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Vice Presidency for Strategic Planning and Supervision

Guideline for Seismic Design of telecommunication systems

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Chapter 1

General

1-General

This guideline has been developed based on similar documents in the high seismicity countries together with local experiences and experiences from other countries outstanding in the field of earthquake engineering. In line with gaining from other countries' experiences; it has been tried to pay attention to the localization issue and present subjects more simple and practical.

1-1-Objective

The objective of this guideline is to secure public safety and prevent serious damage to communication systems due to earthquake

- Vulnerability against earthquake is very critical.
- the objective of this guideline is to secure acceptable safety regarding to rational risk on the basis of economic conditions and the nature of earthquake hazard and vulnerability of natural gas systems.
- The aim of this guideline is to no serious and interfering public safety damage occur by observance of its content.

1-2-Scope

The scopes of this guideline are installations of communication system including transmission equipments,cable line conduits, shared channel (with other arteries), cable, manhole and internal equipments.

- Iran 2800 code and Iran's National Building Regulations are used for seismic design of the structures of this system
- Subjects regarding to National Building Regulations or reliable internal codes can be used for seismic design for foundation of equipment together with results extracted from seismic design of related instrument from this guideline.
- The most important components of aerial and buried telecommunication located in the outside building is shown in figure 1-1.
- There are both aerial and buried telecommunication cables in telecom network. Seismic design of cables of telecommunication like aerial and buried cables of power network.
- Telecom foundation also like aerial foundation in the seismic design of municipal power system.

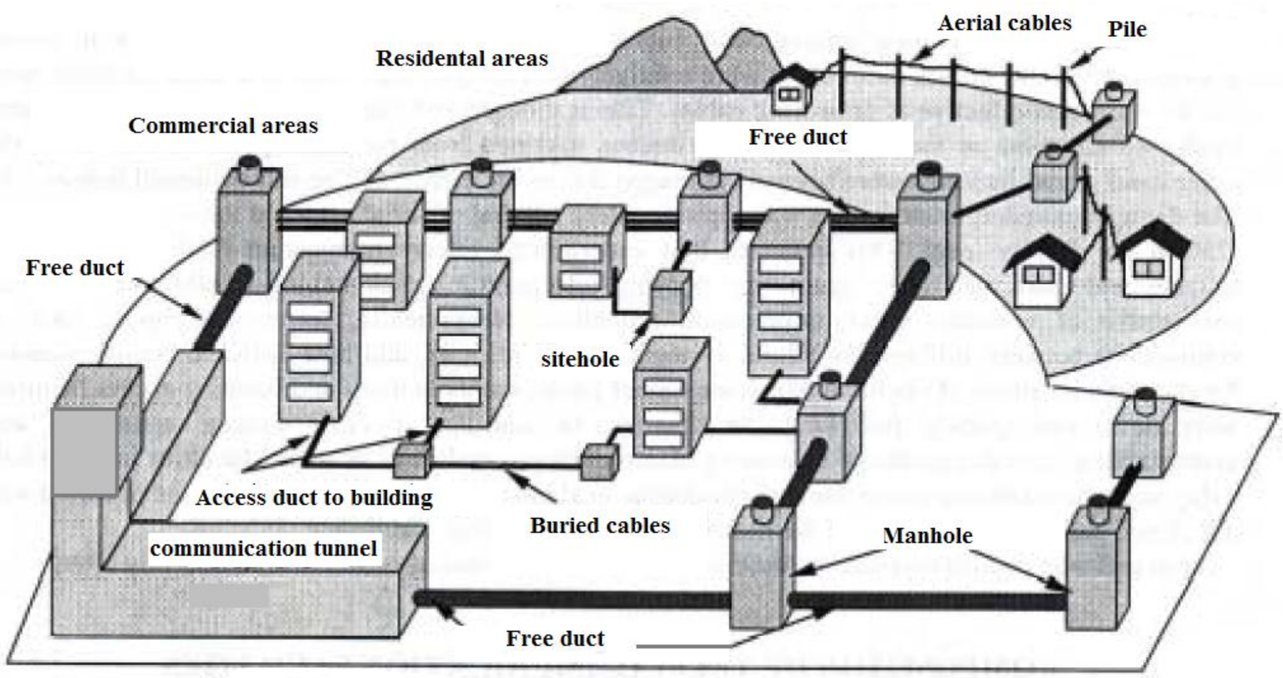


Figure 1-1- overview of the most important telecommunication component in outside

1-2-1-Orgaization of this guideline

This guideline was organized with above mentioned objectives and scope into following chapters:

Chapter 1: general

Chapter 2: principles

Chapter 3: seismic loading

Chapter 4: methods of seismic design and safety control

Chapter 5: seismic design and safety control of aerial equipment of communication system

Chapter 6: seismic design and safety control of communication line conduits

Chapter 7: seismic design and safety control of communication network manhole

Chapter 8: seismic design and safety control of internal equipment bracing

1-2-2-Application notes

Since this document is the first narrative of the respective guideline in the country, like previous guidelines and regulation, the presence of ambiguity and impediments is a normal issue. For lessen these difficulties and remove them as soon as possible, it is worthwhile to consider following notes:

1-It has been tried to remove any contradictions between regulations of this guideline with Iran 2800 code.

2-In the case of scarcity of information about loading intended gas distribution system in this guideline, topic six of National Building Regulations can be implemented.

3- For designing concrete components in this guideline, especially concrete material characteristics, topic 9 of National Building Regulations can be implemented.

4-Topic 10 of National Building Regulations is complementary for obviate any shortcomings regarding steel components of its telecommunication facilities.

5-It is expected from all users of this guideline to send their corrections and recommendations for its better compliance and easier implementation in the country to be used for developer in the future versions.

1-3-Related codes and regulations

1-3-1-Normative references

Various standards, codes, regulations, guidelines and manuals were used in the development of this guideline. The most important of them are as following:

Iran 2800 Code: Iranian Code of Practice for Seismic Resistant Design of Buildings, Standard No. 2800, Building and Housing Research Center, 2005

Euro code 8: Design rules for the earthquake-resistant structures, Part 4: silos, storage tanks and pipeline systems, European Committee for Standardization, 2006

ASCE7-05: minimum load for designing building and other structure.

BCJ1997: JEAC committee for transmission line: aerial transmission line code, 2000(Japan)

Technical journal of telegraph and telephone company of Japan: technology of space structure (1-3), August, September and October 2007(NTT).

IBC2006: international building code, USA, 2006.

UBC97: uniform building code, united states of America,1997.

Building center of Japan: seismic design and construction of building equipment guideline, 1997.

JEAC committee for distribution line: power distribution code, 1999 (Japan).

JEAC committee for transmission line: underground transmission code, 2000(Japan).

Power company of Tokyo: seismic design of high voltage transmission Tower and its foundation, march 1984 (UHV).

JEAC committee for thermal power plant: seismic design of thermal power plant, 2004 (in Japan) JEAC 3605-2004

Japan Gas Association: seismic design of high pressure gas pipeline for liquefaction, JGA-207-01, 2001.

Japan Gas Association: Recommended steps for LNG containers above the ground, August 2002.

High Pressure Gas Safety Institute of Japan (KHK): Seismic Design Code for High Pressure Gas Facilities, 2006

Architecture Institute of Japan: Manual for seismic design of crane, May 1989.

Building Center of Japan (BCJ): manual for structural design and building stacks, 1982 (Stack-82)

Japan Road Association: Specifications for Highway Bridges, Part V, Seismic Design, 2002

1-3-2-Code Abbreviations

Abbreviation	Full Name
ABA	Iran’s National Building codes- design and construction of concrete buildings
	Iran’s National Building Regulations- part 10, design and construction of steel buildings
AIJ1996	Recommendations for designing storage tanks and their Supports, Architecture Institute of Japan, 1996
ALA2005	Seismic Guidelines for water Pipelines, ALA, 2005
ANGTS	Complementary applicable technical information of Alaska State, Alaska Natural Gas Transportation System, 2004
API 5L	API specifications for Line Pipe L5, Pipeline specification, API, 2004
API620	API 620 standard, Design and manufacture, Design and Construction of Large, Welded, Low-Pressure Storage Tanks, API, 2005
API650	API 650 Welded Steel Tanks for Oil Storage, API, 2005
ASCE7	Minimum Design Loads for Buildings and Other Structures, ASCE, 2006
ASCE1984	Seismic Design Guidelines for oil and Gas Pipeline Systems, ASCE, 2006
ASCE1985	Design of Structures to Resist Nuclear Weapons Effects, Manual 42, ASCE, 1985
ASCE1997	Guide lines for seismic evaluation and design of petrochemical facilities, ASCE, 1997
ASME B31	ASME B31 Code for Pressure Piping, ASME B31, 2004, AWWAD100-96
AWWA96	AWWAD100-96
BCJ1997	Specifications of seismic design for building components, Building Center of Japan, 1997
BS EN1998-1	Euro code 8: Design of structures for earthquake resistance. Part 1,General rules, seismic actions and rules for buildings, European Committee for Standardization, 2004
BS EN1998-4	Euro code 8: Design rules for the earthquake-resistant structures, Part 4: silos, storage tanks and pipeline systems, European Committee for Standardization, 2006
BS EN1998-5	Euro code 8: Design of structures for earthquake resistance – Part 5: Foundations, retaining structures and geotechnical aspects
IBC2006	International Building Code, USA, 2006
INBC10	Iranian National Code, Part 10, design and construction of steel structures
IPS-X-XX	Iranian Petroleum Standards
Iran2800	Iran 2800 Code: Iranian Code of Practice for Seismic Resistant Design of Buildings, Standard No. 2800, Building and Housing Research Center, 2005
Iran519	Iran 519 Code, loading of Buildings
JIS B8501	Japanese Industrial Standard, JIS B8501, Welded Steel tanks for oil storage, 2001
JGA1982	Recommended methods for earthquake-resistant design for high and medium pressure gas pipelines, Japan Gas Association, 1982
JGA2000	Recommended methods for earthquake-resistant design for high pressure gas pipelines, Japan Gas Association, 2000
JGA2001	Recommended methods for earthquake-resistant design for high pressure gas pipelines in the areas with potential of liquefaction, , Japan Gas Association, 2001
JRA74	Technical Standard for Oil Pipelines, Japan Road Association, 1974
JRA V	Design Specifications for Highway Bridges, Part V: Seismic Design, Japan Road Association, 1996
JSWA2006	Seismic Design and Construction Guidelines for Sewage Installations, Japan Sewage Works Association, 2006
JWWA1997	Seismic Design and Construction Guidelines for Water Supply Facilities, Japan Water

Abbreviation	Full Name
	Works Association, 1997
KHK	Technical Seismic Design Code for High Pressure Gas Facilities, High Pressure Gas Safety Institute of Japan (KHK), 2000
MCA22	MCA Safety Manual 22
MIAC No.30	Seismic Design Appendix for Fire Defense, Directive No. 3, Ministries of Communications and Internal Affairs, 2005
NIOEC-SP-XX-XX	NIOEC Specifications
NZ1981	Seismic Design of Petrochemical Refinery, Ministry of Energy, New Zealand, 1981
NZ1986	Seismic Design of Storage Containers, Ministry of Energy, Recommendations of Study Group of New Zealand National Association for Earthquake Engineering, New Zealand, 1986
WSP064	Design Standard for Water Pipe Bridge, WSP 064-2007, Metallic Water Pipe Association
UBC97	Uniform Building Code, USA, 1997

Chapter 2

Principles

2-Principles

Principles regarding earthquake design, design methods (safety control) and anticipated performances are presented summarily in this chapter. The most important of these principles are:

1- Two risk level

with attention to two acceleration and velocity spectrum compatible with Iran's condition and in accordance with regulations of Standard No. 2800

2- Consideration of earthquake load in two types including inertia force due to the acceleration effect in mass that exerted in to gravity mass center or centers and the force due to earth displacements and its application through interaction of soil and buried structure to the body in the contact with soil.

3- Using two method for design and safety control including allowable stress in the elastic limit of material behavior for risk level-1(operational earthquake) and ductility method in non-elastic of material behavior for risk level-2(design earthquake).

4- Using two level of performance as damage limit and ultimate limit which unceasing utilization and minimum cease are secured in the former and latter, respectively. With regard of the behavior of structural components, that remains undamaged in the elastic limit, in the former, but in the latter, passes from the yield boundary but bounds to the certain allowable limit of plastic deformation. This allowable limit after yield-allowable ductility- is determined by this guideline according to experimentations or experiences.

They are described summarily in the following.

2-1-Design earthquake

2-1-1-Risk levels of earthquake and their return period

1-Two risk level mentioned in the following must be considered for seismic design of components of telecommunication network system:

Risk level 1: Maximum Operational Earthquake (MOE) (occurrence probability 50% during 50 years of assumed useful service)

Risk level 2: Maximum Considerable Earthquake (MCE) (occurrence probability 10% during 50 years of assumed useful service)

1-In this guideline, useful service life of lifelines is considered relatively around 50 years. Maximum operational earthquake may be occurred once or twice during the service of telecom facilities. Unacceptable failure modes during operation of facilities are confined to seismic risk level and operation of communication system continues reliably. In this risk level, occurrence probability of 50% during 50 years is in accordance with return period of 72 years.

2- Maximum earthquake of design is an earthquake with lower occurrence probability and longer return period than to earthquake of MOE. The behavior of gas system components in the risk level 2 is in the ultimate mode and the whole system, even if a member is damaged, must maintain its stability. The occurrence probability of 10% for earthquake with higher magnitude during 50 years is in accordance with return period of 475 years.

- With regard of risk management, overrun probability of 10% has versatile and suitable application in the economic term with consideration of requisite safety.

3-For estimation of Permanent Ground Deformation (PGD) due to fault displacement (faulting), liquefaction and landslide, earthquake magnitude M and fault distance R are required. M, R and other related seismic parameters are obtained from analytic or experimental relations resulted from designer desired region risk analysis, especially with regard to its seismicity records.

2-1-2-Seismic design spectra

1-Response spectra of seismic design for telecommunication facilities must be computed according to natural period and decay characteristics of structural systems. Also, load due to earthquake must be computed by means of these spectra.
2-Spectrum dynamic analyses for seismic safety control of structural systems must be done by mixing modal spectrum characteristics.
3-One of two following response spectra must be used for designing telecommunication facilities
A-Acceleration response spectrum for computation of inertia force due to the mass of above ground components
B-Velocity response spectrum for computation of interactional force due to the displacement of soil on the body of buried components

1-Spectrum Analysis

1-1-In this guideline, elastic response spectrum is used for decay of 5%

1-2-response spectrum for seismic design is obtained from following methods:

A-Site-specific spectra

- Site-specific spectra are computed according to seismic activities, active faults and geomorphologic conditions.
- Regulations of standard no. 2800 are used for obtaining site-specific spectra.
- Moreover, velocity spectrum must be extracted for buried structures.

B-Probabilistic or deterministic spectra based on record of strong earthquakes

- Probabilistic methods have more engineering applications.
- Spectra from these methods usually have lower values than to their equivalents from deterministic methods.

Generally in determination of design spectra, earthquake occurrence probabilities are considered on the basis of ground strong movements. Deterministic spectra are often used for conservative design and crisis management.

2- Dynamic analysis

Analysis of dynamic response is a method for seismic safety control of structure, especially structures with complex behavior under earthquake.

These analyses compared with the Pseudo-static methods are sensitive to dynamic parameter, expensive and time-consuming and only used when there is difficulty and uncertainty in application of response spectra.

3-Acceleration and velocity response spectrum

- acceleration response spectrum is used for above-ground structures(with considerable mass).
- Also, these spectra are convenient for systems with several degrees of freedom with application of modal analysis method.
- Aacceleration spectrum available in the valid and current version of standard no. 2800 is used for computations relied on acceleration spectrum.

- Velocity response spectra are used for underground structures such as pipelines, tunnels, shared conduit, shaft and also manhole underground reservoirs which their action are controlled with seismic behaviors of surrounding soils.
- Earthquake loading of such structures are done on the basis of displacement response in which, soil deformation in the location of buried structures are computed with usage of velocity response spectrum firstly and then interaction between ground and buried structures is determined by static methods.
- Velocity and acceleration response spectra must be compatible with seismic design of telecom facilities.
- In the first version of this guideline, a velocity response is proposed that is nearly compatible with standard no. 2800.
- For next versions it is required to develop a series of velocity spectrum (as acceleration spectrum in standard no. 2800).

