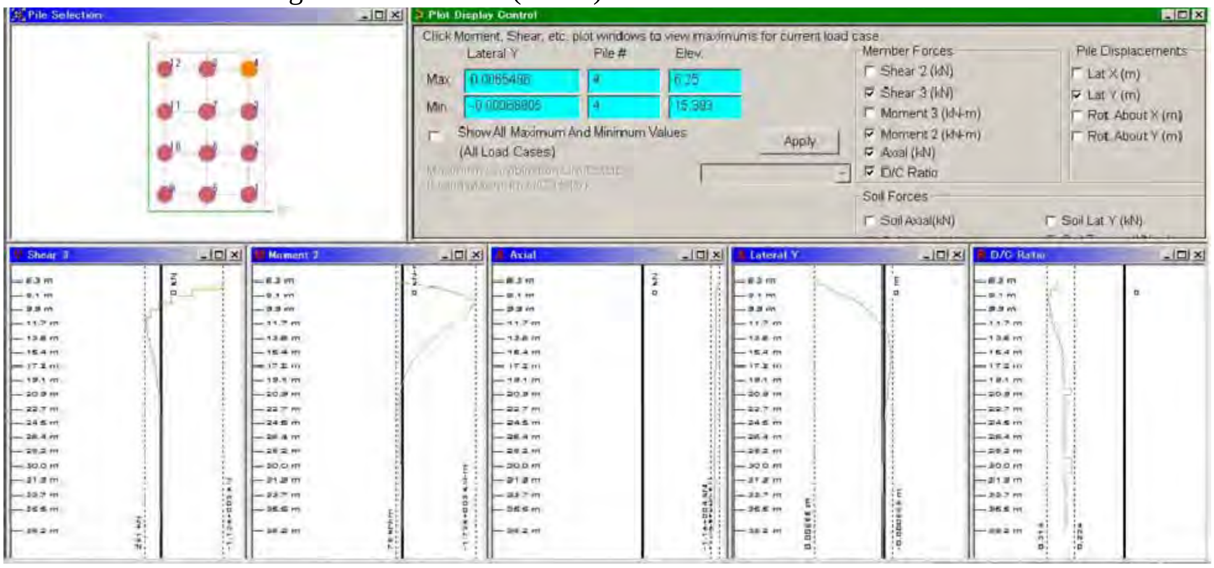


Figure 3.3.3-4 Summarize of calculation results for Longitudinal direction (Nmax)

Table 3.3.3-2 Summarize for the Section Forces at Pile Head (LL, Nmax)

Load Case	Pile No.	N	Hx	Hy	Mx	My	Dis. X	Dis. Y	
1)	1	-7,959	337	-1,079	-2,944	-795	0.00251	0.00873	
	2	-9,690	338	-1,079	-2,943	-794	0.00251	0.00873	
	3	-11,416	340	-1,079	-2,943	-791	0.00251	0.00873	
	4	-11,565	341	-1,079	-2,943	-790	0.00251	0.00873	**Nmax**
	5	-537	338	-1,079	-2,944	-795	0.00251	0.00873	
	6	-1,522	338	-1,079	-2,943	-794	0.00251	0.00873	
	7	-3,249	340	-1,079	-2,943	-791	0.00251	0.00873	
	8	-4,968	340	-1,079	-2,944	-791	0.00251	0.00873	
	9	-528	338	-1,078	-2,945	-794	0.00251	0.00873	
	10	-530	339	-1,078	-2,945	-793	0.00251	0.00873	
	11	-532	339	-1,078	-2,945	-793	0.00251	0.00873	
	12	-534	339	-1,078	-2,945	-792	0.00251	0.00873	

-Case2 Extreme- Ⅰ Longitudinal direction (Nmin)

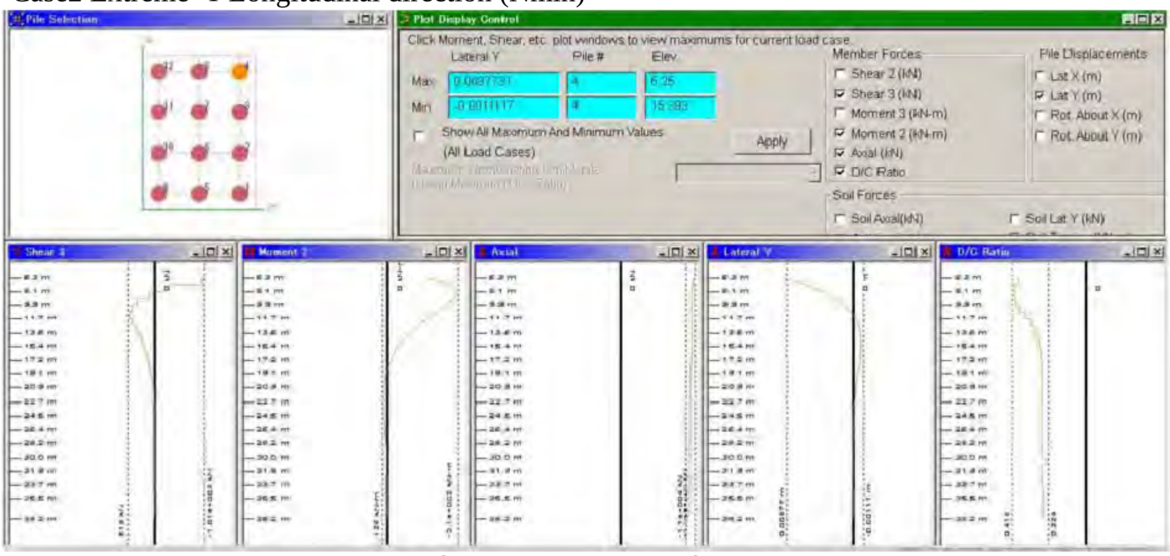


Figure 3.3.3-5 Summarize of calculation results for Longitudinal direction (Nmax)

Table 3.3.3-3 Summarize for the Section Forces at Pile Head (LL, Nmin)

Load Case	Pile No.	N	Hx	Hy	Mx	My	Dis. X	**Dis. Y**	
2)	1	-5,249	300	-958	-4,551	-1,288	0.00322	**0.0110**	
	2	-10,729	301	-959	-4,550	-1,287	0.00322	**0.0110**	
	3	-11,551	302	-959	-4,549	-1,284	0.00322	**0.0110**	
	4	-11,561	303	-959	-4,550	-1,283	0.00322	**0.0110**	
	5	-522	300	-958	-4,551	-1,288	0.00322	**0.0110**	
	6	-528	301	-959	-4,550	-1,287	0.00322	**0.0110**	
	7	-534	302	-959	-4,550	-1,284	0.00322	**0.0110**	
	8	-2,529	302	-958	-4,550	-1,284	0.00322	**0.0110**	
	9	-502	301	-958	**-4,552**	-1,287	0.00322	**0.0110**	**Mmax**
	10	-508	301	-958	-4,552	-1,286	0.00322	**0.0110**	
	11	-513	301	-958	-4,552	-1,286	0.00322	**0.0110**	
	12	-519	301	-958	-4,552	-1,286	0.00322	**0.0110**	

Displacemen Max

< 15 mm (1%of pile diameter)

- OK -

-Case3 Extreme- I Transverse direction (Nmax)

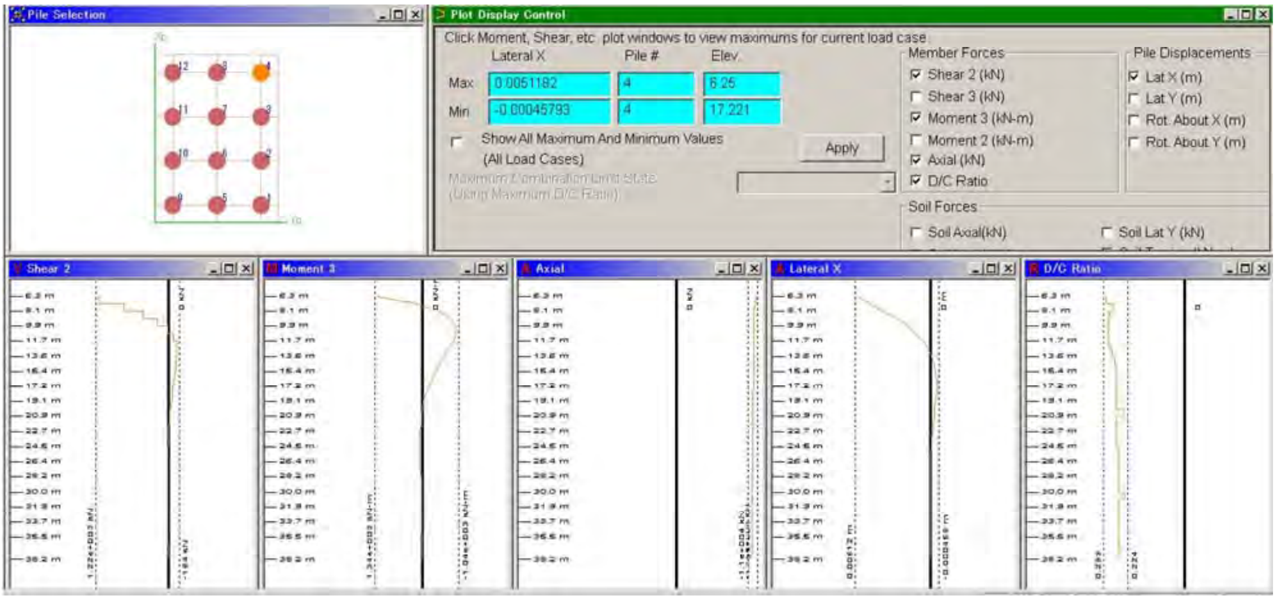


Figure 3.3.3-6 Summarize of calculation results for Transverse direction (Nmax)

Table 3.3.3-4 Summarize for the Section Forces at Pile Head (TT, Nman)