2-1-3-Distribution of seismic intensity in stories

Seismic design intensity of stories is used for communication system components located on other facilities or in the upper stories. in stories simplified coefficient distribution (Ai) use for distribution of seismic intensity(KH), that is given by equation (2-1). This coefficient multiplied at KH in every stories.

$$A_i = 1 \Big/ \sqrt{\frac{H - x}{H}} \qquad (2\text{-}1)$$

H: Total height of stories

x: the height of stories above the stories of i

Seismic intensity for equipment located on the height over equipment or another structure is a function of seismic intensity on their position. For equipment inside structures, equipment behavior in earthquake will depend on story which is based on it.

2-1-4-Vertical seismic intensity of design

Seismic inputs of vertical direction must be considered for equipment which their behavior is sensitive to the vertical component of earthquake such as tower arm, legs, aerial equipment, installation, equipment by control vibration system and braced equipment. Vertical seismic intensity of design Kv is given by equation (2-3):

$$K_V = \frac{1}{2} K_H \qquad (2\text{-}2)$$

Which, KH is seismic intensity of the design in horizontal direction.

In this guideline for each of components that is presented the chapter 5 and next chapters, their horizontal and vertical earthquakes are computed, appropriately. In almost all of the cases, vertical earthquake is accounted as half of horizontal earthquake.

2-2-Seismic safety control (design methods)

2-2-1-Seismic safety control

Safety of designed equipment must be controlled via following methods:

1-Allowable stress design method which must be applied on the risk level 1
2-Ductile design method which compare existing ductile ratio of structure with allowable ductile ratio for risk level 2.
Note:
-In the cases which equipment are of very high importance or certain complexity exists in seismic behavior, convenient dynamic methods are used for control of above mentioned methods according to the design engineer.

1-For serviceability limit state, created stress in the structural component compare with allowable stress in elastic range.Generally, allowable stress design method is used in risk level 1 for operation earthquake
2-The basis of the ductile design method is to give more allowance to structure to absorb more energy (after yield point of material) for more strong earthquake with high acceleration (and velocity) so their components can absorb more strains. Computed ductility coefficient is controlled with formulae of this guideline with appropriate allowable ductility coefficient.
3-2-Anticipated functions in this guideline

Two functions are considered in this guideline for communication system components for given risk levels:

- serviceability function (until before material yield)
  Risk level 1: designed components must not damage communication system function effectively and their function must be continued unceasingly.
- Minimum serviceability of function (after material yield)
  Risk level 2: designed components may inflict drastic physical damage function but without any effect on lives, environment and sustainability of gas distribution system. Inflicted damage must be removable as soon as possible and faulted function must be rehabilitated.

A-In risk level 1

- structural members must not be impaired any physical damage that interrupt system usage.
- This level is called “limit state or mode”.
- In this mode, each building member of system must be in the elastic extent of stress-strain relations and not be reached to yield limit.

B-In risk level 2

- members of structural systems can be physically damaged parochially but systematic and structural sustainability must not be destroyed.
- This level is called “ultimate state or mode”.
- In this mode, non-elastic deformations (after material yield) may be occurred.

Chapter 3

Seismic Loading

3-1- Types of Loads

Calculation of loads in communication system are as follows:

- Dead weight from the equipment and its accessories
- Weight from some materials inside the equipments
- Content's internal pressure of some equipment
- Soil's pressure on buried components
- Thermal pressure
- Earthquake's pressures
- Wind's pressure

For types of loads regarding the communication facility's components, the following considerations are required:

- The wind's pressure is not effective on buried components.
- Unlike buildings, the structures in communication facilities do not have any live human loads.
- Tower, legs and communication cables' resistance against wind force is more than inertia l force caused by earthquake's vibration. Therefore, is only controlled by earthquake.
- Geotechnical hazard caused by earthquake on all communication equipment such as tower, leg, building and so can have divesting effects.

3-2- Weights calculations

To calculate the materials unit weight and different loads, sixth chapter of national buildings regulations should be used.

3-3- Loads combination

In this guideline, regarding each equipment, the implemented loads and their combinations are presented from chapter 5 to 8.If it doesn't present any combination, load factor are taken one.

3-4- Types of equipments by their location

The communication facilities are generally located as follows:

- Aerial equipments
- Inside building equipments
- Underground and buried equipments

3-5- Seismic loads calculation methods

Earthquake-imposed loads on communication facilities are implemented as follows:

1- The inertial force caused by the equipments' mass which their movement is not constrained in soil. This force, generally, is created and calculated in inside building equipment and aerial and on-ground structures.

2- The force from the bed soil's displacement which is imposed on the buried structures. In this case, the soil displacement is multiplied to the spring constant between soil and building and imposes its force on the equipment.

3-6-The effects of earthquake on gas supply facilities

The effects of earthquakes on gas supply facilities could be divided into two effects and then perform the seismic loading calculations from each one:

1- The dynamic effect of earthquake caused by soil vibrations (seismic waves propagation in soil) which results in the three following responses:
   a. Acceleration
   b. Velocity
   c. Displacement

2- The static effect or the so-called geotechnical hazards which results in permanent displacements in soil, including:
   a. Liquefaction (and the lateral spread)
   b. Earthquake
   c. Fault

The method of imposing seismic loads caused by the above-mentioned effects, based on various methods is presented in the guideline for loading methods and vital vessels seismic analysis.

3-7-The method of imposing earthquake effects on communication equipments

1- In order to calculate the imposed load on inside building equipment and aerial and on-ground components, the inertia force caused by the earthquake acceleration's effect on the equipment mass must be calculated. In this method, the acceleration spectrum is used in conformity with the regulations of the 2800 standard.

- Modified pseudo-static method

The pseudo-static method is used in obtaining displacement and stress in a structure with high rigidity and imposing the seismic load as an equivalent static load. The equivalent static load is calculated as the product of multiplying the earthquake coefficient in structure's mass.

In the modified pseudo-static method, natural period, structure's damping, and the inertia force from earthquake are considered and the modified coefficient is used in comparison with the pseudo-static method.

- Dynamic analysis method (spectral or time-history)

In addition to these methods, the spectral or time-history dynamic analysis method is applied for controlling the simple static methods, better understanding the components' seismic behavior, and ensuring the performed design. In such methods, the results' reliability depends on the input accelerations properness and the selected coefficients for damping.

2- In order to calculate the inertia force caused by the effect of acceleration, the "pseudo-static method" is used. In cases where the equipment's period is long and/or complicated (when the structure's first vibration mode is not predominant), the "modified pseudo-static method" is applied.

- In order to calculate the forced from the earth displacement on buried structures, the "displacement response method" is used. In this method, after calculating the earth's displacement in desired points, the imposed force on the structure is calculated by determining the spring constant of soil surrounding the structure.

-In the displacement response method, calculating the soil's strain surrounding the structure, the buried structure's strain could be calculated from the soil's strain, considering the adhesion between the buried structure and its surrounding soil.

3-7-1-importance factor

The structure's importance factor is denoted by β_1 based on its importance and is determined using table 3-1.

Table 3-1- Importance factor, β_1

Importance category	Very high	High	Medium	Low
β_1	1.4	1.2	1	0.8

The need for immediate occupancy of under design components and the necessity for their post-earthquake safety play a significant role in raising the importance factor of that component in communication system. The definition for different categories are presented in table 2.3.

In cases in which two importance levels are presumed, the employer's judgment determines the final importance level.

Table 3-2- Definitions for different importance categories

Importance	Description
Very high	Components which their destruction causes vast casualties and financial losses and damages equipments and environment. As well as components which halting their performance would lead to secondary lives, financial and environmental damages.
High	Components which their destruction would lead to gas supply cut or lives, financial, and environmental losses and damages.
Medium	Components which their destruction would lead to interruptions in gas supply.
Low	Components which their destruction do not have any considerable effect on gas supply system and would not lead to casualty and financial and environmental damages.

3-7-2- Design base acceleration ratio

Design base acceleration ratio, β_2 , can be defined from table 3-3.

Table 3-3-Design Base Acceleration Ratio (β_2)

Seismicity Status	1 (Very High)	2 (High)	3 (Medium)	4 (Low)
β_2	0.35	0.30	0.25	0.20

3-7-3- Amplification factor from the bedrock to ground level due to soil type and zone factor

Magnitude of the earthquake force is related to soil layers amplification of the location. Amplification factor based on soil layers is labeled as β_3.

Table 3-4 shows amplification factor for all types of grounds.

Table3-4- site magnification factor (β_3)

Design base acceleration ratio \ Soil Type	Type 1	Type 2	Type 3	Type 4
Low	1.5	1.5	1.75	2.25
Medium	1.5	1.5	1.75	2.25
High	1.5	1.5	1.75	1.75
Very High	1.5	1.5	1.75	1.75

The design seismic force that is calculated from pseudo static method calculates such as follow.

3-7-4-calculating seismic factor

3-7-4-1-modified pseudo-static method

Design horizontal earthquake factor K_{SH} could be extracted from equation (3-1):

$$K_{SH} = \beta_4 K_H \tag{3-1}$$

In which:

K_{SH} : Design horizontal seismic coefficient. If K_{SH} is calculated less than 0.2, it is considered 0.2.

K_H : Horizontal seismic intensity at surface of earth

$$K_H = 0.3 \cdot \beta_0 . \beta_1 \cdot \beta_2 \cdot \beta_3 \tag{3-2}$$

β_4 : Horizontal response magnification factor, value of this factor is function of structure height from the earth. 1.0 for height less than or equal to 16 meters. 0.0125h+0.8 for height between 16 meters and 35 meters.

β_0 : Earthquake parameter level. For hazard level-1: 0.5 and For hazard level-2: 1

h = structure height (m) is calculated from surface of earth.

3-7-4-2- horizontal earthquake load

Design horizontal earthquake load (static load equivalent) F_{SH}, could be extracted from equation (3-3)

$$F_{SH} = K_{SH} W_H \tag{3-3}$$

In which:

F_H : Design horizontal earthquake load (N)

K_{SH} : Design horizontal seismic coefficient

W_H : Structure weight + live and dead load (N)

3-7-5- modified pseudo-static method

For structures long periods than those in the section 3-1-5, modified pseudo-static method would be used.
After calculating the seismic coefficient from modified pseudo-static method, to determine the vertical or horizontal earthquake load, we should multiple it to structure (equipment) weight.

3-7-5-1- modified pseudo-static factor

Modified seismic coefficient can be gained from(3-4):

$$K_{MH} = \beta_5 K_H \tag{3-4}$$

$$K_{MV} = \beta_6 K_V \tag{3-5}$$

K_{MH} : Horizontal modified seismic coefficient.

β_5 : Amplification factor, extracted from equation (3-7)

$$\beta_5 = \beta_{5n} C_h$$

β_{5n} : Standard response magnification factor shown in Fig(3-1)

C_h : Modification factor depending on structural damping shown in Fig (3-2)

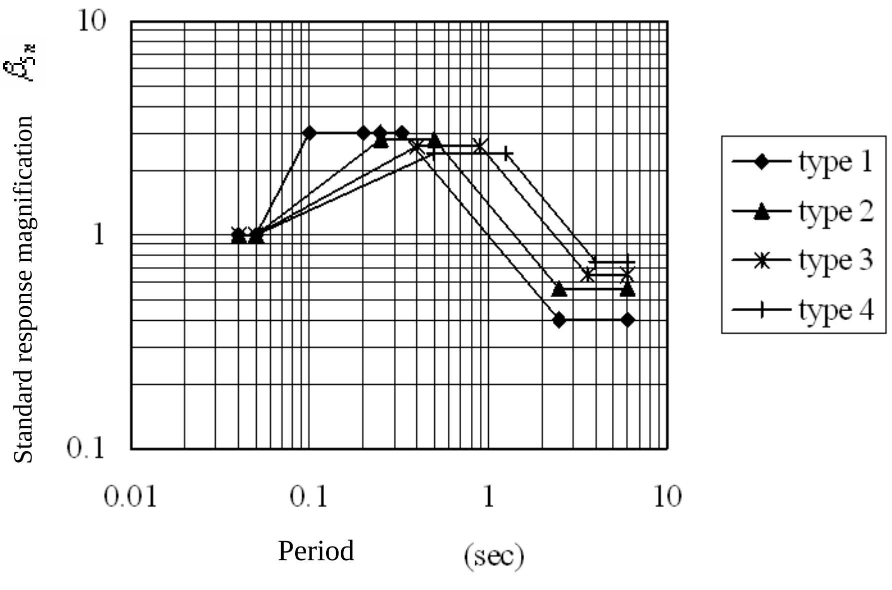


Figure 3-1 β_5 **, Standard response magnification**

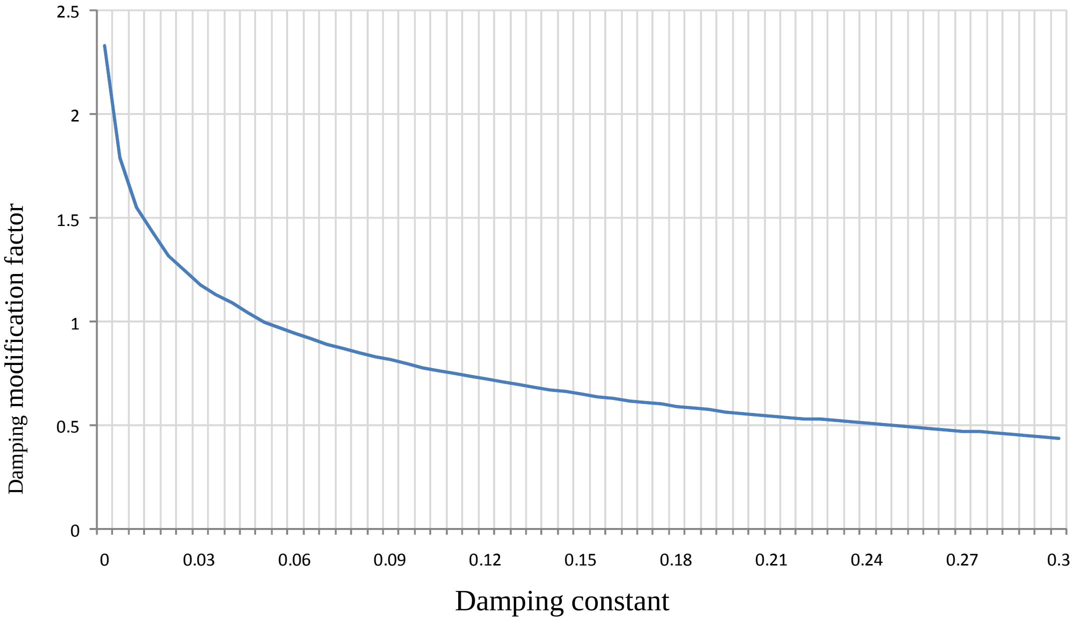


Figure 3-2 C_h, damping modification factor

3-7-5-2- Vertical modified seismic factor (K_{MV})

K_{MV} is calculated from (3-6):

$$K_{MV} = K_{MH}/2 \tag{3-6}$$

For equipment in term of importance, is placed in lower importance range, vertical acceleration isn't require.

3-7-5-3- Modified Earthquake Load

Modified earthquake load can be reached from multiplication of structure weight and modified earthquake coefficient from equations 3-8 and 3-9:

$$F_{MH} = K_{MH} \times W_H \tag{3-7}$$

$$F_{MV} = K_{MV} \times W_H \tag{3-8}$$

K_{MH} : Modified horizontal seismic coefficient

K_{MV} : Modified vertical seismic coefficient

F_{MV}, F_{MH} : Modified horizontal and vertical earthquake load

W_H : Structure weight + live and dead load (N)

3-7-6- Dynamic Methods

For complex structures or with higher importance, dynamic analysis is performed by following methods.

3-7-6-1- Spectrum Method

Horizontal response acceleration for each mode $A_H(T)$ can be extracted from equation 3-9:

$$A_H(T) = \beta_5 \cdot \alpha_H \quad (3\text{-}9)$$

$A_H(T)$: Horizontal response acceleration in natural period T (cm/s^2)

β_5: Horizontal response magnification factor (β_5 =1.5 for periods less than 0.3 sec and β_5 =0.75 for periods more than 0.3 sec)

α_H : Horizontal acceleration on the ground level (cm/s^2) obtained from equation 3-10:

$$\alpha_H = 700 \cdot \beta_1 \cdot \beta_2 \cdot \beta_3 \quad (3\text{-}10)$$

Also vertical response acceleration for each mode $A_V(T)$ can be extracted from equation 3-11:

$$A_V(T) = \beta_6 \cdot \alpha_V \quad (3\text{-}11)$$

$A_V(T)$: Vertical response acceleration in natural period T (cm/s^2)

β_6: Vertical response magnification factor (β_6 =1.5 for domain base towers and β_6 =2 for other sizes)

α_V : Horizontal acceleration on the ground level (cm/s^2) obtained from equation 3-12:

$$\alpha_V = 350 \cdot \beta_1 \cdot \beta_2 \cdot \beta_3 \quad (3\text{-}12)$$

Combination of mode results and other analyze spectrum cases are accomplished according to the criteria of standard 2800.