Load Case	Pile No.	N	Hx	Hy	Mx	My	Dis. X	Dis. Y	
3)	1	-535	1,243	-326	-847	-1,452	0.00619	0.00257	
	2	-4,449	1,244	-327	-846	-1,451	0.00619	0.00257	
	3	-11,153	1,246	-327	-844	-1,450	0.00619	0.00257	
	4	-11,559	**1,246**	-327	-845	-1,450	0.00619	0.00257	**Smax**
	5	-530	1,243	-326	-846	-1,452	0.00619	0.00257	
	6	-573	1,244	-327	-846	-1,452	0.00619	0.00257	
	7	-7,281	1,246	-327	-846	-1,450	0.00619	0.00257	
	8	-11,548	1,246	-326	-846	-1,450	0.00619	0.00257	
	9	-526	1,243	-326	-846	-1,452	0.00619	0.00257	
	10	-533	1,244	-326	-847	-1,451	0.00619	0.00257	
	11	-3,401	1,245	-326	-848	-1,450	0.00619	0.00257	
	12	-10,099	1,246	-326	-848	-1,450	0.00619	0.00257	

- Case4 Extreme- Ⅰ Transverse direction (Nmin)

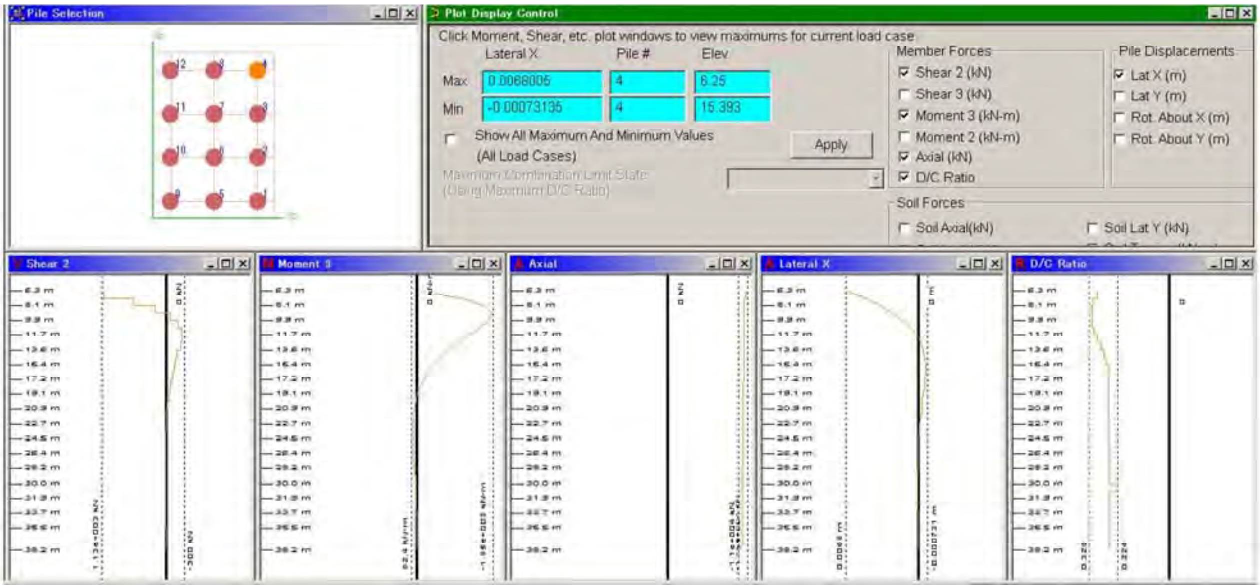


Figure 3.3.3-7 Summarize of calculation results for Longitudinal direction (Nmin)

Table 3.3.3-5 Summarize for the Section Forces at Pile Head (LL, Nmin)

Load Case	Pile No.	N	Hx	Hy	Mx	My	Dis. X	Dis. Y
4)	1	-519	1,125	-281	-1,450	-2,675	0.00841	0.00342
	2	-532	1,126	-281	-1,450	-2,674	0.00841	0.00342
	3	-8,085	1,128	-282	-1,448	-2,672	0.00841	0.00342
	4	-11,563	1,128	-282	-1,448	-2,672	0.00841	0.00342
	5	-513	1,125	-281	-1,450	-2,675	0.00841	0.00342
	6	-526	1,126	-281	-1,449	-2,675	0.00841	0.00342
	7	-2,042	1,128	-281	-1,449	-2,672	0.00841	0.00342
	8	-11,550	1,128	-281	-1,449	-2,672	0.00841	0.00342
	9	-506	1,125	-281	-1,449	-2,675	0.00841	0.00342
	10	-519	1,126	-281	-1,449	-2,674	0.00841	0.00342
	11	-533	1,127	-281	-1,451	-2,673	0.00841	0.00342
	12	-8,354	1,128	-281	-1,450	-2,672	0.00841	0.00342

3) Verification of Pile bearing capacity & foundation stability

- Pile Data

Type of (1 : End Bearing Pile, 2 : Friction Pile)	Type	=	1	-
Diameter of Pile	D	=	1.50	m
Pile Top Elevation	Top E.L	=	-7.25	m
Pile Tip Elevation	Tip E.L	=	-38.25	m
Water Elevation	W.E	=	0.50	m
Unit Weight of Pile	Wp	=	14.50	kN/m3
Length of Pile	L	=	31.00	m
Perimeter of Pile	U	=	4.71	m
Area of Pile Section	Ap	=	1.77	m2

- Ground Data & Shaft Resistance

Pile Self Weight(Ws)

Layer	Soil Type	E.L (m)	Thickness (m)	N-Value	Li (m)	γi (kN/m3)	Ws (kN) =Ap*Li*γi
1	S	-7.25	6.55	21	6.55	7.20	83.34
2	C	-13.80	13.00	8	13.00	7.20	165.40
3	S	-26.80	3.00	17	3.00	7.20	38.17
4	S	-29.80	7.00	31	7.00	8.20	101.43
5	S	-36.80	1.45	50	1.45	10.20	26.14
					31.00	ΣWs =	414.48 kN

Nominal shaft resistance(UΣLifi)

Layer	Soil Type	E.L (m)	Thickness (m)	N-Value	Li (m)	fi (kN/m2)	U･Li･fi (kN)
1	S	-7.25	6.55	21	6.55	105	3,241
2	C	-13.80	13.00	8	13.00	80	4,901
3	S	-26.80	3.00	17	3.00	85	1,202
4	S	-29.80	7.00	31	6.95	155	5,076
5	S	-36.80	1.45	50	0.00	200	0
				Nominal shaft resistance		UΣLifi =	14,420 kN

NOTE : Soil Type = S : Sand C : Clay
E.L = Elevation of Upper of Layer (m)
Thickness = Thickness of the Layer (m)
N-Value = Average N-Value of the Layer
Li = Thickness of Layer considering shaft resistance (m)
fi = Maximum shaft resistance of Layer considering pile shaft resistance (kN/m2)

- Pile Tip Resistance

Soil Type at Pile Tip (1:Sand, 2:Stiff Clay)	Type	=	1	-
N value at Pile Tip	N	=	50	-
Unconfined compressive strength	qu	=	-	kN/m2
Nominal end bearing capacity per unit area	qd	=	5,000	kN/m2
(Sundy Gravelly, according to BSDS Table C5.4.3.3-1)				
Nominal end bearing resistance	qd*Ap	=	8,836	kN

- Bearing Capacity

Nominal bearing capacity of pile (Ru =qd*Ap+UΣLifi)	Ru	=	23,256	kN
Effective weight of pile and soil inside pile (W=Ap*L*Wp)	W	=	794	kN
Effective weight of soil replaced by pile (according to above table)	Ws	=	414	kN
Resistance Factor for pile under extreme event limit (according to BSDS A 5.4.1(5))	ϕ	=	0.65	-
Modification Coefficient Depending on Nominal Be (according to BSDS Table 5.4.3.3-1)	γ	=	1.0	-
Allowable Bearing Capacity	Ra	=	14,322	kN
	> Nmax	=	11,564	kN OK

NOTE : $Ra = \gamma(\phi Ru - Ws) + Ws - W$

Ws = Effective weight of soil replaced by pile (kN) = Ignore (refer to Table 3.3.3-2)

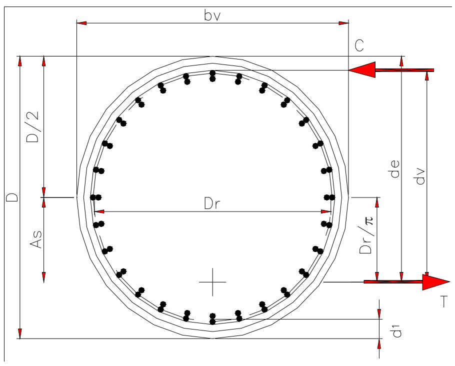
(4) Verification of Pile Section

1) Verification of Minimum Reinforcement

- Section Condition

External diameter of the circular member	b_v =	1,500	mm
External diameter of the circular member	D =	1,500	mm
Total Beam Depth for Shear	=	1,500	mm
Diameter of centers of the longitudinal re-b	Dr =	1,200	mm
Depth from to Steel Centroid	d =	1,350	mm
Distance	$d's$ =	0	mm
Dist. from highest to lowest tension rein.	$d1$ =	150	mm
	$d2$ =	0	mm
Effective Cover to Center of Closest Bar	d_c =	50	mm
Dist. from compression bar to tension bar.	ds =	1,350	mm
Steel Strength	f_y =	414.0	Mpa
Concrete Strength	f'_c =	28.0	Mpa
Elastic modulus	Es =	200,000	Mpa
	Ec =	25,000	Mpa
Number of Tension bar layer	nt= =	1	
Number of Compression bar layer	nc= =	1	

Tension Reinforcem 1	16 - D25	A_{s1} =	8,107	mm^2 ,	@ =	118
Compression Reinfo 1	16 - D25	A'_s =	8,107	mm^2 ,	@ =	118
		ΣA_s =	16,214	mm^2		
Skin Reinforcemetotal;	2.0 - D20	A_{s1} =	628	mm^2 ,	@ =	500
		f_y =	275.0	Mpa		



- Limits for Reinforcement (LRFD 5.7.4.2)

Minimum area of longitudinal reinforcement

As / Ag > 0.0075 (LRFD 5.13.4.6.3.d)

Area of longitudinal reinforcement steel	A_s =	16,214 mm^2
Area of section	A_g =	1,766,250 mm^2

Minimum area of longitudinal reinforcem	A_s / A_g =	0.0092	>	0.0075	[OK]

2) Verification of Nominal Flexural Resistance of Pile

Based on the FB-Multi Pier result, the verification of Nominal Flexural Resistance of pile is obtained as followings, which the reinforcement is change the minimum reinforcement (D25@118) to D28@118 due to require nominal flexural resistance.

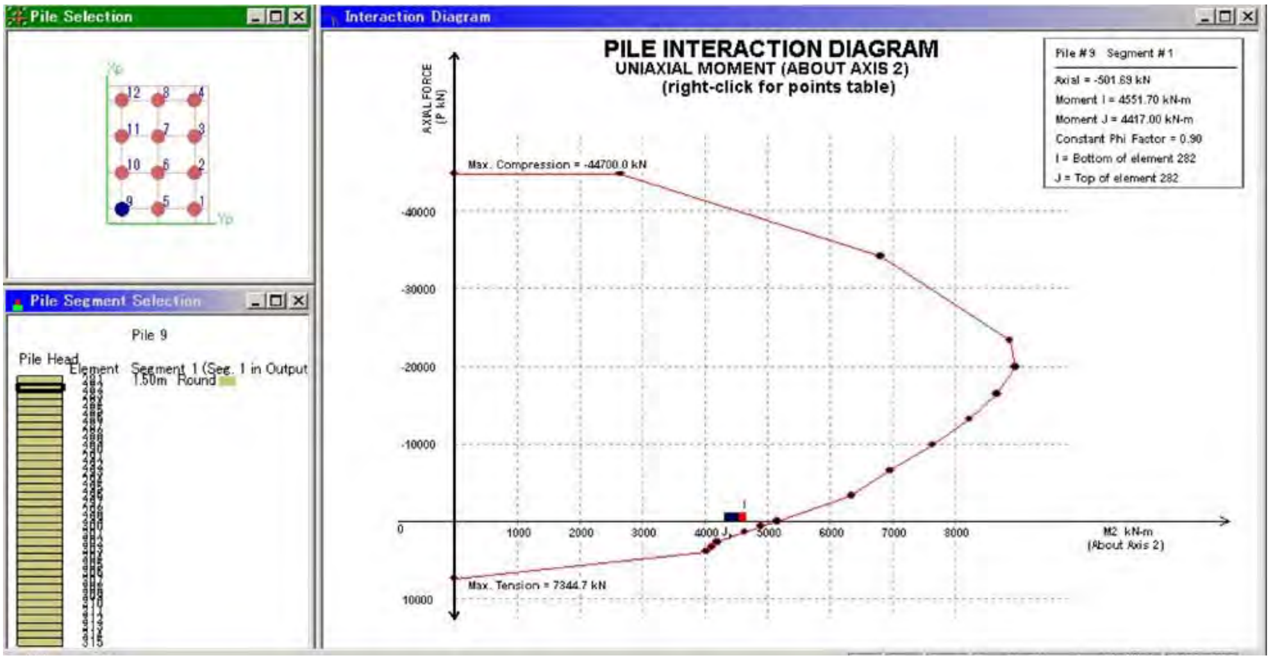


Figure 3.3.3-8 M-N Interaction Diagram & Design Reaction for Longitudinal Direction at No9 pile (LL, Nmin, D28@118)

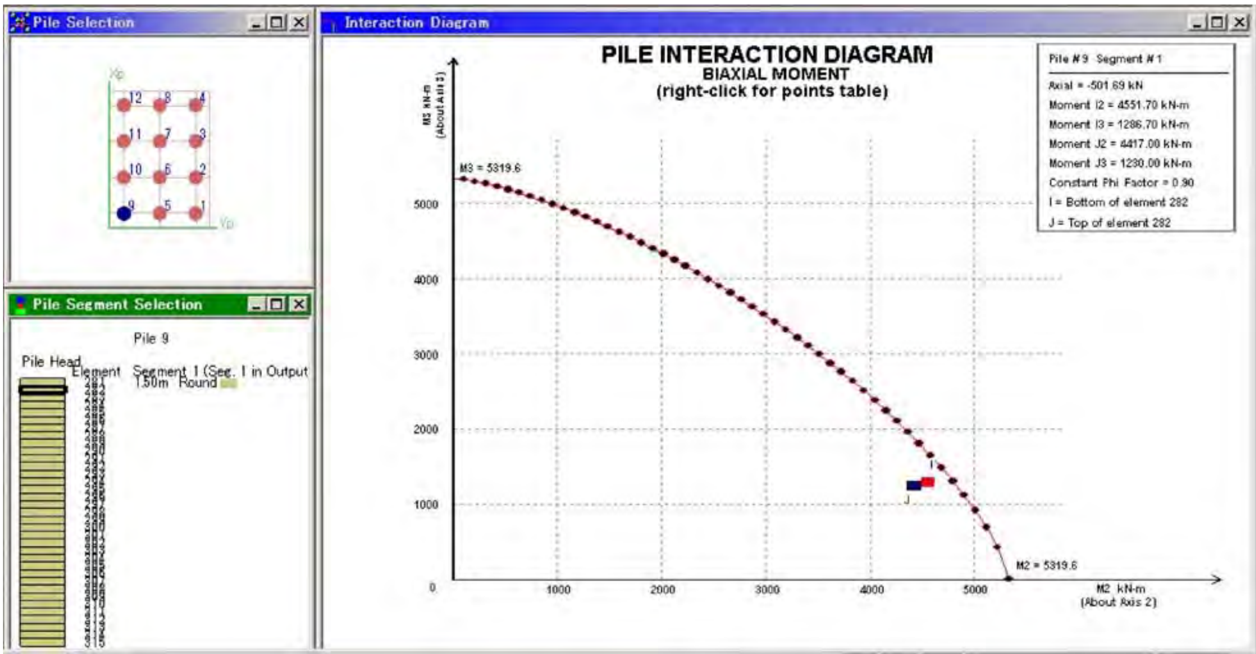


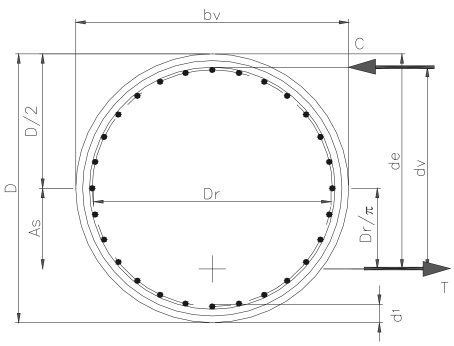
Figure 3.3.3-9 M-M Interaction Diagram & Biaxial Moment of No9 Pile

3) Verification of Nominal Shear Resistance

- Pile Section Data

Beam/Effective Web Width	b, b_v	=	1,500	mm
Total Beam Depth for Moment	D	=	1,500	mm
Total Beam Depth for shear	h	=	1,500	mm
Depth from to Steel centroid	d	=	1,350	mm
Dist. from highest to lowest tension rein.	$d1$	=	150	mm
Dist. from highest to lowest tension rein.	0	=	0	mm
Effective Cover to Center of Closest Bar	d_c	=	50	mm
Dist. from compression bar to tension bar.	Dr	=	1,200	mm
resistance factor (5.5.4.2)	f	=	0.9	(for Strength)
	f	=	1.0	(for Extreme)
Steel Strength	f_y	=	414	Mpa
Concrete Strength	f'_c	=	28	Mpa
Elastic modulus	Es		200,000	Mpa
	Ec		25,000	Mpa
Number of Tension bar layer	nt=		1	
Number of Compression bar layer	nc=		1	

Tension Reinforcement	16 - D28	A_{s1}	= 7,856 mm^2 ,	@	118
Compression Reinforcemei	16 - D28	A'_s	= 7,856 mm^2 ,	@	118
		ΣA_s	=15,712 mm^2		



- Selected Section Force (Shear Max) **(refer to Table 3.3.3-4)**

Pier:	P2		
Load Combination:	Extreme(Nmax)		
Direction (Pile No):	Longitudinal - Pile No.4		
Factored Axial Force:	Nu =	11,559	kN
Factored Moment:	Mu =	1,450	kN.m
Factored Shear force for section:	Vu =	1,246	kN
Factored Torsional Moment:	Tu =	0	kN.m

- Nominal Shear Resistance (LRFD 5.8.2.1)

The factored shear resistance, Vr, shall be taken as;

$V_r = \phi V_n$ (LRFD 5.8.2.1-2)

where

ϕ : resistance factor specified in LRFD A.5.10.11.4.1b

V_n : nominal shear resistance shall be taken as;

$V_u > 0.5\ \phi\ (V_c + V_p)$ (LRFD 5.8.2.4-1)

$V_{n1} = Vc + Vs + V_p$ (LRFD 5.8.3.3-1)

$V_{n2} = 0.25f'_c b_v d_v + V_p$ (LRFD 5.8.3.3-2)

in which :

$V_c = 0.083b(\sqrt{f'c}) b_v d_v$ (LRFD 5.8.3.3-3)