3-7-6-2- Time history analysis method

In Time history analysis method, an appropriate accelerometer should be chosen then its maximum horizontal acceleration based on the location can be extracted from one of following methods:

1- If records of the ground level are available:

$$\alpha'_{HT} = 700 \cdot \beta_1 \cdot \beta_2 \quad (3\text{-}13)$$

α'_{HT} : Maximum horizontal acceleration in the ground level (cm/s^2)

2- If records of the ground surface are available

$$\alpha'_H = \alpha_H = 700 \cdot \beta_1 \cdot \beta_2 \cdot \beta_3 \quad (3\text{-}14)$$

α_H : Horizontal acceleration in the ground level in spectrum analysis (cm/s^2)

α'_H : Maximum horizontal acceleration in the ground level in the Time history analysis

3-7-6-3-Displacement response method (for buried structures)

This method is devised based on beam on an elastic bed theory.
In this method, using the earthquake velocity response spectrum and considering the first mode of shear vibration of soil, the displacement is calculated and according to the soil's resilience is transformed to the effective force on the structure.

3-8-Loading caused by earthquake's geotechnical hazards on equipments

In addition to the earth's vibrations during earthquake, the equipments must be safe against geotechnical hazards caused by earthquake. The most important of which are liquefaction (and lateral spreading), landslide, and faulting.

3-8-1-Liquefaction

Even though there is low potential for liquefaction in Iran, in seaside, riversides, and in regions with fine-grained sandy texture along with high underground water level, this hazard threatens different equipments, especially the buried types.

- Seismic design against liquefaction must be carried out by examining the seismic performance caused by permanent displacement of earth from liquefaction and considering the soil's conditions.
- Regions with need of seismic design against liquefaction must be selected based on geology and geomorphology, ground situation, and gas facilities installation position.

The earth permanent displacement caused by liquefaction should be considered as follows.

- Horizontal displacement caused by lateral spreading on sloped surfaces of ground
- Horizontal displacement caused by earth lateral spreading in seaside regions
- Ground settlement

The effect of liquefaction is measured as vertical and horizontal displacements and is applied consistent with the given distribution on the buried structure.
If gas facilities such as pipelines are installed on structures, there would be no need to consider the ground settlement.

3-8-2-Landslide

In mountains regions with high ground slope and weak layers, there is a possibility for landslides and may cause damages to different communication equipments.
In order to prevent the landslide hazard caused by permanent ground displacement (PGD), the evaluation must be carried out based on the following steps:

- Evaluation of the ground susceptibility to landslides.
- Evaluating the potential for triggering landslides and slope deformation.
- Evaluating the probability of landslide and slope deformation occurrence.
- Evaluating hazards resulted from landslides and slope deformation

3-8-3-Faulting

It is impossible to avoid regions with faulting potential in communication systems. Therefore, the effects caused by displacement of active faults which structures of this system are installed through them, should be considered as far as possible.

- Existence of the active fault shall be determined by the specific geological features of the active fault shape.
- Area through which it is possible to cross an active fault should be confirmed by geological survey, geophysical explorations, boring explorations, and trench survey.
- Whenever gas facilities cross an active fault, they should be designed considering the permanent ground displacement from faulting in order to realize the seismic performance.
- Whenever the effect of faulting becomes visible on ground surface, the gas facilities should be seismic designed for faulting.

Chapter 3 of vital vessels indicates how to calculate and apply loads caused by seismic geotechnical hazards.

3-9-Soil classification

For simpler use of 2800-standard equations, the same soil classification is used as in this standard.

Chapter 4

Methods of seismic design and safety control

4-1-Seismic design

4-1-1-General

Seismic design of communication system is performed according to site conditions and its seismic specifications using methods given in this guideline.
The aims of seismic designing communication system components are:
In operational earthquake in hazard level 1, material behavior isn't exceeded elastic range and their operation is continued unceasingly.
In earthquake design in hazard level 2, despite exceeding material behavior from elastic range, their ductility is confined in a manner that no rapture occur in them, limited probable damages is removable quickly (through compulsive repairs) and their operation is continued unceasingly.

Damages in equipment can be considered as three following modes:

- Physical damages in which, a component may suffers huge deformation but no crack occur that may influence function (damage)
- Function damage in which crack or rupture occur that led to material leaking, short circuit or other similar states and disorder function (failure)
- Systematic damage in which, continuation of system function isn't possible and its function is stopped due to aggravation and propagation of functional damages (instability)

4-2-Principles of seismic design method

Generally, communication system components based on used hazard level is designed by one of allowable stress methods (elastic behavior) or ductile (plastic behavior).
Allowable stress method is used when hazard level 1 is used.
In the cases that hazard level 2 is used, seismic design is performed through ductile method.

- In allowable stress method, stresses in components must not exceed allowable values. Unless, irreversible deformations is remained in components after earthquake occurrence.
- In designing through ductile method, plastic deformations in components must be lower than allowable plastic deformation values. In this case, equipment function during and after earthquake is operable with minimum suspension.

4-2-1-Designing through allowable stress method

4-2-1-1-Stress calculation

Final stress is obtained with combination of stresses resulted from various loads.

4-2-1-2-Allowable stress in seismic analysis

Allowable stresses of materials are defined according to equipment type and its location.

4-2-1-3-Investigting calculative stress

esign of a structure is acceptable when all calculative stresses are lower than values of allowable stresses

4-2-1-4-material characteristics in designing through allowable stress method

1-concrete structures and reinforced concrete

1-1-minimum required compressive strength for concrete is as follow:

1-1-1-For prefabricated pipe supporters (pipe rack): $f_c \geq 25\ N/mm^2$ (3500p.s.i)

-For structures, foundations, floor-buildings and other structural building: $f_c \geq 21.1\ N/mm^2$ (3000p.s.i)

-For fire-resistant components and for channels: $f_c \geq 18\ N/mm^2$ (2500p.s.i)

-For lean concrete: $f_c \geq 8.0\ N/mm^2$ (1100p.s.i)

-For concretes that use sulfate-resistant cement, cement grade mustn't be lower than $310 kg/m^3$.

- For other cements, the grade mustn't be lower than $310 kg/m^3$.

1-2-Reinfocing steel

a) Deformed reinforcements

Deformed reinforcement must have 60 degree end hook (with minimum yield strength $f_y = 414\ N/mm^2$) according to ASTM615 or equivalent similar materials.

b) Smooth reinforcement must have 40 degree end hook (with minimum yield strength $f_y = 276\ N/mm^2$) according to ASTM615 or equivalent similar materials.

c) Welded Steel Mesh must have 70 degree end hook (with minimum yield strength $f_y = 485\ N/mm^2$) according to ASTM615, A496, A497 or similar materials.

1-3-Bracers, sheets and other steel materials used in concrete

Required materials for bracers, sheets and other steel materials for replacement in concrete must be of ASTM A36, weld able according to ASTM standard or similar materials.

1-4-Allowable stresses

Allowable stresses for concrete and steel must be selected according to part 9 of national building regulations and Iranian concrete code (ABA).

1-5-Allowable deformations and rises

Allowable rises of concrete component must be according to part 9 of national building regulations and Iranian concrete code (ABA).

2-steel structures

2-1-materials

- Materials must be according to ASTM A36 or similar standard.

2-2-Bolts

-Bolts used in connections must be according to ASTM A235 or similar standard.

-Bolts used in secondary connections must be according to ASTM A307 Grade A or similar standard.

-Common and high strength nuts must be according to ASTM A563 or similar standard.

-Washer must be according to ASTM F436 or similar standard.

3-Allowable values of cable

Allowable values of cable are varied according to its producers. Failure control of cable under earthquake is examined through elongation test.

4-Supporter and supporting components

Designing supporter and supporting components of internal equipment is performed on the basis of allowable short-term stress.

Section of components that act only during earthquake must be designed with consideration of allowable short-term stress and components that permanent load are applied on them must be designed for justification of allowable long-term stress and short-term stress due to permanent loads and earthquake.

For steel rebar, allowable short-term stress is 1.5 times of allowable long-term stress

Table 4-1-Applied load combination related to long and short-term stresses in internal equipments

Allowable stress	Target load	force
Long-term allowable stress	Constant load + Live load	Long-term force
short-term allowable stress	Constant load + Live load+ Seismic load	Short-term force

4-2-2-Method of seismic analysis for ductile design method

4-2-2-1-Seismic design

Method of ductile seismic analysis is used for seismic analysis of structure in hazard level 2 for design earthquake

4-2-2-2-Seismoc response analysis

Elastoplast deformation in each point of structure can be derived with response analysis under effect of design earthquake.

Plastic deformations for seismic design of structures are obtained from response analysis through one of following methods.

1-Deisgn method of ultimate plastic deformation

In structures that the first mode of vibration is dominant in them, ductility factor can be derived through applying constant energy law for probable failure mode.

1-1-Modified factor of design earthquake

Modified factor of design earthquake is calculated according to chapter 3.

1-2-Ductility factor

Ductility factor μ_p of damaged part of structure can be derived using relation 4-1.

$$\mu_p = \frac{1}{4C}\left\{\left(\frac{K_{MH}}{K_y}\right)^2 - 1\right\} \qquad (4\text{-}1)$$

Where

μ_p : Ductility factor of component related to failure mode when $K_y \geq K_{MH}$ then $\mu_p = 0$.

K_{MH} :Modified horizontal earthquake factor of design related to intended structure

K_y :Horizontal earthquake factor in the beginning of yield of damaged component

C Factor that is related to hysteresis behavior in energy absorption and reaching to intended failure mode as following:

C=2n When residue characteristic is of perfect elastoplastic type.

C=1n when residue characteristic is of sliding type.

n number of suitable cycle from related hysteresis curve. When determination of cycle number isn't possible through detailed equations, its value is considered to be one, conservatively.

In these conditions, vertical earthquake factor must be applied in the worst state.

1-3-Estimation of plastic deformations

In ductile design method, relation 2-4 must be justified ultimately.

$$\mu_p \leq \mu_{pa} \tag{4-2}$$

μ_p Ductility factor of component that is exposed with failure

μ_{pa} Allowable ductility factor

2-Design method of yield strength (for framed structures)

For frames, plastic seismic design is performed using yield strength design method

2-1-Modified seismic factor of design

This factor is obtained according to the procedure given in chapter 3 about modified semi-static method.

2-2-Structural characteristic factor

Structural characteristic factor D_s is obtained from relation 4-3 or from values given in other codes. D_s value is varied between 0.25 and 0.5

$$D_S = \frac{1}{\sqrt{1+4C\mu_{pa}}} \tag{4-3}$$

Where

D_s Structural characteristic factor (approximately equal to inverse of behavior factor R of structures in 2800 code)

2-3-Seismic capacity

Seismic capacity is obtained using relation 4-4.

$$Q_u = K_y W_0 \tag{4-4}$$

Where

Q_u: seismic capacity

K_y: yield horizontal earthquake factor in the beginning point of yield of component that is exposed with failure

In this case, vertical earthquake factor must be applied in the worst state.

Where

W_0 Operational weight of intended structure

2-4-Required seismic capacity

Required seismic capacity is obtained from relation 4-5.

$$Q_{un} = D_S K_{MH} W_0 \tag{4-5}$$

Q_{un} Required seismic capacity

D_S Structural characteristic factor that is obtained in (b)

K_{MH} Modified horizontal earthquake factor

W_0 Operational weight of structure

2-5-Estimation required seismic capacity

Required seismic capacity Q_{un} must not be exceeded from seismic capacity Q_u .

3-Anlaysis of linear modal response

Analysis of equivalent linear modal response for non-linear components that exceed from yield capacity is performed with reduction hardness from elastic hardness (depends on non-linearity degree and equivalent decay factor)

Analysis of linear modal response using acceleration response analysis is performed on the basis of steps a) to f) in the following:

a) Vertical and horizontal spectrum of the design is computed on the basis of relations 4-6 and 4-7.

$$A_H^{(i)} = 350\beta_1\beta_2\beta_5 \quad (4\text{-}6)$$

$$A_V^{(i)} = 175\beta_1\beta_2\beta_6 \quad (4\text{-}7)$$

Where

$A_H^{(i)}$ Horizontal response acceleration of design for each mode of vibration in terms of cm/s^2

$A_V^{(i)}$ Vertical response acceleration of design for each mode of vibration in terms of cm/s^2

β_1 Importance factor using table 3-1

β_2 Ratio of base acceleration of design using table 3-4

β_5 Magnification factor of horizontal response (for ease and with conservatism, for periods lower than 0.3 seconds and periods higher than 0.3 seconds, 1.5 and 0.75 are considered respectively)

β_6 Magnification factor of vertical response

b) Hardness of component must be decreased based on degree of non-linearity of structure

c) Equivalent decay factor related on plastic strain energy is obtained from nonlinear analysis of structure

d) Response value, R, such as shear force, moment and required for design of each vibration mode is obtained from root mean square method.

$$R = \sqrt{\sum_i R_i^2} \quad (4\text{-}8)$$

Where R is response size of ith mode

e) Response displacement must be obtained from response size of component.

f) Be sure that ductility factor obtained from e) isn’t exceeded from allowable ductility factor.

4-Response analysis of non-linear time history

Time history response analysis is performed as following.

a) Specifications of load-displacement of structure must be based on non-linear hysteresis model and results related to each intended time is directly obtained from time history analysis.

b) Import earthquake wave with maximum acceleration in intended point

c) Ductility factor is obtained from component displacement.

d) Obtained ductility factor mustn't exceed from allowable ductility factor.

5-Displacement response method

This method is performed through doing steps a) through e) that are explained in the following.

a) Displacement of point with certain dislocation is same as response displacement in supports or foundation displacement due to ground movement.

b) Component displacement is obtained from static analysis of model using non-linear relation of displacement-load.

d) Ductility factor is obtained using results of component deformation.

e) Ductility factor mustn't exceed from its allowable value.

4-2-2-3-Ductility factor

Ductility factor is obtained from plastic deformation of failure mode seismic analysis of intended equipment.

4-2-2-4-Allowable ductility factor

Allowable ductility factor of component is determined on the basis of elastoplastic behavior such as fatigue and buckling in alternative loading in terms of seismic failure mode of intended components.

4-2-2-5-Estimation of plastic deformation factor

When allowable ductility factor of all parts of important components is equal or higher than intended ductility factor, estimation of seismic function is acceptable.

4-3-Design considerations in selection of kind, dimension and shape of material

4-3-1-Tower

1-Metal tower: steel type of metallic tower or pole must be confirmed by employer. Metallic components must satisfy provisions of table 4-2.

Table 4-2-Slender factor of metallic components of tower

Supporter	component	Slender factor
Metallic pole Metallic tower	Main pole and arm	200 or lower
	Other components that are used as compressive component	220 or lower
	All compressive spare components	250 or lower

Table 4-3-Minimum thickness of metallic tower components

supporter		Minimum thickness (mm)	component
Metallic pole	Metallic pole made of from plate	1 or more	Mast members
	Metallic pole made of from pipe	2 or more	Mast members
	Other metallic poles	4 or more	Main poles and arms
		3 or more	Other members
Metallic tower	Metallic poles made of pipe	2.4 or more	Main poles and arms
		1.6 or more	Other members
	Other towers	5 or more	Main poles and arms
		3 or more	Other members

2-Bolts: Used bolts in towers must be confirmed by employers.

4-3-2-Channal

Materials and connections of channels must endure loads and displacements due to earthquakes as well as enduring normal loads.

Compressive and tensional axial forces, bending and shearing due to ground deformation and relative displacement formed in pipe during earthquake must be minimized during design.

Standard specifications of transportation pipe line material are presented that include FRP materials, welded steel pipe, PVC and convoluted rubber connections.