$V_s = A_v f_y d_v \cot \theta / s$ (LRFD C5.8.3.3-1)

resistance factor specified (LRFD A.5.10.11.4.1b)	ϕ =	0.9	
	d_s =	1,350	mm
the corresponding effectiv $V_c = 0.083b(\sqrt{f'c}) b_v s/f_y$ (LRFD 5.8.3.3-3)			
$de = As\ fy\ ds/As\ fy$	d_e =	1,132	mm
effective shear depth (LRFD A 5.8.2.9) max(0.9de, 0.72h)	d_v =	1,080	mm
effective web width taken as the minimum web width within	b_v =	1,500	mm
factor indicating ability of diagonally cracked concrete to transmit t (LRFD A 5.8.3.4.1)	β =	2.00	
angle of inclination of diagonal compressive stress (LRFD A 5.8.3.	θ =	45.0	deg
angle of inclination of transverse reinforcement to longitudinal	α =	90	deg
area of shear reinforcement within a distance *s* A_v' =	D20 =	314.2	mm^2
number of stirrups	S_s =	2	p
area of shear reinforcement within a dist *(Av=n*s)*	A_v	628.3	mm^2
spacing of stirrups	s =	150	mm
nominal shear resistance provided by tensile stress in concrete	V_c =	1,423	kN
$Vc = 0.083b(\sqrt{f'c})bvdv$ (LRFD 5.8.3.3-3)			
shear resistance provided by shear reinforcement	V_s =	1,873	kN
$Vs = A_v f_y d_v cotq/s$ (LRFD C5.8.3.3-4)			
nominal resistance (LRFD 5.8.3.3-1, -2)	V_n =	3,296	kN
$V_{n1} = Vc + Vs + V_p$ (LRFD 5.8.3.3-1)	=	3,296	kN
$V_{n2} = 0.25f'_c b_v d_v + V_p$ (LRFD 5.8.3.3-2)	=	11,340	kN

Regions requiring transverse reinforcement (LRFD 5.8.2.4-1)

V_u = 1,246 kN > $Vn = 0.5\ \varphi\ (V_c + V_p)$

640 kN (not enough)

Transverse reinforcement is necessary

The factored resistance	1,246 kN	<	φVn =	2,966 kN	-O.K-

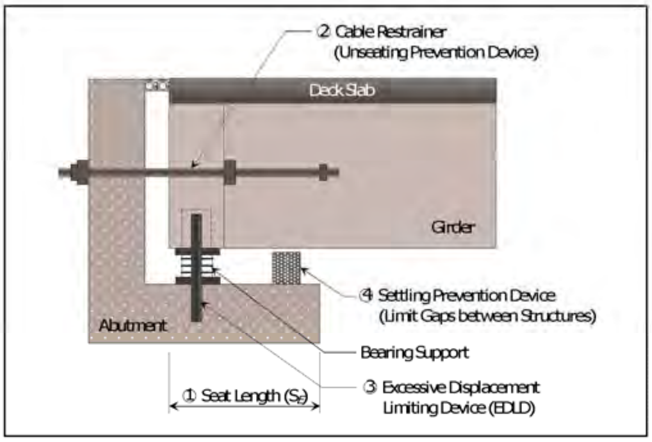
4. Design Example of Unseating Prevention System

4.1 Design Requirement

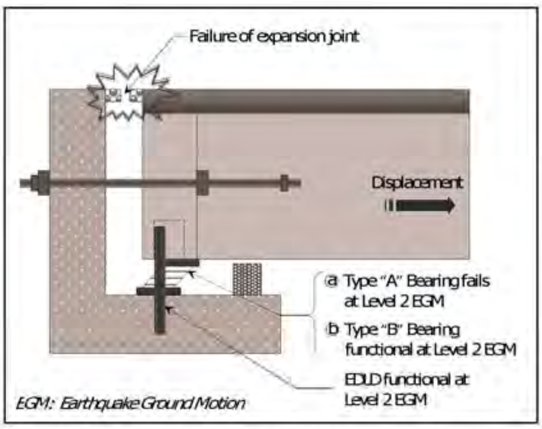
An unseating prevention system consists of the seat length of the girder at the support, unseating prevention device, device limiting excessive displacement, and device to prevent the superstructure from settling (limiting vertical gap in superstructure). These components shall be appropriately selected in accordance with the bridge type, type of bearing supports, ground conditions, and other factors. In this chapter, the design example of the unseating prevention system is explained in accordance with the SECTION 7 of DPWH BSDS.

The superstructure is generally connected to the substructures through bearings that may be structurally weak point against huge seismic forces. As such, the superstructure and the substructure are separated functionally; significantly critical state such as bridge falling down may be caused due to large relative displacements between them in case of failure of bearings under unexpected seismic forces.

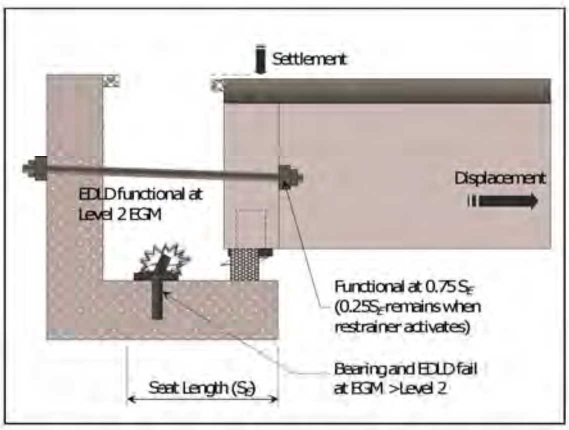
For a functional system preventing such severe state, detailed philosophy and articulate design concepts are explicitly specified in the new DPWH BSDS as “Unseating Prevention System” based on accumulated data and experiences from large number of seismic damages. The aim is to provide multiple mechanisms that can complement each other efficiently, shown as follows.



(a) Layout of Unseating Prevention System



(b) Performance under design earthquake
(Level 2 EGM)



(c) Performance under unexpected earthquake
(EGM > Level 2)

Figure 4.1-1 Mechanism of Unseating Prevention System

For Mawo bridge has 3 spans with 4 supports longitudinally. However, the JICA Study Team recommends continuous bridge not multi-simple-supported bridge from the aspect of seismic behavior. Therefore, major countermeasures around the intermediated piers may not be necessary to be treated. The unseating prevention system specified in the new DPWH BSDS consists of multiple devices or countermeasure in order to prevent the severe state from the important point of view of failsafe functions. In the new DPWH BSDS, in order to understand the fundamental principle of the system smoothly, the following figure is described, which may be useful and efficiently utilized even by the bridge engineer who has not ever designed this system.

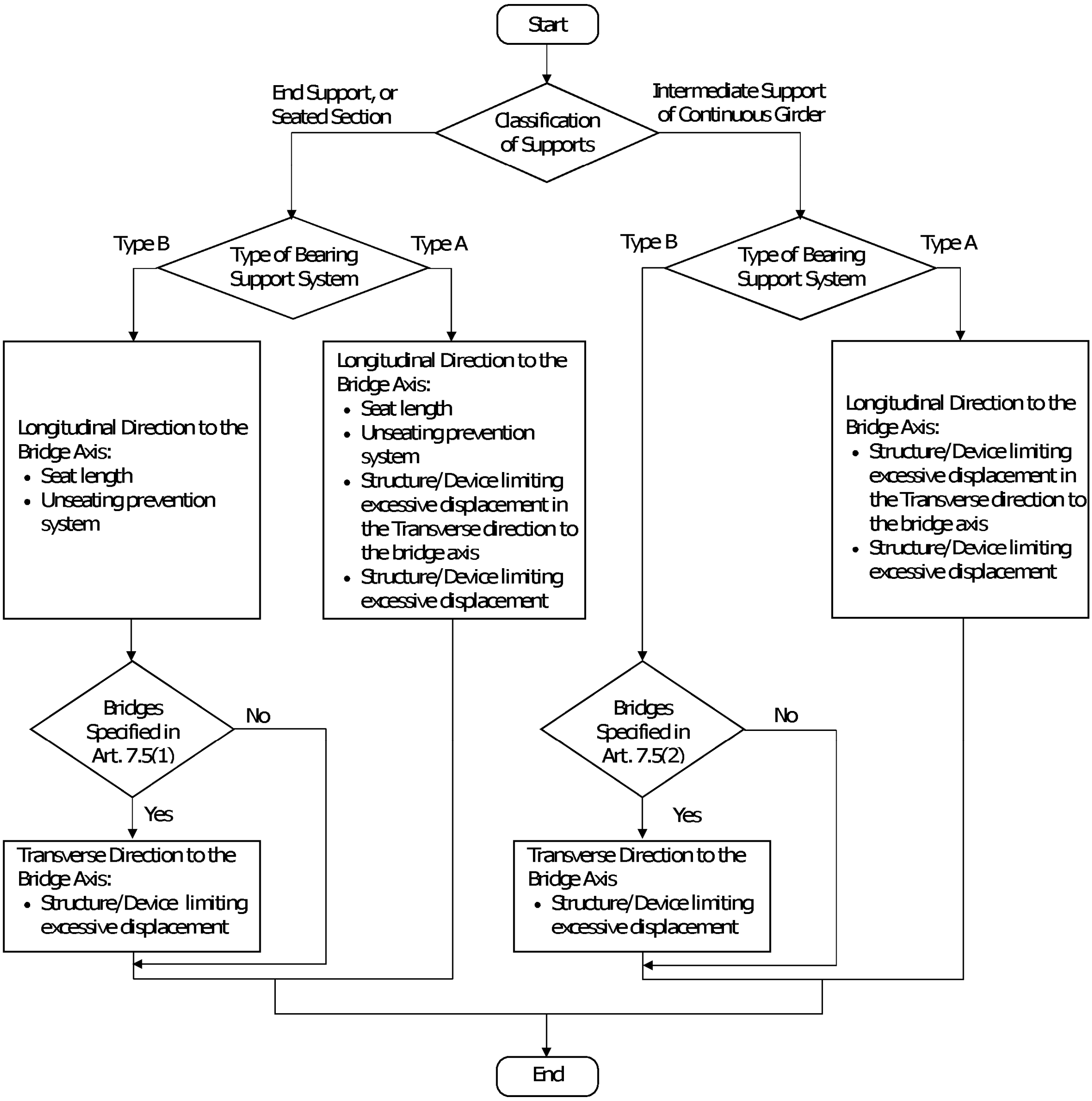


Figure 4.1-2 Fundamental Principles of Unseating Prevention System

According to the fundamental principle, the specified requirements of the system for Mawo bridge can be automatically determined shown as following tables.