Table 4-4-Specification of pipe material

Pipe material	Material specifications
PFP, FRP	1-Having high strength against compressive and bending forces 2-Resistance against abrasion due to inner coverage of poly ethylene and high resin 3-Flangic or mechanical connections are earthquake-resistant.
steel	1-High strength, toughness and high tenacity. Electrical corrosion potential is high and it is better to be protected by vinyl coating 2-welded connections have high strength against tensional and bending displacement. 3-High weight of pipe and implementation of connections require certain administrative considerations.
PVC	1-Have lower strength than two above states and is brittle in low temperature. Strength against corrosion is high. Weakness against organic solutions, heat and ultraviolet ray. 2-convoluted rubber connections are separated in high deformations due to earthquake. Various section connections of TS have high damage record in earthquakes. 3-Light weight and easy implementation

1-Pipe materials for pipeline with disconnected parts

1-1-Specifications of FRP pipe are as follow:

-Good mechanical characteristics

-Low sensitivity against heat

-Resistant against impact

-Good insulator

-Low heat expansion
-Negligible weathering
-No environmental pollution
-Potential of manufacture with big diameters
-Easy installation
-Easy repair

1-2-Specifications of FPF pipe are as follow:
-Little weathering and corrosion
-Light
-High deformation capacity
-Tight
-Resistance against heat in heating cable
-Suitable for connection to structure
-Low friction
-Insulator
-Capability of easy bending

Table 4-5-Dimensions of PRP pipes

Diameter (mm)	bend		straight
	Radius of curvature (m)	Length (m)	Length (m)
100	6,5,8,10,15,20,25	1,2	1, 2, 4
125	5,.6,8,10,15,20,25	1,2	1, 2, 4
130	5,.6,8,10,15,20,25,35	1,2	1, 2, 4
150	5,.6,8,10,15,20,25,35	1,2	1, 2, 4
200	5,.6,8,10,15,20,25,35	1,2	1, 2, 4
250	5,.6,8,10,15,20,25,35	1,2	1, 2, 4
300	5,.6,8,10,15,20,25	1,2	1, 2, 4

Table 4-6-Dimensions of PFP pipes

weight)kg(	Connection					Pipe body					type
	External length S2	Internal length S1	External diameter D2	thickness T1	Internal diameter d2	Pipe length L	External length of connection D1	Connection length l	thickness T	Internal radius d1	Nominal diameter
9 16 31	130	80	150	10	130	1000 2000 4000	120	83	10	100	100
11 18 35	130	80	175	10	155		145	83	10	125	125
12 19 37	130	80	180	10	160		150	83	10	130	130
14 26 51	130	80	208	12	184		174	83	12	150	150
24 44 85	150	100	270	15	240		230	103	15	200	200
36 66 127	150	100	332	18	296		286	103	18	280	250
52 95 180	200	120	398	21	356		342	123	21	300	300

Table 4-7-Materials and connections of PFP pipes

Diameter		Different pipe connection for AS	Different pipe connection for FRP	Different pipe connection for GP	Connection
100	○	—	—	○	○
125	○	○	○	○	○
130	○	—	—	—	○
150	○	○	○	○	○
200	○	—	○	○	○
250	○	—	—	—	○

2-Pipe materials for pipeline with continuous parts

Function of this type of pipeline is absorption of displacement by pipe body. On the other words, welded steel pipes and PVC pipe with socket connection (TS connection) are used for these lines.

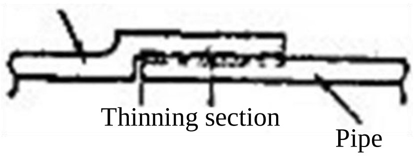


Figure 4-1-Connection of polymeric pipe

3-Function of pipe materials

3-1-PFP and FRP pipes

FRP pipes have suitable function in earthquake. This function is improved through using PFP connections.

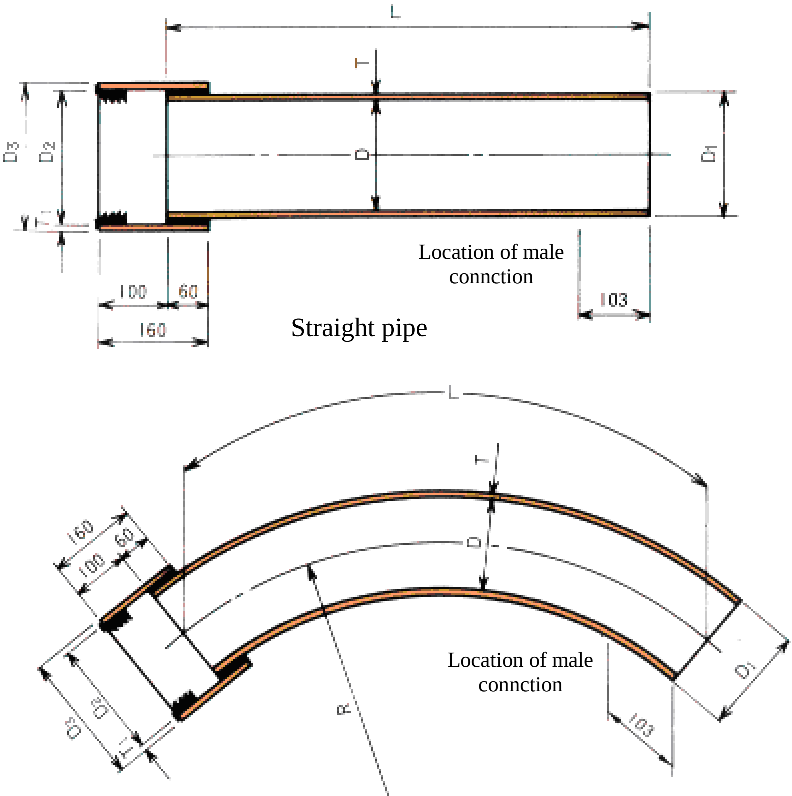


Figure 4-2-PFP pipe before and after earthquake effect

Table 4-8

Nominal diameter	Weight		L		D3	T1	D2	D1	T	D
	Connection (kg)	Pipe body (kg/m)	bent	straight						
100	0.44	2.4	1000 2000	4000 2000	124	4	116	108	4	100
125	0.53	2.9			149	4	141	133	4	125
130	0.54	3.0			154	4	146	138	4	130
150	0.62	3.5			174	4	166	158	4	150
175	0.90	5.1			203	5	193	185	5	175
200	1.01	5.8			228	5	218	210	5	200
250	1.24	7.2			278	5	268	260	5	250
300	1.78	10.4			334	6	322	312	6	300
350	2.41	14.1			388	7	374	364	7	350

3-2-Steel pipes

Steel pipes have high strength, ductility and toughness and are produced in various diameters, long length and different bents. Their connections are as welded and totally require protection against corrosion. For this purpose, corrosion-resistant cupper covers are used. Welded connections are in two types of intruded and angle and angle type is used only for T-shape connections.

Table 4-9

Type	Elongation		Yield point	Tensile strength
	In width direction	In straight direction	N/mm2	N/mm2
STW 290	۲۵>	۳۰>	-	۲۹۰>
STW 370	۲۵>	۳۰>	۲۱۵>	۳۷۰>
STW 400	۱۸>	۳۰>	۲۲۵>	۴۰۰>

3-3-PVC pipe

PVC pipes are used in low diameters and their usual diameter is 75mm.

These pipes are light, very isolative and resistant against corrosion but have low performance against heat.

Table 4-10, present characteristics of this pipe.

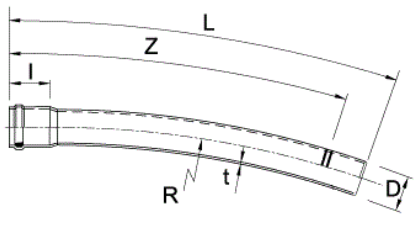


Figure 4-3-Shape of PVC pipe

Table 4-10-Characteristics of PVC pipe

Test name		Method	Value	Unit	Remarks
physical	Specific gravity	JIS X-7112	1.4		
	hardness	X-7215	100-110		
	absorption	X-7209	0.04-0.06	mg/cm^2	
mechanical	Tensile strength	JIS X-7113	53-58	N/mm^2	15C
	Elongation	X-7113	50-150	٪	
	Elastic module of elongation	X-7113	0.27×10^4	N/mm^2	15C
	Compressive strength	X-7208	65	N/mm^2	
	Poisson ratio	X-7113	0.35-0.40		
	Bending strength	X-7203	100-800	N/mm^2	15C
	Elastic module of bending	X-7203	0.28×10^4	N/mm^2	
heat	Expansion factor	X-7112	$6\text{-}8\times10^{-5}$	/C	
	Specific heat		0.85-1.17	J/gK	30C
	Heat conductivity		0.20-0.21	W/mK	

Table 4-11-Dimensions of PVC pipe

Nominal diameter	t	R	Z	D	L	l
150	225	1165	165	1000	5000 10000	8.9

3-4-Box Culvert

Concrete box culvert is one type of conduits for passage of cable lines. This type of conduit doesn't have enough strength against earthquake. Figure 4-4 shows one sample of these culverts.

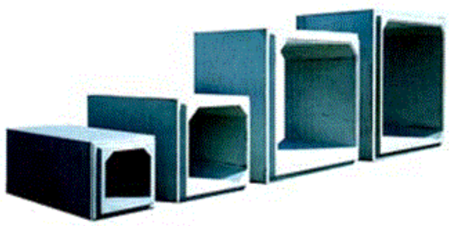


Figure 4-4-Samples of concrete box culvert

4-3-3-Conduit connections

Conduit connections must be designed in such a way that endures displacement due to earthquake. Figure 4-5 shows types of conduit connection to manholes, bridges and connection in soft soils.

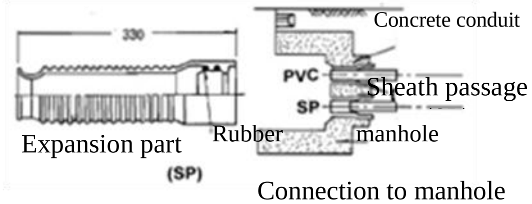
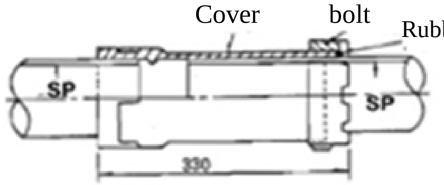
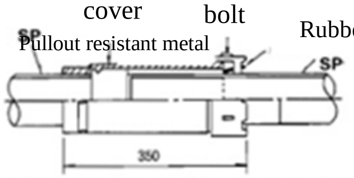
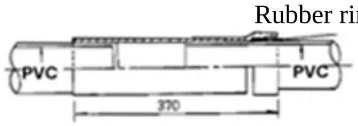
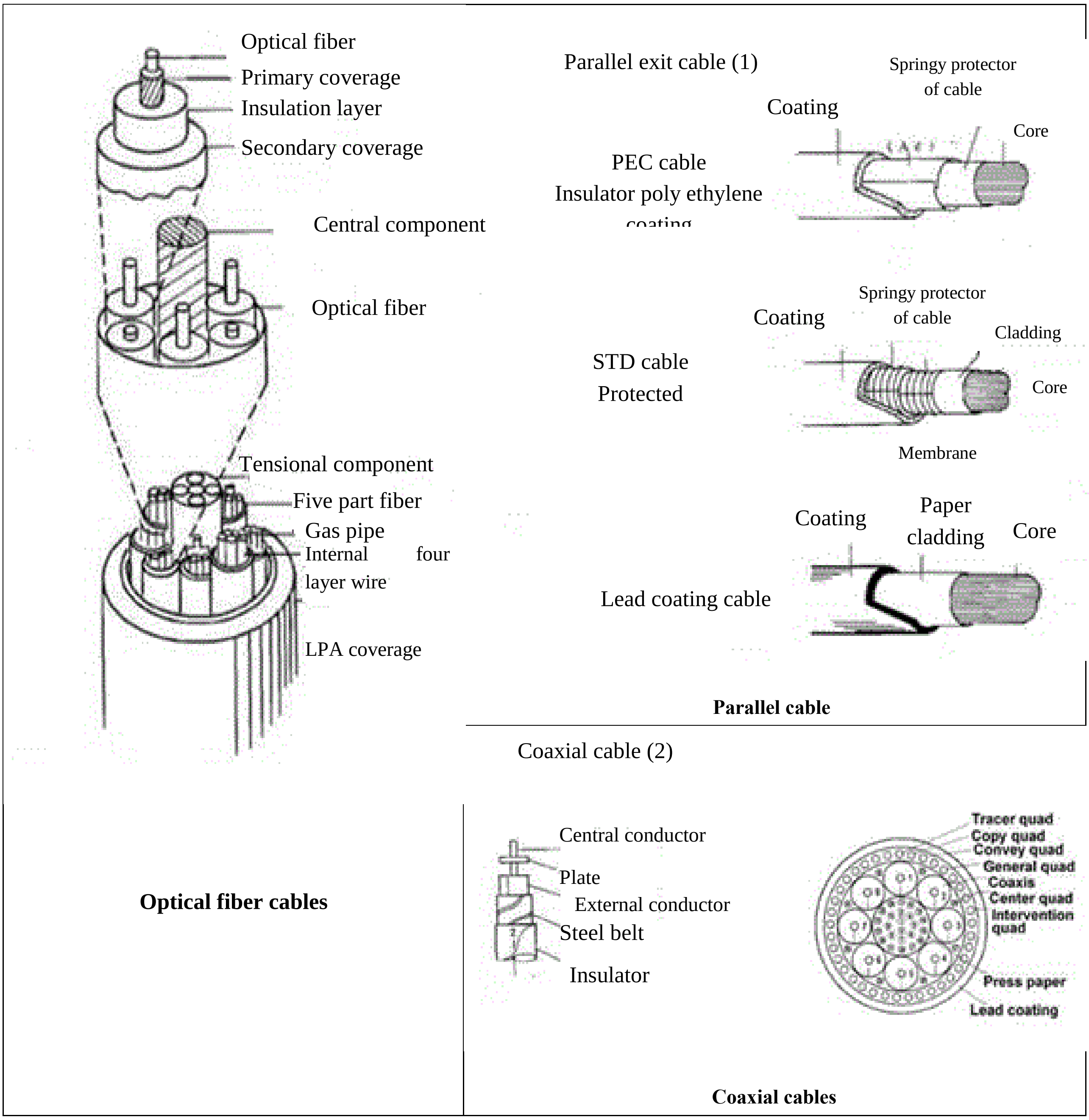
Name	Application	structure
Coverage connection	Part connection to manhole	
Flexible connection	Part connection to protective concrete	
pull-out resistant connections	Connection of supports	
Connection to bridge	Connection of bridges	

Figure 4-5-Seismic resistant cable conduit

4-3-4-Cables

According to the following figure, three types of cable including parallel cable, coaxial cable and optical fiber are used in communication networks.



Optical fiber cables

Parallel cable

Coaxial cables

Figure 4-6- Parallel, coaxial and optical fiber cables

Chapter 5

Seismic design and safety control of aerial equipment of communication networks

5-1-Seismic design

Seismic designs of following equipments are discussed in this chapter:

- Radio masts and towers
- Wireless communication towers
- Poles
- Arial equipments

All of these structures are loaded and designed appropriately through static or dynamic methods for two hazard level 1 and 2. For design and control, Allowable stress method and ductile method are used for hazard level 1 and hazard level 2, respectively.

5-1-1-Mast and tower

- ✓ Seismic safety of these structures is evaluated with consideration of site characteristics, mast/tower itself and its base.
- ✓ Designing masts is usually done using semi-static method.
- ✓ It is recommended to use dynamic analysis in certain geographic, structural and geotechnical circumstances.
- ✓ Sometimes, stresses due to earthquake in metallic tower exceed from corresponding values due to wind load in upper components of the mast/tower.
- ✓ Safe towers/masts against wind have appropriate function against earthquake unless in cases that earthquake leads to instability of mast/tower base.
- ✓ Based on preliminary investigation, type and dimension of structure is characterized for determination of its seismic specification.
- ✓ Preliminary design can be performed under combination of operation loads or wind load.
- ✓ Inertia force is calculated for seismic design of mast/tower and over surface base.
- ✓ Displacement response method is used for buried foundation instead of inertia force.
- ✓ Dynamic analysis is used when semi-static cannot be used for any reason.
- ✓ In most earthquakes, minimum vertical acceleration is the half of horizontal acceleration.
- ✓ Effect of vertical acceleration in total seismic response of masts is generally negligible but in seismic design of arms, half of horizontal acceleration must be considered as vertical acceleration.
- ✓ Tower and its dimension can be calculated with attention to the following:
- ✓ Since structure hardness is lower than is of foundation, there is no difference in natural period and mode shape of mast/tower with solid foundation and ground-foundation-mast.
- Results of separate analysis of foundation and mast/tower haven't considerable difference with results of integral analysis of mast/tower-foundation and soil.
- Natural period of mast/tower is variable according to type, elevation and its vibration characteristics in the direction of transmission and perpendicular direction of it.

5-1-2-Wireless communication towers

- ✓ Metallic towers of wireless communication usually are situated on the roof of structures and their seismic behavior is related to behavior of structure.
- ✓ These towers are predominantly made from metal, especially steel.
- ✓ loader system of these towers are spatial truss

- ✓ wind load is more determinant than earthquake load due to lightness of towers than to windward surface

5-1-3-Poles

- ✓ Communication poles may be made from various materials such as steel, concrete or wood.
- ✓ Main applied loads are
  - Wind load: forces perpendicular on cable line route, make maximum load on the bottom of pole and its foundation.
  - Cable tensional force: cable tensional load are applied on pole in the direction of the axis
  - Vertical force: pole weight
  - Vertical force: weight of pole, weight of connections and cables consist vertical forces that apply on the pole.
  - Earthquake force: Earthquake force for poles is considered as secondary load.
- ✓ In these structures, wind load is higher than earthquake load.
- ✓ Semi-static method is used for loading and seismic analysis of these poles

Distance from other cables and height from ground surface must be considered in design

5-1-4-Aerial equipments

Arial equipments are installed on masts /towers and poles through three methods involving dangle connection, direct connection and connection with seat sheet methods.
Pendulous motions of installed equipments must be considered in connection design.
In earthquake, metallic and bolt connection of equipment installation usually endure acceleration higher than gravity acceleration.
Semi-static method is used for seismic loading of these equipments.
Designing aerial equipment of distribution lines is performed in three stages:

- Designing dangled part and wire or cable in which, tensile strength of wire is selected on the basis of cable types
- Pole design that is variable based on type and number of cables.
- Designing junctions that are performed on the basis of imbalance forces applying on poles according to type and number of cables.

5-2-Calculations

5-2-1-Masts

5-2-1-1-Calculation of natural period

Natural periods of metallic masts can be calculated from relation 5-1 and 5-2.
In the direction of the line

$$T_0 = 1.23X^{0.29} \qquad 5\text{-}1$$

perpendicular to the line direction

$$T_0 = 1.14X^{0.29} \qquad 5\text{-}2$$

Where

T0 is period of natural vibration in terms of second and X is obtained from relation 5-3 in terms of second.

$$X = \sqrt{\frac{(W_T + W_C)H^2}{g \cdot E \cdot I_B}} \qquad 5\text{-}3$$

Where

W_T Weight of mast in ton that involve body, arms, wire and all accessories

WC Effective weight of wire in ton according to table 5-4.

H Elevation of mast/tower (meter)

g Gravity acceleration (m/s2)

E Elastic module of metallic materials of mast/tower (ton/m^2)

IB Bending inertia moment of mast in application point of bending (m4)

Table 5-1-Determination of Effective weight of wire

type	**Effective weight (percent of total weight)**	**Direction of input earthquake**
suspending	**0** **0** **50%**	In the direction of line Perpendicular to the line direction vertical
tensile	**50%** **0** **50%**	In the direction of line Perpendicular to the line direction vertical

5-2-1-2-Determination of shear force and bending moment related to various level of mast

Shear force and bending moment due to earthquake in steel mast/tower in the direction of line and perpendicular to it is given by relation 5-4 and 5-5.

$$Q_i = C_{Si} \cdot W_i \qquad 5\text{-}4$$

$$M_i = C_{Mi} \cdot W_i \cdot H_i \qquad 5\text{-}5$$

Where

Q_i Shear force of story in elevation h_{bi} from base level of mast in ton.

Mi Bending moment story in elevation h_{bi} from base level of mast in ton.

Cs_i, CM_1 Factors of shear and moment of story in elevation h_{bi} from base level of mast in ton.

$$W_i = \sum_{J=1}^{i} W_J \qquad 5\text{-}6$$

Where

W_i weight of mast/tower from intended level to upper sector

H_i distance between h_{bi} and gravity center of upper section, h_{bi} in ton

$$H_i = \sum_{j=1}^{i} W_J \left(h_{bj} - h_{bi}\right) / W_i \qquad 5\text{-}7$$

Where

h_{bi} height from mast base to intended section

h_{bj} height from mast to panel j in terms of meter

W_J weight of panel j in terms of meter

Shear factor of story and moment factor of story are given by relations 5-8 and 5-9.

$$C_{Si} = R_S \cdot A_{Si} \cdot K_H \qquad 5\text{-}8$$

$$C_{Mi} = R_M \cdot A_{Mi} \cdot K_H \qquad 5\text{-}9$$

Where

R_S ‘R_m Response factor for calculation of shear and moment factors of story

A_{si} ‘A_{mi} Factors of shear distribution and moment of story

K_H factor of horizontal earthquake of design (according to Appendix of this guideline: seismic loading of lifelines)

2-Moreover, shear force and moment of arms in each direction is obtained from following relations.

In line direction and perpendicular ti it:

$$Q_{Ai} = A_{ASi} \cdot R_S \cdot K_H \cdot W_{Ai} \qquad 5\text{-}10$$

$$M_{Ai} = A_{AMi} \cdot R_M \cdot K_H \cdot W_{Ai} \qquad 5\text{-}11$$

In perpendicular direction

$$Q_{Ai} = A_{ASi} \cdot K_V \cdot W_{Ai} \qquad 5\text{-}12$$

$$M_{Ai} = A_{AMi} \cdot K_V \cdot W_{Ai} \cdot l_x \qquad 5\text{-}13$$

Where

Q_{Ai} shear force of arm in level of h_{bi} from mast/tower base in terms of ton that is assumed positive in the direction of arm

M_{bi} shear moment of arm in level of h_{bi} from mast/tower base in terms of ton.meter

A_{Asi}, A_{Ami} distribution factors of shear and moment of arm level

W_{Ai} Arm weight involving weight of accessory parts in terms of ton

l_x arm length from beginning to end in meter

K_V Earthquake factor in vertical direction is assumed to be $\frac{1}{2} K_H$

5-2-1-3-Determination of response factor for calculation of shear factor and story moment

Response factors, R_s related to shear factor of story and R_m related to factor of story moment are determined using following diagrams on the basis of natural period T_0 and ground type.

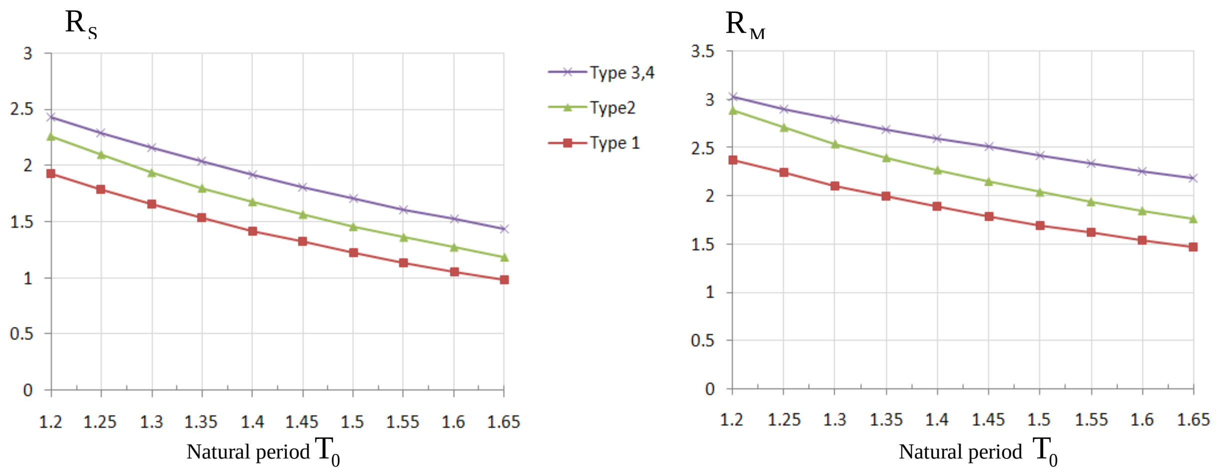


Figure 5-1-R_s and R_m related to mast of two sided dangle in the direction of transmission line

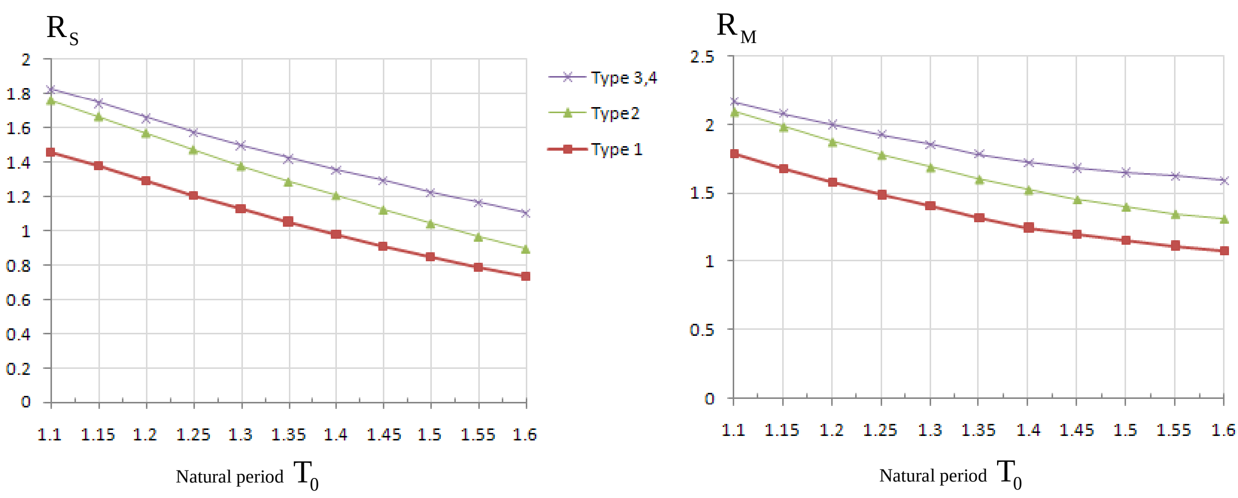


Figure 5-2-R_s and R_m related to mast of two sided dangle in the perpendicular direction to transmission line

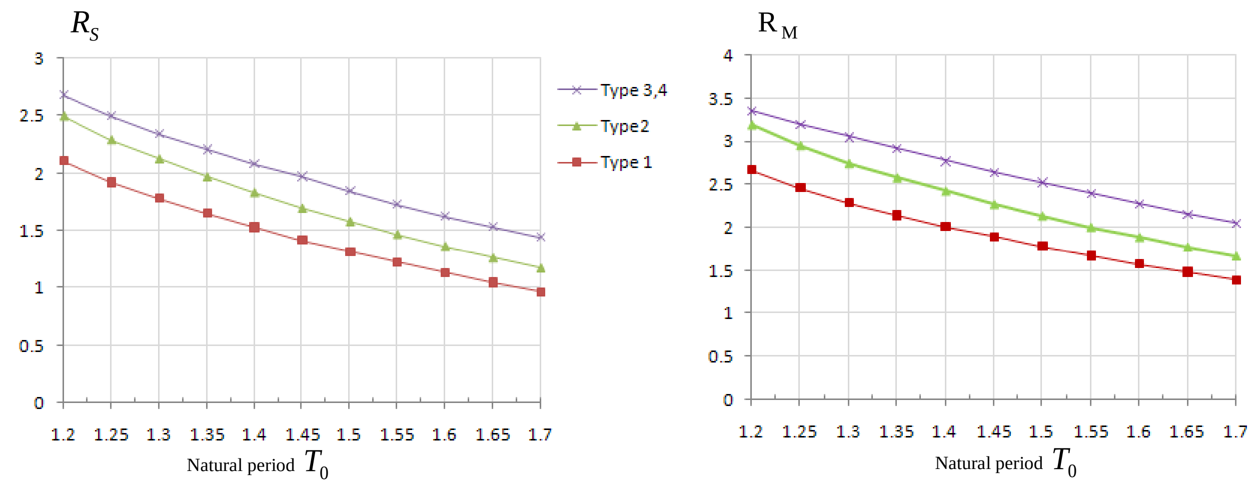


Figure 5-3-R_s and R_m related to tensional mast in the direction of transmission line

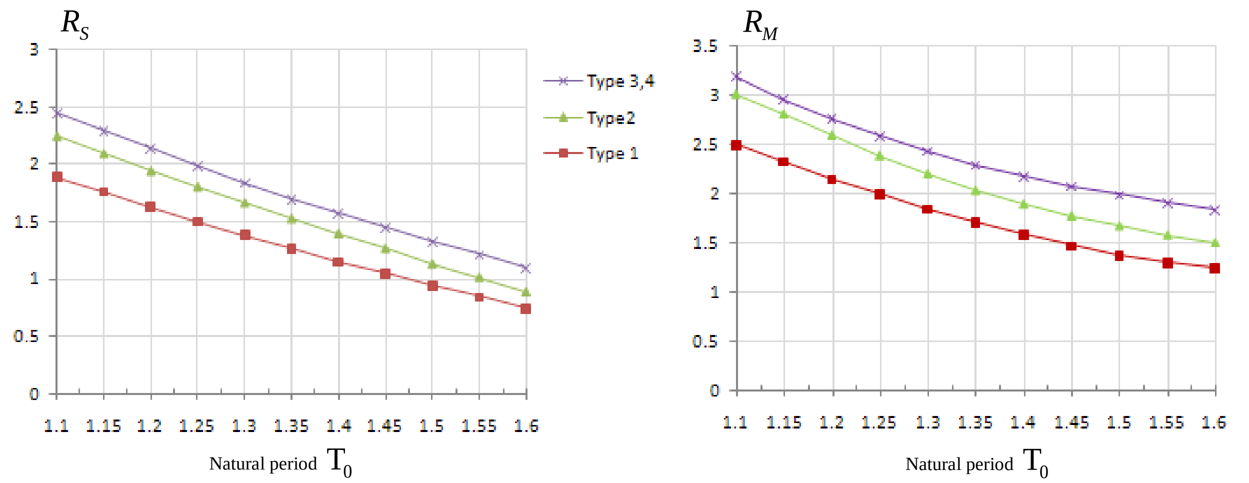


Figure 5-4-R_s and R_m related to tensional mast in the perpendicular direction to transmission line

5-2-1-4-Determination of stress distribution factor and story moment for mast/tower

Shear distribution factor and story moment for mast/tower are determined using following diagrams. In these diagrams:

Wi weight of upper section from hbi level

W total weight of mast/tower involving effective weight of wires and accessory parts

Hi distance hbi to gravity center of its upper section

Hb height of mast/tower gravity center from base

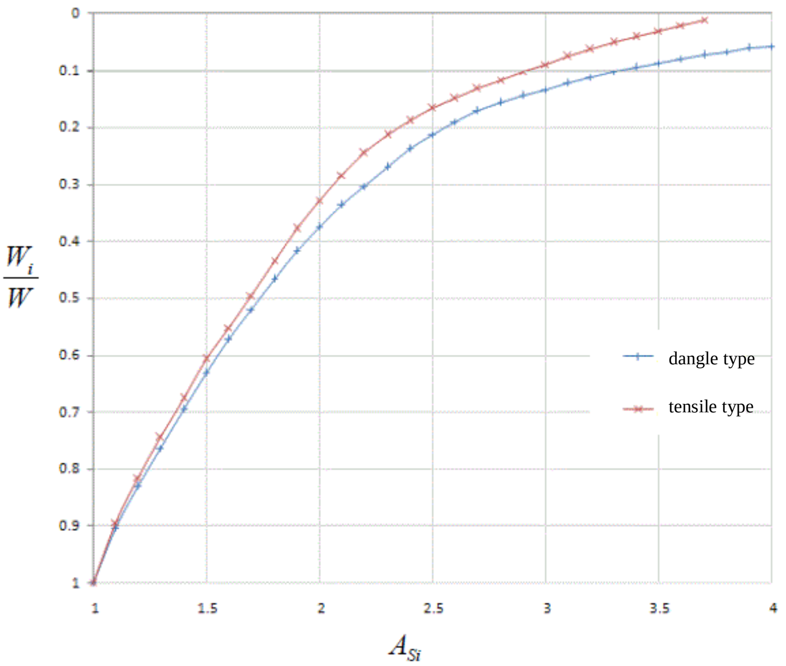


Figure 5-5-Shear distribution factor of story in the direction of transmission line, A_{Si}