Table 4.1-1 Design Requirement of Unseating Prevention System for Mawo Br.

END SUPPORT (A1 and A2)	
Bearing Type	- **Type B: Laminated Rubber Bearing** (If Type A bearing selected, limiting excessive displacement device is necessary for both directions such like dowel bars.
Devices for Longitudinal Direction	- **Adequate Seat Length** - **Unseating Prevention Device**
Devices for Transversal Direction	No Requirements, Structures and Devices (Mawo bridge is NOT skew and curve bridge)
INTERMEDIATE SUPPORT (P1 and P2)	
Bearing Type	- **Type B: Laminated Rubber Bearing** (If Type A bearing selected, limiting excessive displacement device is necessary for both directions such like dowel bars.
Devices for Longitudinal Direction	No Requirements, Structures and Devices
Devices for Transversal Direction	No Requirements, Structures and Devices

4.2 Seat Length

The seat length of a girder at its support shall not be less than the value obtained from either the value of S_E or S_{EM} specified in the new DPWH BSDS. Here, the seat length shall be measured in the direction perpendicular to the bearing support line when the direction of soil pressure acting on the substructure differs from the bridge axis, as in cases of a skew bridge or a curved bridge.
The values of S_E or S_{EM} can be obtained by using following the equations, specified in BSDS.

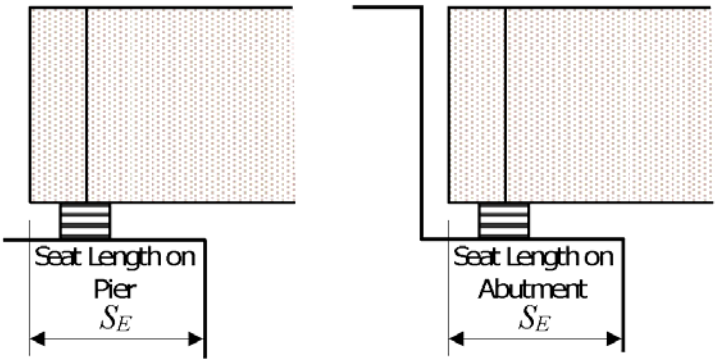


Figure 4.2-1 Value of SE

$S_E = u_R + u_G \geq S_{EM}$ Eq. 7.2-1 in DPWH BSDS
$S_{EM} = 0.70 + 0.005l$: l = Span length (m) Eq. 7.2-2 in DPWH BSDS
$u_G = \varepsilon_G L$ Eq. 7.2-3 in DPWH BSDS

Where:
SE : Seat length of the girder at the support, (m). SE is the length measured from the end of girder to the edge of the top of the substructure, or the girder length on the hinge/bearing joint, as shown in above figure.
u_R : Maximum relative displacement between the superstructure and the edge of the top of the substructure due to Level 2 Earthquake Ground Motion, (m). In calculating u_R, the effects of the unseating prevention structure and the structure limiting excessive displacement shall not be considered. When soil liquefaction and lateral spreading may affect displacement of the bridges, such effects shall be considered.
u_G : Relative displacement of the ground caused by seismic ground strain, (m).
S_{EM} : Minimum seating length of a girder at the support, (m).

ε_G : Seismic ground strain. ε_G can be assumed as 0.0025, 0.00375 and 0.005 for Ground Types I, II and III, respectively.
L : Distance between two substructures for determining the seat length (m).
l : Length of the effective span (m). When two superstructures with different span lengths are supported on one bridge pier, the longer of the two shall be used.

Here, based on the results of response spectrum analysis, the required seat length of both abutments are obtained.

< Determination of the value f u_R>
The value of u_R is Maximum relative displacement between the superstructure and the edge of the top of the substructure due to Level 2 Earthquake Ground Motion, (m). From the result of response spectrum analysis, the value can be obtained as 0.350m.

< Determination of the value u_G>

$u_G = \varepsilon_G L$

Where,

ε_G : Seismic ground strain: A1 (Ground Type I) 0.0025, A2 (Ground Type III) 0.0050

L : Distance between two substructures for determining the seat length (m) as follows
61.55m + 80m/2 = 101.55m

Therefore, u_G = 0.0025 x 101.55= 0.254m for A1
0.0050 x 101.55= 0.508m for A2

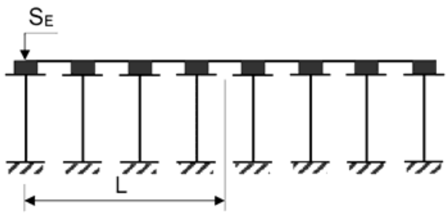


Figure 4.2-2 Value of L under Rubber Bearings, specified in BSDS

< Determination of required seat length>
$S_{EM} = 0.70 + 0.005l$: l = Span length (m)
0.70 + 0.005 x 61.55m = 1.01m

Where,

$S_E = u_R + u_G$:
0.350 + 0.254 = 0.614 m for A1 < S_{EM}
0.350 + 0.508 = 0.858 m for A2 < S_{EM}

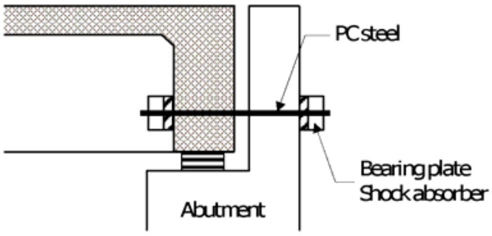
Thus,
The value of S_{EM} is applied to required seat length for both side of abutments.
Therefore,

$S_E = S_{EM} = 1.01m ≒ 1.1m$ or more

4.3 Unseating Prevention Device

As the unseating prevention device, various type of devices are being applied in Japan. For Mawo bridge, the unseating prevention device is required to be installed only at end support for longitudinal direction based on the design requirement. Therefore, the following table shows that explanation and applicability for end-support condition of generally utilized types of unseating prevention devices..
In this design example, a type of longitudinal cable restrainer type is adopted from the efficiency of the most appropriate applicability and generally utilized for new bridge.

Table 4.3-1 Unseating Prevention Device for End-Support

Longitudinal Cable Restrainer Type 	- Connected by PC cable between the abutment parapet and end cross beam of girder - Applicable to various bridge types - Not require brackets to be anchored to the girder - No need to modify primary members - Commonly utilized for New bridge 
Outside Installation Type (Chain Type) 	- Connected by Chain between outside of substructure and girder - Easy installation - Extra works required - Damage primal members - Few actual results applied to new bridges - Ordinary utilized in bridge retrofitting project
Outside Installation Type (Cable Type) 	- Connected by PC cable between outside of substructure and girder - Easy installation - Extra works required - Damage primal members - Few actual results applied to new bridges - Ordinary utilized in bridge retrofitting project
Bracket Type 	- Install steel or concrete bracket onto abutment and girder - Complicated installation - Disturb maintenance of other devices due to this extra devices - Extra works required - Damage primal members - Few actual results applied to new bridges - Ordinary utilized in bridge retrofitting project

In the DPWH BSDS, the ultimate strength of an unseating prevention device shall not be less than the design seismic force determined by following equations.

1) When the unseating prevention device directly connects the superstructure with the substructure, the design seismic force shall be:

$H_F = P_{LG}$

however, $H_F \leq 1.5\ R_d$.

where,

H_F : Design seismic force of the unseating prevention device, (kN).

P_{LG} : For abutments, this shall be the lesser value corresponding to the lateral (horizontal) capacity of the breast wall calculated from its nominal flexural resistance, or the nominal shear resistance of the breast wall, (kN).

In this case, the equation 1) shall be applied for the outline design.

The following table shows the comparison of design forces such as H_F and R_d .

Table 4.3-2 Unseating Prevention Device for End-Support

Design Forces	Value of Design Forces
P_{LG}	Design force from ultimate strength: Shear capacity of abutment wall: S_y = 24000 kN $P_{Sy} = S_y$ = 24000 kN
1.5 R_d	Design force from dead load reaction: R_d = 8350 kN 1.5 R_d = 12525 kN
Applied Force	1.5 $R_d < P_{Sy} = S_y$ = 24000 kN Therefore, "1.5 R_d " is applied for the design force.

Herein, in this example, comparison in the above table is introduced; however, for the new bridge design, basically the design forces for the unseating device is taken as 1.5 times the dead load reaction, when the unseating device connects adjacent superstructures directly or when the unseating device connects the superstructure directly to the substructure. However, in some cases when the unseating device directly connects the superstructure with the substructure, the horizontal design force taken as $1.5R_d$ may result to unnecessarily oversized or overspecified unseating device. In this case, the design forces for the unseating prevention device can be taken as the P_{LG} especially for retrofit or maintenance design.

Design force: 1.5 R_d = 12525 kN

Here, 4 of PC cables resist the design force,

Thus, 12525/4nos = 3132kN/nos

Therefore,

The design force per a set of the restrainer: 3132kN/nos

Based on the obtained design force, the following stress check can be obtained.