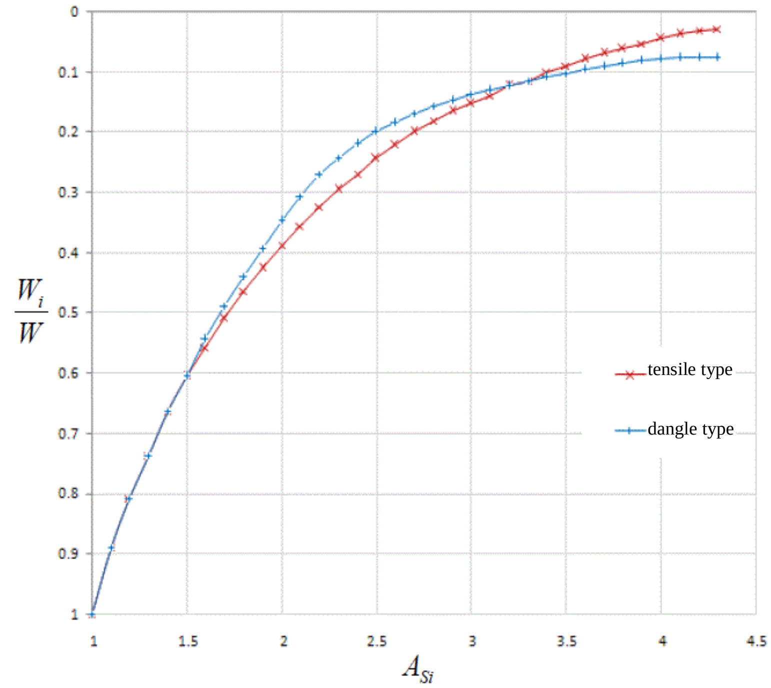


Figure 5-6-Shear distribution factor of story in the perpendicular direction to transmission line, A_{Si}

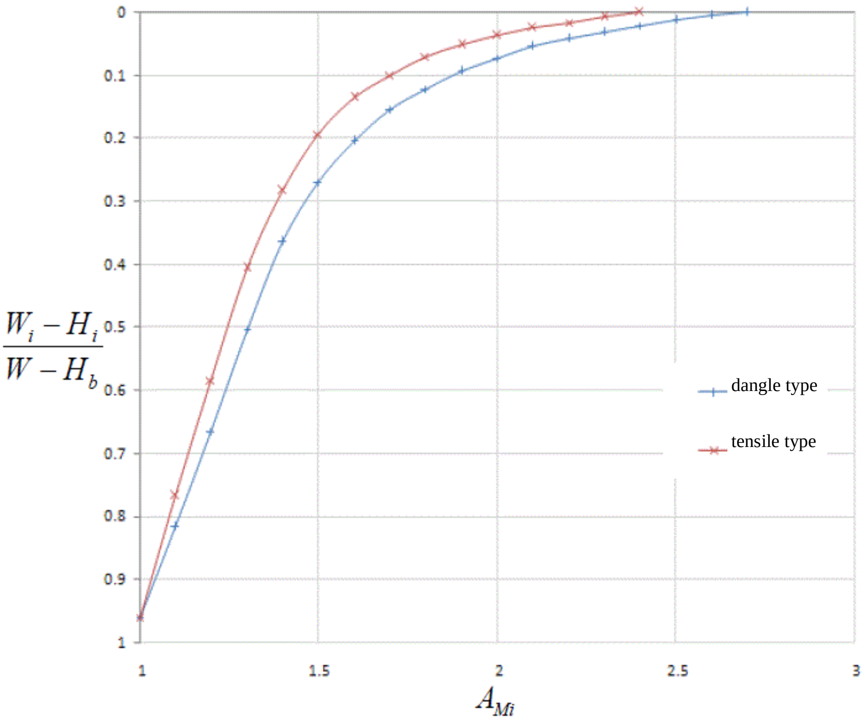


Figure 5-7-Factor of moment distribution in the direction of transmission line, $A_{M\,i}$

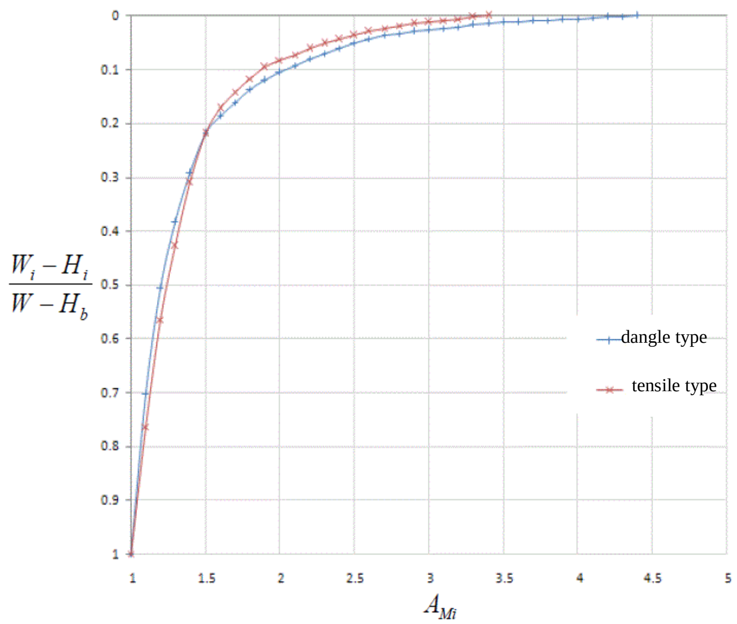


Figure 5-8- Factor of moment distribution in the perpendicular direction to transmission line, $A_{M\,i}$

5-2-1-5-Shear distribution and arm moment factors

Shear distribution and arm moment factors are obtained using following diagrams. diagram parameters are as pervious.

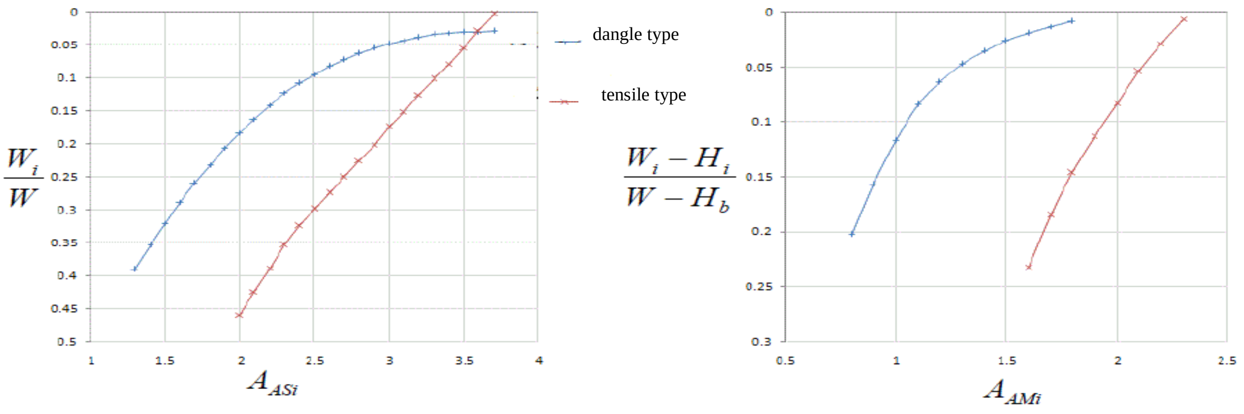


Figure 5-9-Shear distribution and arm level moment factors in the direction of transmission line, $\mathrm{A}_{\mathrm{AMi}}$ and $\mathrm{A}_{\mathrm{ASi}}$

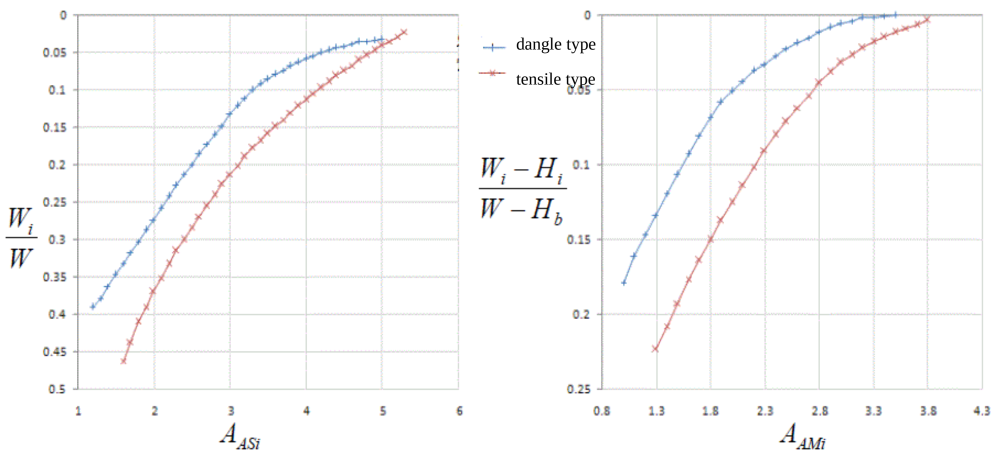


Figure 5-10-Shear distribution and arm level moment factors in perpendicular direction to transmission line, $\mathrm{A}_{\mathrm{AMi}}$ and $\mathrm{A}_{\mathrm{ASi}}$

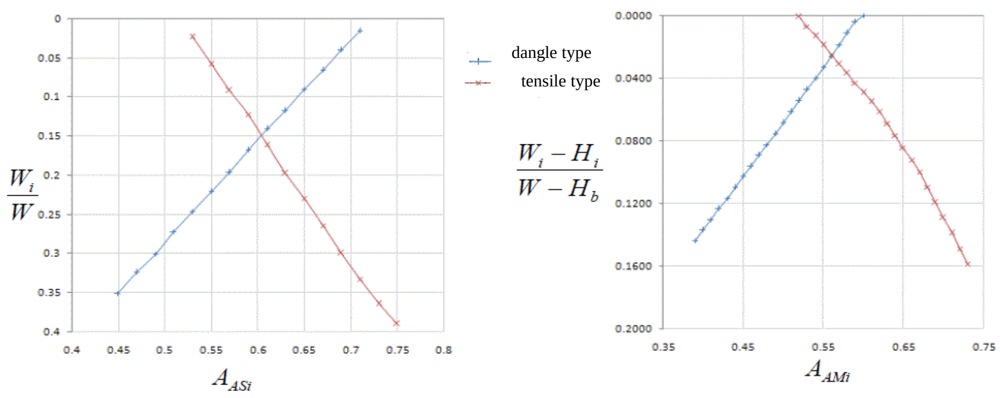


Figure 5-11-Shear distribution and arm level moment factors in the vertical direction of transmission, $\mathrm{A}_{\mathrm{AMi}}$ and $\mathrm{A}_{\mathrm{ASi}}$

5-2-1-6-Calculation of component stress under earthquake effect

Produced stress in components must be calculated under shear and story moment effect
These stresses must be considered in two directions and in arm and also in the vertical direction.

5-2-1-7-Calculation of combinational stress

Stress of components must be combined with stresses due to dead loads and extension of wire and earthquake.

5-2-1-8-Size control of component section

Values of combinational stresses mustn't exceed from related allowable stress.

5-2-1-9-Calculation of foundation design force

1-Foundation design force must be computed on the basis of combinational effect of normal loads and earthquake

2-Transferred force from mast to foundation is obtained using shear values and story moment due to earthquake and earthquake effect in both directions must be considered to derive maximum compressive and tensional force in foundation.

5-2-2-Wireless communication masts

5-2-2-1-Design steps

1-Modified earthquake factor is used in design through allowable stress method

2-Dynamic analysis id used for design through ductile method

5-2-2-2- Steps of design through semi-static method

1-This method is applicable for RC structures or SRC (Steel Reinforce Concrete) with elevation of 30 m or lower.

2-This method is applicable for masts with elevation lower than 60 m and maximum 4 bases.

3-Decay is 1% and natural period is obtained from following relations:

ast

$$T = 0.015H \qquad 5\text{-}14$$

Building

$$T' = 0.03 + 0.06F \qquad 5\text{-}15$$

Where

H mast height

F number of building stories above the ground surface

4-Factors of standard shear force is calculated as following:

For wireless mast that is installed on the ground:

$$C_0 = 0.6\left\{1 - 0.2(T/0.6 - 1)^2\right\},\quad C_0 \le 0.6 \qquad 5\text{-}16$$

For wireless mast that is installed on the roof:

$$C_0 = 0.6 + 1.2\left\{2 - (T/T')/1.2)\right\}^2, 0.6 \le C_0 \le 0.6 \qquad 5\text{-}17$$

5-Factors of shear force of various levels is equal to following:

$$C_0' = C_0 \cdot \alpha \qquad 5\text{-}18$$

Where α is obtained from figure 5-12.

6-Seismic design force in various levels is equal to following:

$$F = C_0' \cdot \sum W \cdot Z \qquad 5\text{-}19$$

Where ΣW is sum of dead load in intended point and Z is zone factor.

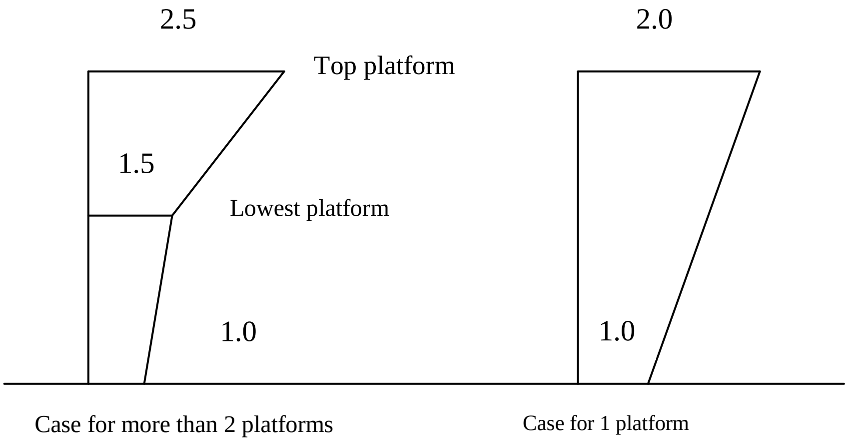


Figure 5-12-α values

5-3-Baese

5-3-1-Design loads

1-Wind load is the main load of bases and these structures are designed under wind load and wire tension.

2-Earthquake inertia force is applied for base stability control.

5-3-2-Procedure of designing for bases of distribution lines

1-Calculation wind load

Wind load is calculated using relation 5-20

$$P_w = 1/2 \cdot \rho \cdot 9.8 \cdot C \cdot V^2 \qquad 5\text{-}20$$

PW wind pressure N/m2

ρ air density Kg/m3

C strength factor

V wind velocity m/s

Strength factor is based on mast shape, magnitude and velocity of wind that is obtained from wind tunnel test.

There are three types of wind load:

A type: 80 kg/m2 for base, 100 kg/m3 for cable and 160 kg/m3 for accessory.

B type: half of A type with consideration of snow with density of 0.9 and thickness of 6 mm over cables

Type C: half of A type

Type C is usually applied for civil lines and type A for suburban lines and type B for snowy areas. Meanwhile type of region climate and topography must be considered in calculation of wind load.

2-Cable tensile force

Cable tensile force is calculated as following.

$$T = W_L \cdot S^2 / (8 \cdot d) \qquad 5\text{-}21$$

Where

T tensile force (N)

d cable rise (m)

WL effective weight of cable in length unit (N/m)

S span length (m)

Rise d is shown in figure 5-13.