Table 4.3-3 Verification of Longitudinal Restrainer

	1.5Rd: Design Forces/ nos	Allowance
PC Cable Type 19 x 12.7Hmm 4-nos/ Abutment	3132 kN/nos.	3667 kN

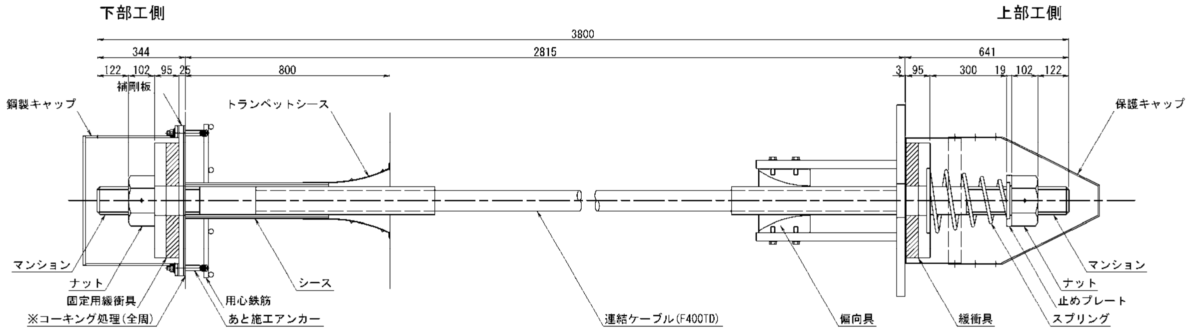


Figure 4.3-1 Longitudinal Restrainer for Mawo Bridge

4.4 Countermeasure to Prevent Superstructure from Settling

This structure or device is not an element of the unseating prevention system but a structure to keep any gaps on the road surface as small as possible after an earthquake with a large magnitude when the bearing support system is damaged, so that residents can proceed with an emergency evacuation and that the emergency vehicles can pass as soon as possible.

Therefore, this structure or device can not be connected directly with bridge unseating prevention system; however the total height of bearings applied to Mawo bridge in this outline design stage are about 600mm, which is comparatively taller than any other general bearing being adopted in Philippines; thus, adequate countermeasure to prevent superstructure from settling may be considered.

The design methodology of structure to prevent superstructure from settling is not complicated just for preventing the superstructure from settling can resist the vertical load from the superstructure. The commonly utilized structure in Japan is designed by RC structure installed on bearing seat, however, for the bearing seat of some small abutment or piers, this structures may occupy the seat, disturbing maintenance works around the bearing. Therefore, a lot of bridge owners request to the engineer to omit the installation of this structure. From the reason, the application for Mawo bridge should be carefully determined with close discussion between the engineer and the bridge owner in detail design stage.

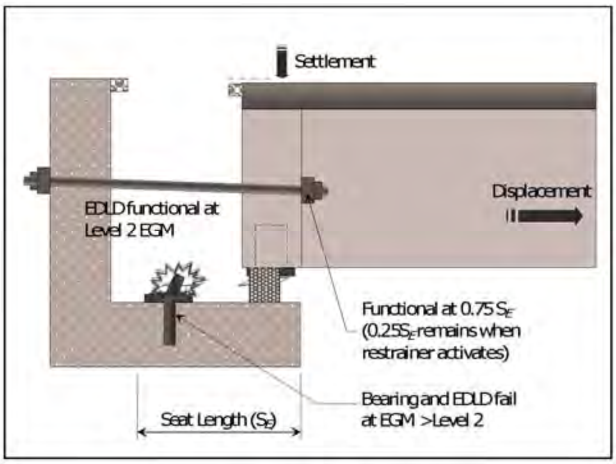


Figure 4.4-1 Concept of Prevention of Superstructure Settling

4.5 Structure/ Device to Limit Excessive Displacement

For Mawo bridge, Limiting Excessive Displacement devices are not required to be installed because of following the reasons.

- Apply Type B Laminated Rubber Bearing
- Not Curved Bridge, perfectly straight bridge
- Not Skew Bridge, perfectly 90 degrees of crossing angle

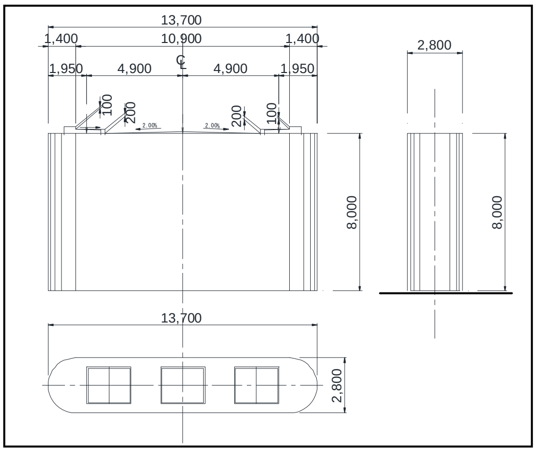
The verification of curved bridge and skew bridge is specified in the DPWH of BSDS

-Reference for using R factor design -

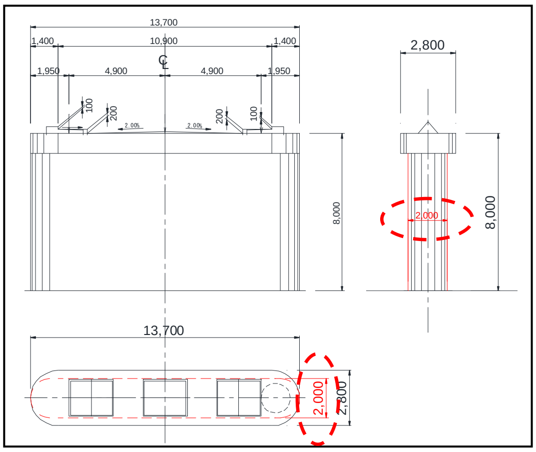
-Column Dimension

For column design of Mawo bridge, R factor (response modification factor) is not required to be installed because of the verification of limit for reinforcement of column and nominal flexural resistance (refer to 3.3.3 Column Design).

In this reference for using R factor design, the verification of Nominal Flexural Resistance of column is conducted for R factor 1.5, which is reconsidered for the column dimension and minimum reinforcement as followings drawings.



Column Dimension of Outline Design



Column Dimension of Outline Reference Design

-Design Load

Based on the BSDS Table 3.8.1-1, the design force of column is obtained as followings, which the R factor (response modification factor) 1.5 could be considered for the longitudinal direction (LL). In the transverse direction (TT), the R factor could not be considered due to the high nominal flexural resistance for this direction (Mn=926,942kNm >> Mt=161,724kNm, refer to Figure 3.3.3-1).

Summarize of Design Forces at Bottom of Column (R=1, as Table 3.3.2-1)

Longitudinal Direction at Section A-A (bottom of Column)

	DL		LL		EQ			SUM of LONGITUDINAL				
	N (kN)	φ	N (kN)	φ	H (kN)	M (kNm)	φ	N (kN)	Hl (kN)	Ml (kNm)	Ht (kN)	Mt (kNm)
P2(Nmax)	41,089	1.25	3,800	0.50	17,742	143,831	1.00	**53,270**	**17,742**	**143,831**	5,371	48,517
(Nmin)	41,089	0.90	3,800	0.50	17,742	143,831	1.00	**38,890**	**17,742**	**143,831**	5,371	48,517

Transverse Direction at Section A-A (bottom of column)

	DL		LL		EQ			SUM of TRANSVERSAL				
	N (kN)	φ	N (kN)	φ	H (kN)	M (kNm)	φ	N (kN)	Hl (kN)	Ml (kNm)	**Ht (kN)**	**Mt (kNm)**
P2(Nmax)	41,089	1.25	3,800	0.50	17,903	161,724	1.00	**53,270**	5,323	43,149	**17,903**	**161,724**
(Nmin)	41,089	0.90	3,800	0.50	17,903	161,724	1.00	**38,890**	5,323	43,149	**17,903**	**161,724**

Summarize of Design Forces at Bottom of Column (R=1.5 for LL, R=1.0 for TT)

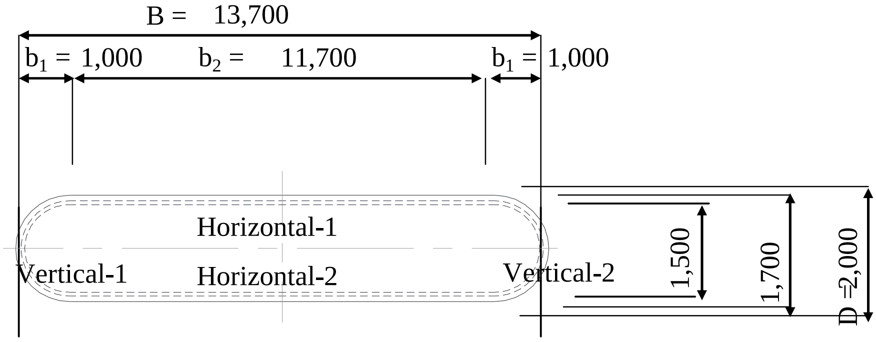
	R=	1.5				R=	1.0			
	LL					TT				
	N (kN)	Hl (kN)	**Ml (kNm)**	Ht (kN)	Mt (kNm)	N (kN)	Hl (kN)	**Ml (kNm)**	Ht (kN)	Mt (kNm)
P2 (Nmax)	53,270	17,742	**95,887**	5,371	48,517	53,270	5,323	**28,766**	17,903	161,724
(Nmin)	38,890	17,742	**95,887**	5,371	48,517	38,890	5,323	**28,766**	17,903	161,724

- Verification of Limit for Reinforcement

According to the AASHTO LRFD specifications, the verification of limit for reinforcement is obtained as followings, which can be reduced to the vertical bar D32@125 to D28@125 due to reduce to the area of column section.

1) Cross Section of Column

B =	13,700	mm
b_1 =	1,000	mm
b_2 =	11,700	mm
h =	2,000	mm



2) Longitudinal Reinforcement

		Num.	d_{ba}					
Horizontal-1	Layer-1	94	- D28	A_{st1} =	57,904	mm^2	@=	125
	Layer-2	94	- D28	A_{st2} =	57,904	mm^2	@=	125
n =	2.0	188.0		A_{st} =	115,808	mm^2		

		Num.	d_{ba}					
Horizontal-2	Layer-1	94.00	- D28	A_{st1} =	57,904	mm^2	@=	125
	Layer-2	94.00	- D28	A_{st2} =	57,904	mm^2	@=	125
n =	2.0	188.0		A_{st} =	115,808	mm^2		

		Num.	d_{ba}					
Vertical-1	Layer-1	20.00	- D28	A_{sc1} =	12,320	mm^2	@=	125
	Layer-2	16.00	- D28	A_{sc2} =	9,856	mm^2	@=	125
n =	2.0	36.0		A_{ct} =	22,176	mm^2		

		Num.	d_{ba}					
Vertical-2	Layer-1	20.00	- D28	A_{st1} =	12,320			
	Layer-2	16.00	- D28	A_{st2} =	9,856	mm^2	@=	125
n =	2.0	36.0		A_{st} =	22,176	mm^2		

n = 448 $\boxed{\Sigma A_{st} = 275,968\ mm^2}$

3) Transverse Reinforcement

4.0 - D25 A_{s1} = 2,027 mm^2 @= 150

Steel Strength	f_y =	414.0	MPa
Concrete Strength	f'_c =	28.0	MPa
Elastic Modulus	E_s =	200,000	MPa

4) Limits for Reinforcement (LRFD 5.7.4.2)

- Minimum area of longitudinal reinforcement

$A_s f_y / A_g f'_c > 0.135$ *(LRFD 5.7.4.2-3)*

where:

Area of longitudinal reinforcement steel		A_s =	275,968	mm^2
Yield strength of reinforcing bars		f_y =	414	MPa
Area of section	1	A_g =	26,540,000	mm^2
Compressive strength of concrete		f'_c =	28	MPa

Minimum area of longitudinal reinforcem	$A_sf_y / A_gf'_c$ =	0.154	>	0.135	[OK]

As / Ag > 0.01 (LRFD 5.10.11.4.1a)

Minimum area of longitudinal reinforcem	A_s / A_g =	0.0104	>	0.010	[OK]

- Maximum area of longitudinal reinforcement

As / Ag < 0.08 (LRFD 5.7.4.2-1)

As / Ag < 0.04 (LRFD 5.10.11.4.1a)

Maximum area of longitudinal reinforcen	A_s / A_g =	0.010	<	0.040	[OK]

- Minimum area of Transverse reinforcement (LRFD 5.8.2.5)

$V_c = 0.083b(\sqrt{f'c})b_v s/f_y$ *(LRFD 5.8.3.3-3)*

where:

· Minimum transverse reinforcement (LRFD 5.8.2.5)

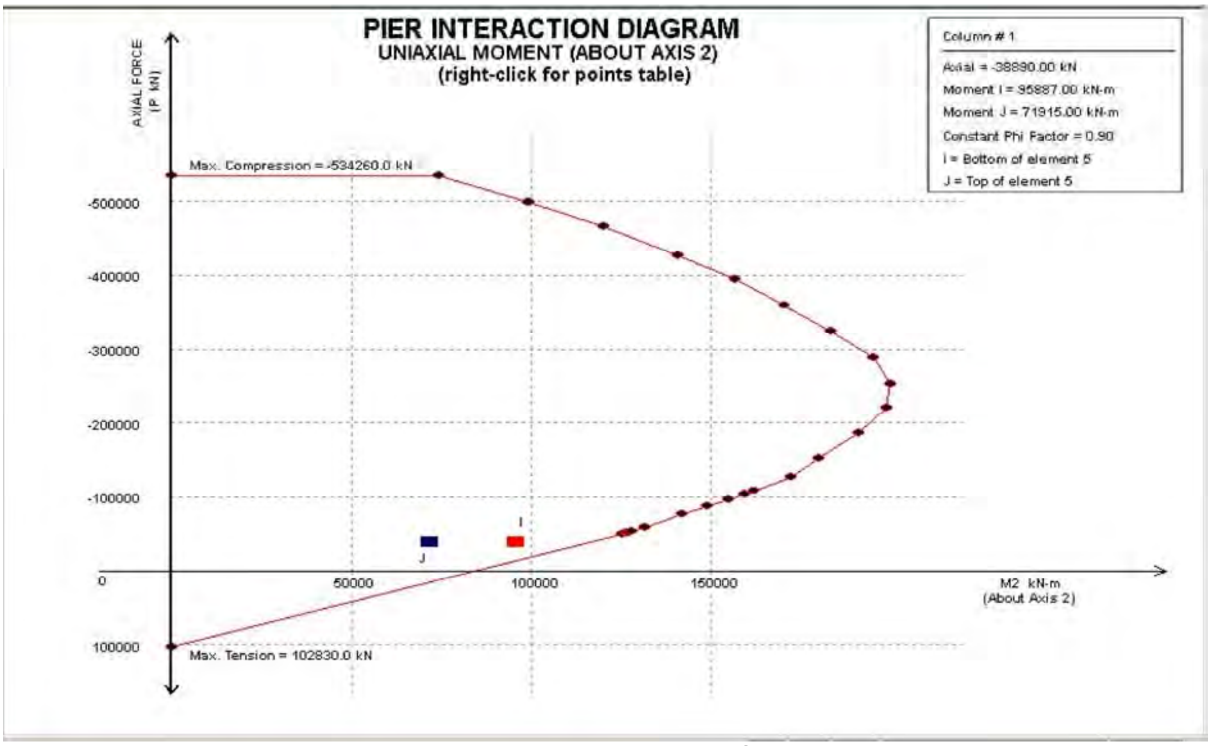
area of shear reinforcement within a distance	Av =	2027	mm^2
spacing of stirrups	*S* =	150	mm
effective web width taken as the minimum web (LRFD A5.8.2	b_v =	12,300	mm

(assumed as; bv=B-b 1))

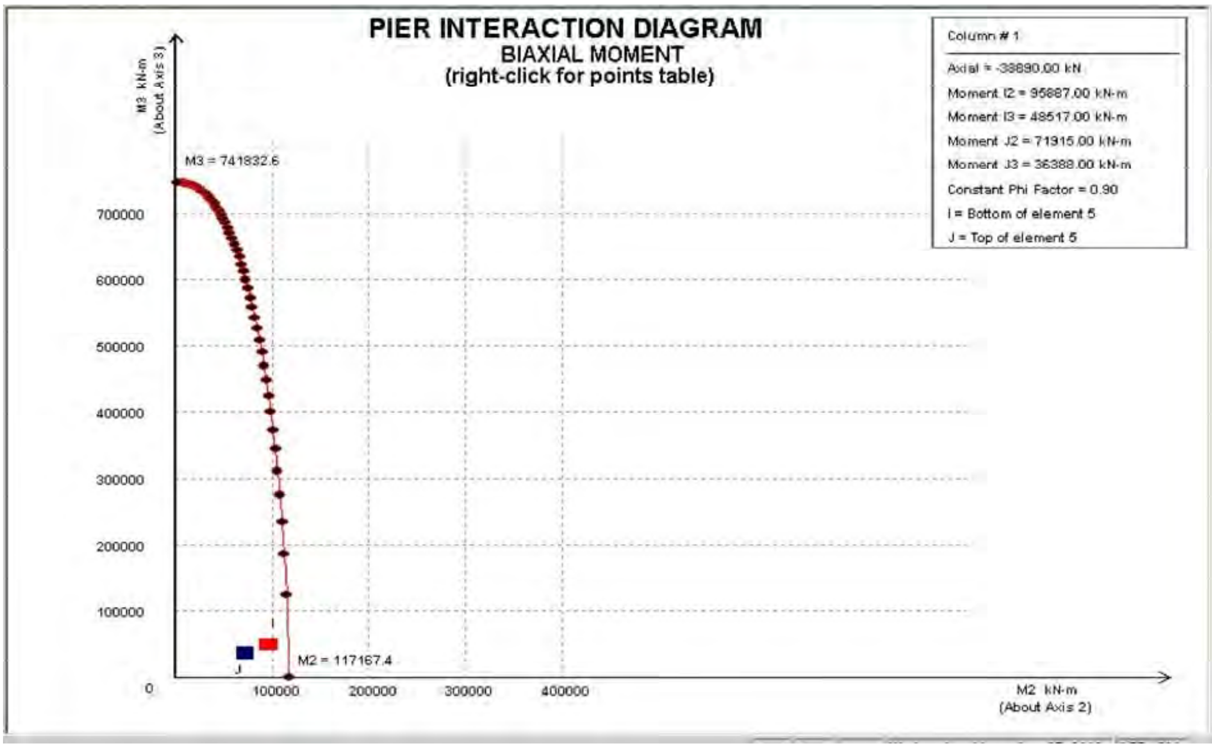
Min. transverse rein (mm^2)	A_{min}=	1,957	<	Av =	2,027	mm^2	[OK]

- Verification of Nominal Flexural Resistance of Column

Based on the FB-Multi Pier result, the verification of Nominal Flexural Resistance of column is obtained as followings.



M-N Interaction Diagram & Design Reaction for longitudinal Direction
(2-D28@125 LL, Nmin,)



M-M Interaction Diagram & Biaxial Moment for Design Axial Force
(at N=38,890kN, LL, Nmin, Mn=117,167kNm)

- Verification of Foundation Stability

Based on the results of seismic analysis and column design, summarize of design force of pile foundation is conducted for the following table, which are considered for outline design (Table 3.3.3-1, no hinge) and the column overstrength condition (refer to BSDS Appendix 4 (E), =1.3), which are not so different from no hinge design.
The verification of foundation stability shall be obtained as followings.