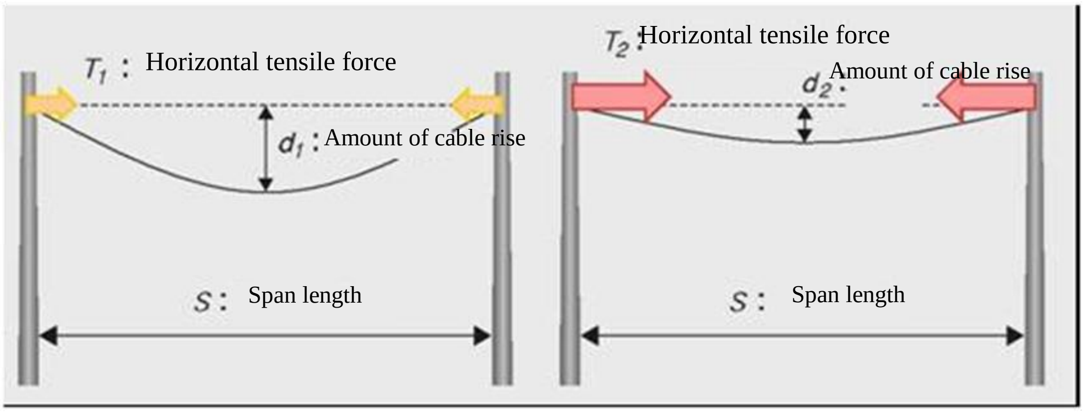


Figure 5-13-Cable rise and tensile force

According to figure 5-14, effective weight of cable in length unit is equal to

$$W_L = \sqrt{w^2 + P_c^{\,2}} \qquad 5\text{-}22$$

Where

W_L effective weight of cable in length unit (N/m)

P_c wind force in length unit (N/m)

w cable weight in length unit (N/m)

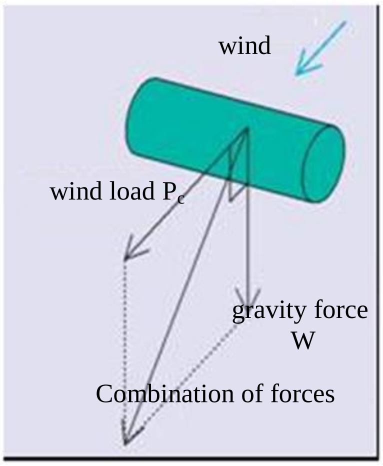


Figure 5-14-Combination of wind load and gravity load for base design

Maximum tensile force in minimum temperature is calculated to consider effect of heat stress

3-Calculation of vertical force

Weight of base and accessory, snow and ice over it, wire with consideration snow and ice, vertical component of weight of base bracer cables and weight of workers and tools during installation are presented as vertical loads.

4-Load effect point

Figure 5-15 shows equilibrium model of base. Force and base moment are obtained on the basis of this model.

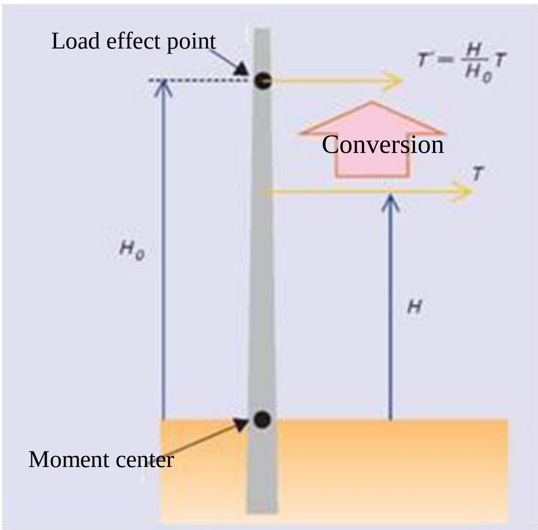


Figure 5-15-Effect point of loads in base

5-Base design

5-1-Types of base

Bases are usually in two types of metallic and concrete. Concrete bases are constructed from hollow column of reinforced concrete that has fine shape with low cost but relatively high weight.

Metallic bases often made from galvanized sheets in cylindrical, conical or prism shape with lower weight but higher cost than to concrete bases.

5-2-Base shape

According to figure 5-16, base section is usually in conical shape. slope of this cone is given by

$$\alpha = (D' - D)/L \qquad 5\text{-}23$$

Where for concrete and metallic bases, this slope is assumed to be $\frac{1}{75}$.

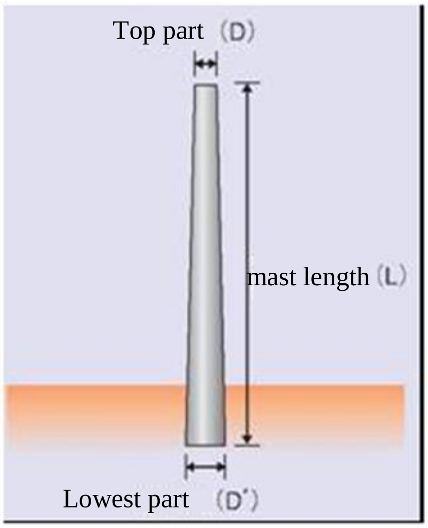


Figure -5-16-Determination of conical base slope

According to 5-16, maximum bending moment of base is produced in earthing part. Control of base section is usually done in the ground level and other parts have smaller sections.

5-3-Base strength and bearing capacity

There is two mode of base failure involving base failure in hard ground and base tilting in soft ground.

Allowable strength of base is determined from bending test (figure 5-17).

In concrete bases, cracks wider than 0.25 mm mustn't occur due to maximum load and after load lifting, reminded cracks mustn't have wide bigger than 0.05mm.

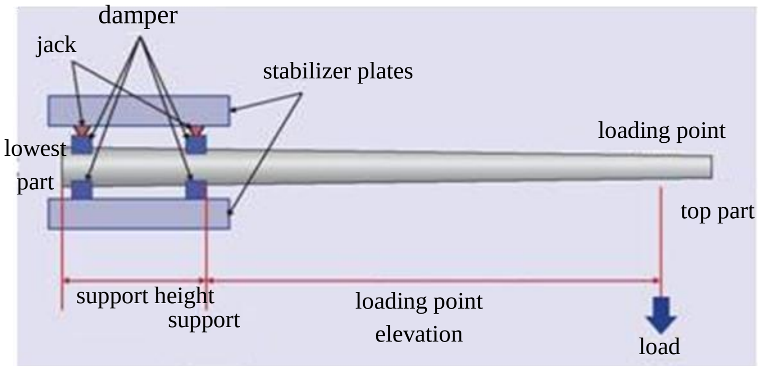


Figure 5-17-Base bending test

Bearing capacity of the ground

Maximum moment of base in earthling is given by:

$$M_{ot} = P \cdot (h + t_o) \text{ (kN.m)} \qquad 5\text{-}24$$

Where

P force (kN)

h height of force application from ground level (m)

t_0 depth of rotation center from ground level according to figure 5-18.

Maximum moment of tilting threshold is called maximum strength moment of the ground which is used in design through application of safety factor

$$M_{ot} \le M_{oa} \qquad 5\text{-}25$$

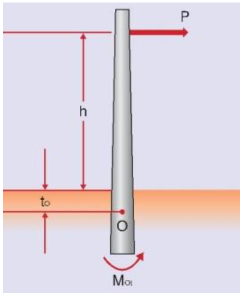


Figure 5-18-Applied moment on center of base rotation

The deeper buried length of base, the more maximum resistant moment of ground. This length is variable based on ground type.

5-4-Seismic design of base

Procedure of base design against wind is applicable for earthquake force, as well.

Earthquake force is in second degree of importance in designing bases.

For designing bases against earthquake, P´ must be obtained at first as following and used in above mentioned relations instead of P.

If moment due to earthquake force is higher than moment due to wind force, earthquake force doesn't combined with wind force and doesn't considered when is lesser than wind force.

Force due to earthquake is given by:

$$P' = k \cdot W_s \qquad 5\text{-}26$$

Where

k design earthquake factor

W_s base weight kN

5-4-Allowable values

Allowable values of design include stresses, bearing capacities and displacements.

1-Allowable stress

In semi-static method, displacement response method and story shear factor method, allowable stress is considered to be 1.5 times of allowable stress related to operational mode.

2-Foundation displacement

Vertical or horizontal uniform displacement of foundation usually don't cause problem in masts.

When relational displacement occurs between mast bases, it causes additional stresses in components.

This problem is aggravated in masts that are located on slopes.

1-Foundation displacement in soft grounds

If foundation is built on soft ground, displacement among mast bases under seismic waves may be very high. When direction of seismic waves is parallel to the line or perpendicular to it, values of displacement are relatively low.

In the case of applying earthquake load with 45 degree to line direction, this displacement must be calculated and controlled.

Now, allowable value of relative displacement is considered to be $\frac{1}{800}$ that is obtained based on amount of displacement absorption due to tolerance and looseness of bolts.

2-Displacement of mast foundation in common ground

Displacement of mast foundation in common ground under earthquake is low and doesn't cause any problem.

When ground condition of bases are variable or foundation is located on slope, displacements are higher that can cause high additional stress.

Allowable displacement of foundation in common ground is considered to be 1.5 times of allowable displacement that is 15mm in usual condition.

When extended foundation is used and adequate bearing capacity is supplied, there is no need to control deformation.

5-5-Acceptance criterion

Combined stress due to earthquake in all sections of components and stress of designed arm for wind force mustn't be exceeding from allowable stress. Displacement due to earthquake mustn't be exceeding from allowable values of displacement.

Chapter 6

Seismic design of communication line conduits

6-1-Principal components of buried conduits of communication lines

Principal components for conduits of cable lines are:

1-Butied conduits

2-Culvert and tunnel

3-Accessory equipments for conduit

Seismic design of line conduits is performed in two parts:

1-Overground conduits

2-Underground conduits

6-2-Design method on the basis of conduit function

6-2-1-General

Seismic design of transmission conduits must be performed with regarding function level of operation to fulfill intended conditions after earthquake. In limit state of damage, circuit must maintain seismic hazard level 1 of its function immediately after maximum earthquake of operation; i.e. behave elastically under this earthquake.

In ultimate limit state, conduit must be repairable and recyclable in minimum time after earthquake, namely, doesn’t rapture or totally fracture under earthquake of seismic hazard level 2.

Allowable stress method is used for earthquake of seismic hazard level 1. Ductile method is used in earthquake of seismic hazard level 2.

6-2-2-Procedure of seismic design for underground pipelines

Diagram of seismic design under effect of seismic waves for underground pipeline and its accessory must be prepared for allowable stress and ductile design methods.

Same method must be used for designing conduits and its accessories to investigate of earthquake wave effect in both allowable stress and ductile methods. Wave effect is described with inertia force (acceleration spectrum).

6-2-3- Procedure of seismic design for buried conduits

6-2-3-1-Effect of earthquake wave on buried conduits

Diagram of seismic design under effect of seismic waves for buried pipeline and its accessory must be prepared for allowable stress and ductile design methods.

Same method must be used for designing conduits and its accessories to investigate of earthquake wave effect in both allowable stress and ductile methods. Wave effect is described velocity response spectrum.

6-2-3-2-Effect of permanent ground displacement in buried conduits

In this case, seismic design is performed fir following areas:

1-Intersection of fault
2-Liquefaction areas
3-Sliding areas

Permanent ground displacement under earthquake of seismic hazard level 2 usually inflict severe damages on pipelines so seismic design for permanent ground displacement (PGD) in earthquake of seismic hazard level 2 due to fault movement and liquefaction must be considered.

6-3-Required function level for components and equipment of cable transmission conduits and seismic hazard levels

6-3-1-Pipe line design method

1-Seismic design of these structures is performed through semi-static method. It is recommend that dynamic analysis to be used for safety control.
2-Buried structure is designed with semi-static method or displacement response.
3-When structure displacement under earthquake of seismic hazard level 2 is complex; dynamic analysis method must be used.
4-Semi-static method is used for semi-buried structure, as well.

6-3-2-Seismic function of pipeline

As a general level, pipeline safety during earthquake must be controlled with regard of strength and flexibility of pipeline.

With consideration of functional investigation, pipeline conduits are divided into general sets:

1-Discontinued conduits that their flexibility is depend predominately on behavior of connections.
2-Continuous conduits that their flexibility is depend predominately on pipe materials.

First type includes FRP and PVC pipes and second type involves steel pipes. Table 6-1 presents primary framework for seismic function of conduits.

In discontinued and continued conduits, connection deformation capacity and allowable stress of material are controlling factors respectively. Stress for both seismic hazard level mustn’t exceed from allowable stress of pipe materials. In discontinued pipeline, produced deformation under live load and normal condition mustn’t be higher than allowable values of design.

Table 6-1-Basic assumptions for safety control of conduits

Seismic hazard level	Standard and control		Loading conditions
	Continuous pipeline	Discontinuous pipeline	
Seismic hazard level 1	Pipe body stress (elastic mode)>allowable stress of pipe body	Pipe body in elastic mode: allowable stress≥ pipe body stress displacement absorption capacity of connection ultimate mode≥ expansion of proportion of connection	Permanent load+ load of earthquake of seismic hazard level 1 (operational earthquake)
Seismic hazard level 2	Pipe body stress (plastic mode)>ultimate limit stress of pipe body	Pipe body in non-elastic mode: Ultimate limit stress≥ pipe body stress displacement absorption capacity of connection ultimate mode≥ expansion of proportion of connection	Permanent load+ load of earthquake of seismic hazard level 2 (design earthquake)

Maximum bulky expansion is based on connection expansion.

In welded metallic pipe, stress due to earthquake of seismic hazard level 1 must be lower than allowable stress.

In discontinued pipe, investigation of expansion capacity of connection is enough in normal condition and there is no need for body stress control.

In steel pipe in earthquake of seismic hazard level 1, material stress of body under live load must be lower than yield stress of material. Corresponding strain with this stress is as following:

$$\varepsilon = \sigma/E = 2{,}400/2{,}100{,}000 = 0.11\%$$

In order to keep the appearance of pipe, pipe strain must be lower than (23t/D) % or about (0.15-0.2) %. In earthquake of seismic hazard level 2, allowable strain is lower than (46t/D) % or about (0.3-0.4) % with consideration of stationary free load that is another statement of allowable stress control.

Positional buckling strain is about 0.5% that gives positional buckling strain as (58t/D) % which is converted as (46t/D) % with application of safety factor of 1.25.

Since compressive capacity is determinant, buckling criterion is considered as main criterion for controlling seismic behavior.

In the occurrence of great ground displacement, adequate deformation capacity must be predicted in connection, as well.

6-3-3-General consideration in seismic design of conduit

- Pipes that pass instable ground must have sufficient capability in body and connections to endure forces and deformations due to earthquake. If needed, appropriate measure can be taken for structural stability of foundation or soil compaction or both.
- Artificial grounds such as coasts or ancient rivers suffer great deformations in earthquake. In these grounds, pipes must have high capacity for deformation.
- Pipes that are located upper slopes must have sufficient stability against slope sliding.
- Due to probability of occurring relative ground displacements resulted from earthquake, pipeline must have adequate strength for its adsorption especially in specific topographical conditions.
- In liquefactive grounds, pipe must have enough strength.
- Grounds adjacent to shore walls and slopes need high seismic strength
- If crossing across active fault is inevitable, adequate ductility and energy adsorption must be

supplied for pipe and in the case of damage, pipe function must be sustained or suitable alternative system must be predicted for earthquake time.

- Capacity for absorbing deformation must be existed in the connection of pipes to structures.
- It is necessary to consider precise prudence for endurance imposed inertia force due to earthquake as well as instable common force in sections where elevation of pipe changes such as positions in a joint container or civil conduits. In addition to seismic preparations such as tensional and flexible connections (pipe), special prudence must be considered in parts that relative displacement of buried pipeline in earthquake may be increased.
- In arcs and bents of discontinued pipes, parts of bent pipe must be used to increase ductility.
- In positions where pipe crosses roads, pipe must be buried in appropriate depth to prevent interfere with other components of road and ease its repair and maintenance
- In underground intersections between lines, minimum 30 cm clearance must be observed.
- There is needed to place manholes in appropriate points for thick pipes to investigate pipe situation.
- Abrupt bents of high angle must be avoided in lines as long as possible.

In seismic design, geologic investigations and surveying must be accomplished in following positions accurately:

1-Since ground movements affect pipe behavior and due to damage records of lines in past earthquakes, following items must be carefully investigated and analyzed.

1-1-Embarkments

Embankments may be undergone sliding, collapse and intense movements due to earthquakes.

1-2-Artifical lands

Artificial lands in seashores, rivers and old reservoirs may suffer huge liquefaction and displacements in earthquake.

1-3-Locations on top of slopes

Controlling landslides due to earthquake is necessary in top of slopes

1-4-Existance of weak and thick underground layers

Sacrificial deformation may be very significant if there are weak and thick layers underground. Moreover, lowering underground waters and resulted subsidence in soft grounds aggravated these deformations.