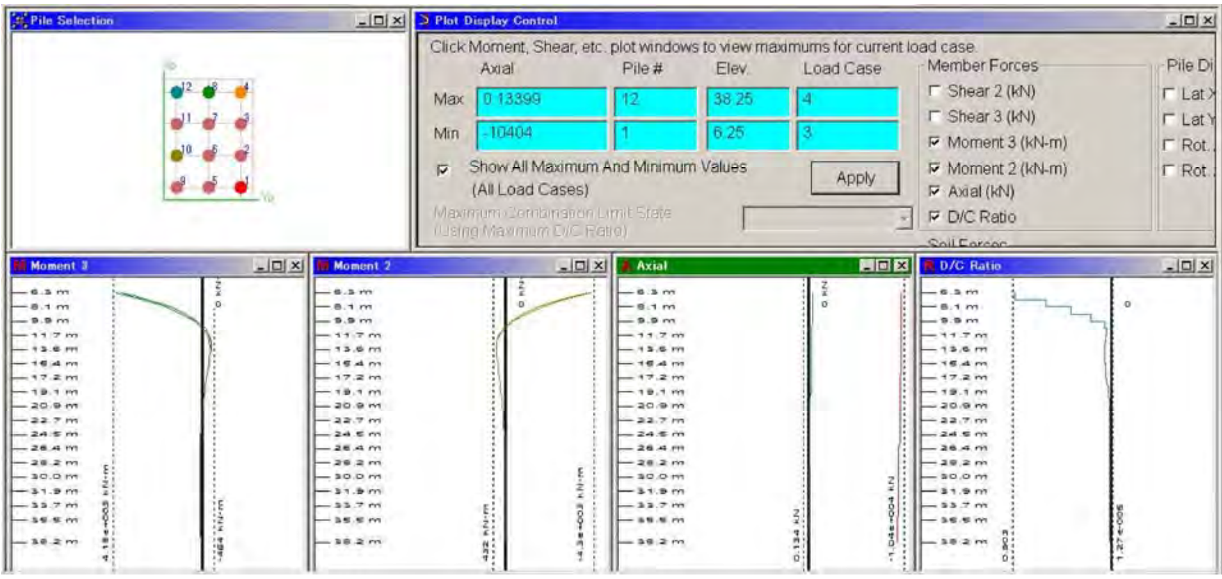
Summarize of Design Forces at Bottom of Pile Cap (no hinge, =1.0, as Table3.3.3-1)

LOAD CASE	N (kN)	Hl (kN)	Ml (kNm)	Ht (kN)	Mt (kNm)
1) Extreme- I LL(Nmax)	62,430	18,503	181,430	5,662	59,910
2) Extreme- I LL(Nmin)	45,480	18,503	181,430	5,662	59,910
3) Extreme- I TT(Nmax)	62,430	5,551	54,429	18,872	199,701
4) Extreme- I TT(Nmin)	45,480	5,551	54,429	18,872	199,701

Summarize of Design Forces at Bottom of Pile Cap (hinge, =1.3, LL)

<table>
<tr><th>LOAD CASE</th><th>N (kN)</th><th>Hl (kN)</th><th colspan="2">Ml (kNm)</th><th>Ht (kN)</th><th>Mt (kNm)</th></tr>
<tr><td>1) Extreme- I LL(Nmax)</td><td>62,430</td><td>18,503</td><td rowspan="2">1.3Mn</td><td>152,317</td><td>5,662</td><td>59,910</td></tr>
<tr><td>2) Extreme- I LL(Nmin)</td><td>45,480</td><td>18,503</td><td>152,317</td><td>5,662</td><td>59,910</td></tr>
<tr><td>3) Extreme- I TT(Nmax)</td><td>62,430</td><td>5,551</td><td rowspan="2">1.3Mn*0.3</td><td>45,695</td><td>18,872</td><td>199,701</td></tr>
<tr><td>4) Extreme- I TT(Nmin)</td><td>45,480</td><td>5,551</td><td>45,695</td><td>18,872</td><td>199,701</td></tr>
</table>

*Mn=117,167kNm (refer to previous page)



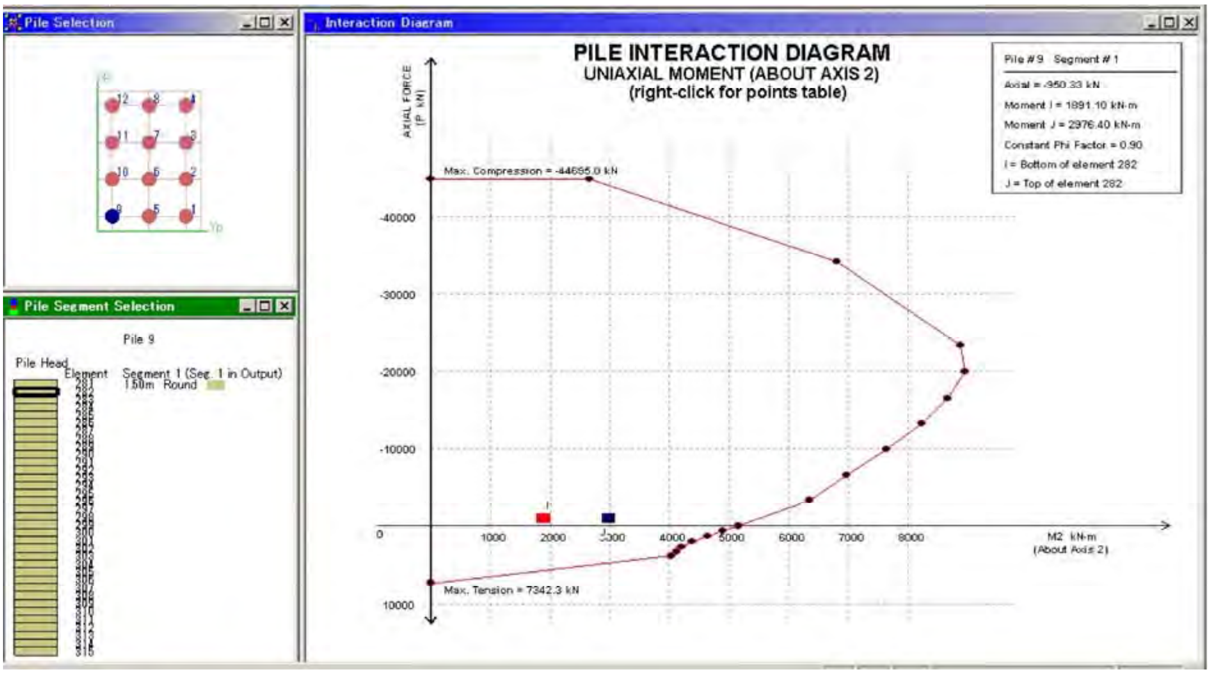
Summarize for calculation results for Longitudinal direction (Nmax, N=10,404kN)

Verification of Foundation Stability	Ra = 14,322 kN* >	Nmax = 10,404 kN	-ok-

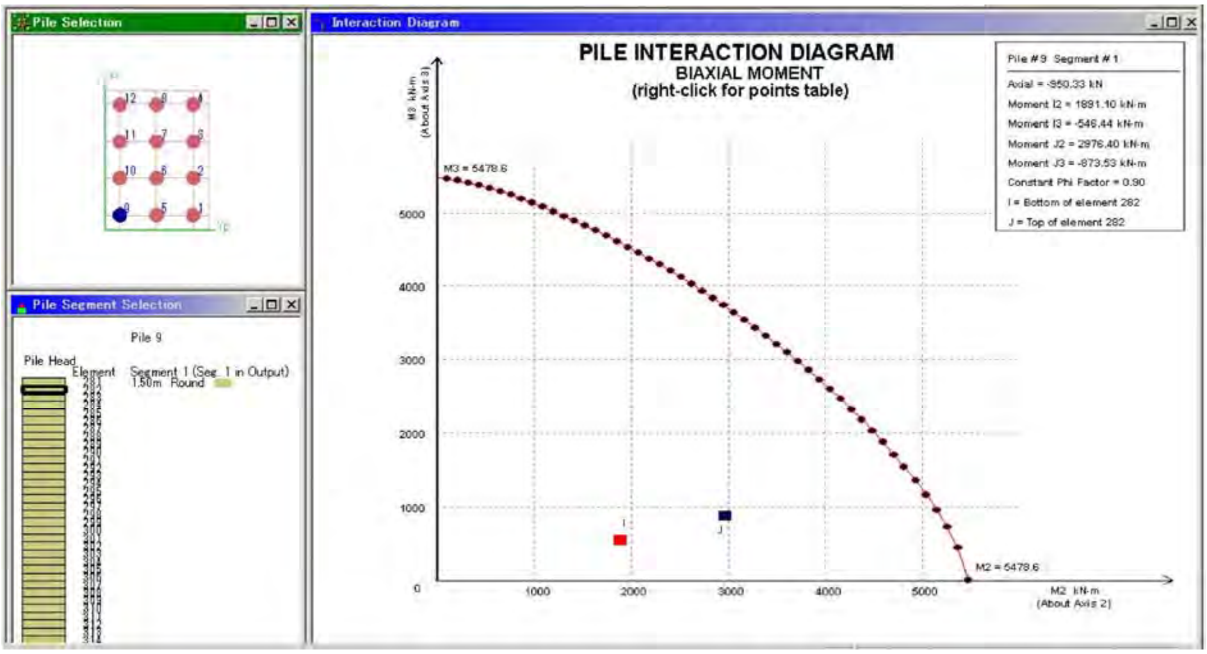
*Ra; refer to 3.3.3. (3) 3) "Verification of pile bearing capacity & foundation stability"

- Verification of Nominal Flexural Resistance of Pile

Based on the FB-Multi Pier result, the verification of Nominal Flexural Resistance of pile is obtained as followings, which the reinforcement is required to D28@118 as no hinge design.



M-N Interaction Diagram & Design Reaction for Longitudinal Direction at No9 pile (LL, Nmin, D25@118)



M-M Interaction Diagram & Biaxial Moment of No9 Pile

- Comparison between R=1 & R=1.5

Base on the outline design for no hinge (R=1) & hinge (R=1.5), summarize of design are conducted for the following drawings, which are not so different because of following the reason.

- required to the high minimum longitudinal reinforcement ratio of column
- not so different for the response modification factor (R =1.5) & overstrength condition (=1.3)