1-5-Geologic and topographic variation in narrow sections

Geologic and topographic variation in narrow and boundary sections between bedrock and alluvium can aggravate relative motion potential due to abrupt changes of ground conditions between layers.

1-6- Liquefactive areas

Liquefaction and lateral expansion potential may be very high in the areas with any fine grained layer or areas that are located beneath underground level.

1-7- Lands adjacent to protected seashores

In lands adjacent to protected seashores, severe lateral movements may be occurred due to failure of protecting walls.

1-8- Ramp grounds

Shelvy grounds have high sliding potential in earthquake that can impose extra lateral forces to pipe line.

1-9-Active fault

Active fault mustn't cross pipeline as far as possible, unless providing sufficient flexibility is essential.

2-Areas that must be examined structurally

In some areas, pipeline must be carefully examined with regard of leakages and deformations, including:

2-1-Connection of pipeline to structure that is concentration point of stress and abrupt deformation. Extensional and reflexive connection can be used to reduce or prevent damages in these areas. If these conditions happen in problematic ground, pipe must endure all deformations.

2-2-Pipes crossing interior of conduit coating: in the cases that pipe is coming upward from deep points, displacement of top section to bottom in earthquake must be considered.

6-3-4-Steps of designing conduits

6-3-4-1-General method for seismic designing of pipelines

1-Investigation of pipe installation ground

Ground investigation of pipe installation position involve geologic, topographic and land use aspects that consists following steps:

1-1-Investigation of location on the basis of existing information and known active forms

1-2- Preparation of layout and profile of pipe installation sections

1-3-Compilation geologic maps involving N values, specific gravity of soil and groundwater level

1-4-Refining geologic maps based on complimentary exploration as needed

2-Geologic investigation

2-1-Geologic investigation

Reconnaissance boreholes must be drilled in maximum distance of 100 meter and in lower distance to intended depth in problematic grounds.

2-2-Extractioning dynamic magnification factor of the ground

Dynamic magnification factor of the ground must be determined for each layer of soil.

3-Implementation method and materials of pipe

3-1-Selecting implementation method and materials of pipe

Selecting implementation method and materials of pipe is performed on the basis of geologic conditions, implementation constraints and path type.

3-2-Determination of required number of pipe pieces and needed equipments

Determination of required number of pipe pieces and needed equipments is performed on the basis of intended function.

3-3-Selection of pipe material and thickness and types of connections

Selection of pipe material and thickness and types of connections is performed on the basis of ground type, implementation method and behavior. pipe characteristics is determined based on applied loads involving live and dead load, lateral pressure of load, heat, destabilizing forces and earthquake.

4-seismic calculations

Condition of performing seismic calculations is explained in the following. Behavior of direct pipes and bents must be investigated separately.

5-Safety control

Safety control is performed on the basis of anticipated behavior of structure.

6-3-4-2-Steps of designing pipeline bridges

1-Pipeline bridges are very important structures and their seismic loading is performed predominately by means of semi-static method and horizontal seismic capacity.

2-As road bridges, seismic design of this structure is performed with bridge characteristics

3-According to very low hardness of these structures in the vertical direction, their energy absorption capacity is high in earthquake. Instead, lateral hardness of these structures against wind is lower than road bridges. In previous earthquakes, these structures damaged due to failure of supporting system and their sustainability.

4-In designing through allowable stress method, seismic loading is performed through semi-static method. However most critical compositions resulted from wind or earthquake must be considered.

5- Horizontal seismic capacity is used in ductile designing method.

6-This method cannot be applied for bridge slabs that aren't positioned on the ground.

7-Cable Conduit bridge usually have maximum two predominant vibrations so its design can be performed using semi-static method and dynamic analysis is only used when structure behavior is complex, such as:

- bridges that have significant vertical vibration in earthquake and have high period, for instance, more than 1.5 second, bridges with high bases (higher than 30 m), bridges with high vibration modes and bridges that their displacement are very high.

-Suspension bridges

-bridges involving certain bases and body or new untested systems

Spectrum analysis and time history method can be used for dynamic analysis and their selection depends on design aims. There are two dynamic models involving pipe continuous mass model and discontinuous mass.

Required accelergraph or spectrum must be corresponded with introduced spectrum in section 1-4.

6-3-4-3-Steps of designing cable pipe line through allowable stress method

1-In seismic calculation of buried pipe line, seismic safety is controlled with displacement response method. For discontinuous pipes, rotation of connections must be controlled as well as controlling pipe expansion and contraction.

2-In problematic grounds, buried pipe must have capability of adsorbing ground displacement.

3-Displacement response method or semi-static method can be used for seismic design of shafts, coated conduits, common conduits and buffers. Response in connection to structures and buildings are controlled with performing dynamic analysis. Due to smaller of pipe conduit density than to ground and its longevity, its movement is a function of ground movement and

so must endure ground deformations.

Buried pipeline, shafts, coated conduits, buried common conduits and anchored flexible retaining walls don't move due to vibration of body but displace due to movement of surrounding soils.

Displacement response method is used for seismic loading of these conduits and their safety is tested through controlling stress and strain obtained from this method.

In unfavorable grounds such as cracked or slippery grounds, pipeline safety must be investigated in sight of adequate flexibility against these displacements.

Semi-static method is a standard method in loading of these conduits.

Displacement response methods are used for these buried parts.

In junctions and positions of changing ground specifications, relative stress, strain or displacement is produced. So in these locations, displacement response method must be used as well as dynamic analysis. In this respect, designer must consider and apply structural effects with consideration of seismic load as well as normal load.

6-3-5-Steps of pipeline calculations

6-3-5-1-Underground pipelines

6-3-5-1-1-general

1-Earthquake effects include inertial force due to structure weight (with appendix of cable weight inside pipe), soil pressure, liquefaction and lateral extension 2-Stress, deformation and stability of conduit with account of cable inside it must be controlled.

6-3-5-1-2-Design earthquake factor of underground structures

1-Horizontal earthquake factor of design, KSH in semi-static method is obtained from relation 6-1.

$$K_{SH} = \beta_4 K_H \qquad 6\text{-}1$$

$$K_H = 0.3 \cdot \beta_0 . \beta_1 \cdot \beta_2 \cdot \beta_3 \qquad 6\text{-}2$$

Where

K_{SH} Design earthquake factor based on seismic loading and analysis guideline of lifelines of Iran.

β4 magnification factor of horizontal response. This factor is obtained from seismic loading and analysis guideline of lifelines of Iran based on structure elevation.

K_H horizontal seismic intensity in ground level

β_1 importance factor (table 3-1)

β_2 basic acceleration ratio of design (table 3-3)

β_3 seismic amplification factor from bedrock to ground level according to soil type (table 3-4)

β_0 Earthquake hazard level parameter. Value 0.5 for hazard level 1 and value 1 for hazard level 2.

2-Modified horizontal earthquake factor of design K_{MH} and modified vertical earthquake factor of design K_{MV} in modified semi-static method are obtained from these relations:

$$K_{MH} = \beta_5 K_H \qquad 6\text{-}3$$

$K_{MV} = \beta_6 K_H$ 6-4

Where

β_5 amplification factor of horizontal response (guideline of lifeline seismic loading and analysis) (table 3-5)

β_6 factor of vertical response based on relation 3-11, in most cases, K_{MV} is approximately half of K_{MH}

6-3-5-1-3-Horizontal seismic load of superstructure based on Pipe Bridge

1-Seismic load involve inertia force that semi-static method is used for its determination.

2-Lateral loads due to expansion of structures in bridge are obtained on the basis of coefficient of static friction and structure weight. Coefficient of friction is usually lesser than earthquake factor.

3-Combination of above-mentioned loads must be used in designing structure.

4-Effect point of load is located in the base of structure and along the axis of bridge that is the vertical image of gravity center of structure on the base.

5-Inertia force is applied independently in two directions namely in the direction of bridge axis and perpendicular to it, according to following figure.

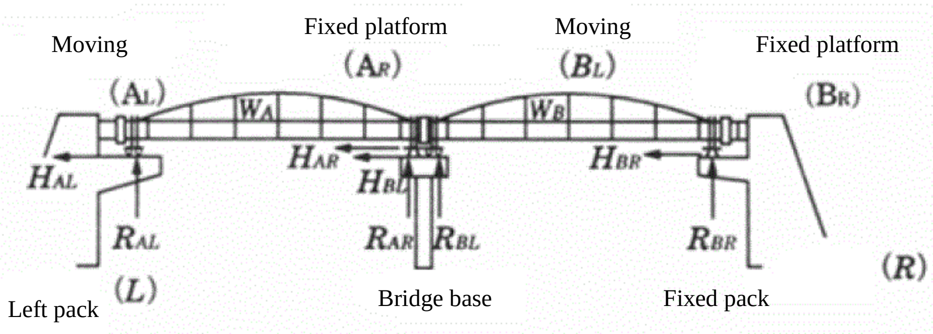


Figure 6-1-Application of inertia force

6-3-5-1-4-Horizontal inertial force

1-Applied inertial force on left pack is given by:

$$H_{AL} = R_{AL} \cdot f_{AL} \qquad 6\text{-}5$$

$$H_{AL} \le \frac{1}{2} K_{SH} \cdot W_A \qquad 6\text{-}6$$

2- Applied inertial force on bases is given by:

$$H_{AR} + H_{BL} = K_{SH} \cdot W_A (H_{BL} = 0) \qquad 6\text{-}7$$

or

$$K_{SH} \cdot W_A - f_{AL} \cdot R_{AL} + f_{BL} \cdot R_{BL} \qquad 6\text{-}8$$

$$R_{AL} \cdot f_{AL} \le \frac{1}{2} K_{SH} \cdot W_A$$

$$R_{BL} \cdot f_{BL} \le \frac{1}{2} K_{SH} \cdot W_B \qquad 6\text{-}9$$

3- Applied inertial force on right pack is given by:

$$H_{BR} = K_{SH} \cdot W_B \qquad 6\text{-}10$$

Where

K_{SH} :Horizontal earthquake factor

W_A, W_B Dead weight of superstructure A and B (kN)

R_{AL}, R_{AR} Reaction of bridge bases and bridge platform (L) due to W_A (kN)

R_{BL}, R_{BR} Reaction of bridge bases and bridge platform (L) due to W_B (kN)

H_{AL}: Inertial or frictional force applied on bridge platform (L) due to W_A (kN)

H_{AR} :Inertial force applied on bridge bases due to W_A (kN)

H_{BL}: Inertial or frictional force applied on bridge platform (L) due to W_B(kN)

H_{BR}: Inertial force applied on bridge bases due to W_B (kN)

f_{AL}: Coefficient of static friction of extensional support A_L

f_{BL}: Coefficient of static friction of extensional support B_L

Inertial force in perpendicular direction to bridge axis is obtained from multiplication of dead load to horizontal earthquake factor.

6-3-5-2-Steps of buried pipeline calculations

6-3-5-2-1-Ground strain

In displacement response method, ground strain in the pipeline is given by:

$$\varepsilon_G = \frac{\pi U_h}{L} \qquad 6\text{-}11$$

Where

ε_G Ground strain

U_h Horizontal displacement of the ground in the pipeline direction

L Wavelength

π Pi number

6-3-5-2-2-Coefficient of soil hardness in allowable stress method

In seismic calculations through allowable stress method, sliding between soil and pipeline isn't considered. Coefficient of soil hardness in the pipe direction and perpendicular to it, K_{g1} and K_{g2} is calculated according to relations 6-12 and 6-13 that are as following:

$$K_{g1} = C_1 \frac{\gamma_t}{g} V_S^2 \qquad 6\text{-}12$$

$$K_{g2} = C_2 \frac{\gamma_t}{g} V_S^2 \qquad 6\text{-}13$$

where

K_{g1}, K_{g_2} : Coefficients of soil hardness in length unit in the pipe direction and perpendicular to it in terms of Pa.

γ_t : Soil specific gravity

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

V_S :velocity of elastic shear wave in ground level (m/s)

C_1 , C_2 : coefficient in two directions and usually are assumed to be C_1=1.5 and C2=3.

These factors are obtained from finite element method more accurately. Following relation is applied for determination of these coefficients. in this relation, layer thickness is varied between 5 and 30 m and pipe diameter is varied between 150 and 3000 mm.

$$C_1 = 1.3H^{-0.4}D^{0.25} \qquad 6\text{-}14$$

$$C_2 = 2.3H^{-0.4}D^{0.25} \qquad 6\text{-}15$$

Where

H: thickness of surface layer (m)

D: pipe diameter (m)

6-3-5-2-3-Frictional force of ground in ductile design method

In ductile design method, relative sliding of pipe and soil is considered to be about 0.01 Mpa with consideration of friction.

In peculiar and important pipes, nonlinear model is considered for friction.

6-3-5-2-4-Soil hardness for multilayer conduits

In multilayer conduits, equivalent hardness is used that involves axial and bending hardness.

Axial hardness is the sum of hardness of conduits in their gravity centers and bending hardness is the sum of bending harnesses with consideration of two crossing lines from gravity centers that is shown in figure 6-2.

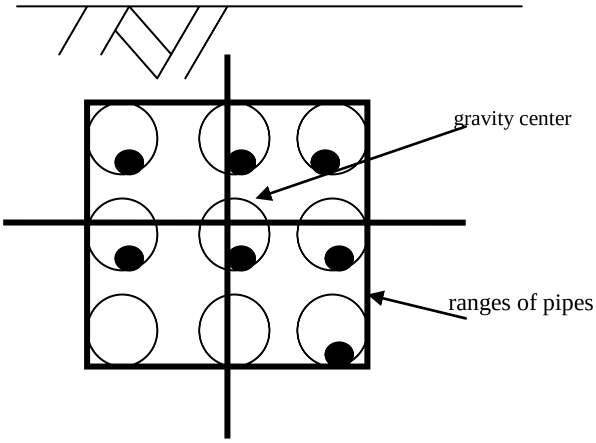


Figure 6-2-Equivalent harness of pipe and soil

6-3-5-3-Seismic calculations related to continuous pipe through displacement response method

6-3-5-3-1-General

In calculation of strain and stress through allowable stress method, sliding between pipe and soil is neglected but this sliding is accounted in ductility design method.

6-3-5-3-2-Calculation of continuous pipe body stress in allowable stress method

Stress in pipe body through allowable stress method can be obtained from relation 6-16:

$$\sigma_{1L} = \alpha_1 \frac{\pi U_h}{L} E \qquad 6\text{-}16$$

$$\sigma_{1B} = \alpha_2 \frac{\pi^2 D U_h}{L^2} E \qquad 6\text{-}17$$

$$\sigma_{1x} = \sqrt{\alpha_{1L}{}^2 + \alpha_{1B}{}^2} \qquad 6\text{-}18$$

Where

σ_{1L} Axial stress of buried pipe (Pa)

σ_{1B} Bending stress of buried pipe (Pa)

σ_{1x} Combinational axial and bending stress (Pa)

α_1, α_2 Coefficient of ground displacement inversion in axial and transversal direction of pipe

$$\left.\begin{aligned} \alpha_1 &= \frac{1}{1+\left(\frac{2\pi}{\lambda_1 L'}\right)^2} \\ \alpha_2 &= \frac{1}{1+\left(\frac{2\pi}{\lambda_2 L}\right)^4} \end{aligned}\right\} \qquad 6\text{-}19$$

$$\left.\begin{aligned} \lambda_1 &= \sqrt{\frac{K_{g1}}{EA}}\,(1/m) \\ \lambda_2 &= \sqrt[4]{\frac{K_{g2}}{EA}}\,(1/m) \end{aligned}\right\} \qquad 6\text{-}20$$

Where

L´ : wavelength (=√2L) in terms of meter

L : wavelength (m)

K_{g1}, K_{g2} :coefficient of soil hardness in unit of length in the direction of pipe direction and perpendicular to it (Pa)

E : elastic module of buried pipe (Pa)

A: cross section of buried pipe (m^2)

I: inertial moment of buried pipe section (m^4)

U_h: horizontal displacement of the ground

D: external diameter of buried pipe (m)

6-3-5-3-3-Computation of continuous pipe body stress in design through ductility method

1-Axial stress of pipe is given by:

$$\sigma_{2L} = \frac{\pi D \tau L'}{4A} \qquad 6\text{-}21$$

Where

σ_{2L} axial stress (Pa)

τ frictional stress between pipe and ground (Pa)

2-Bending and combinational stresses are calculated according relations 6-17 and 6-18.

3- σ_{2L} is multiplied in a number between 1 and 3.12 according to importance level

6-3-5-3-4-Calculation of continuous pipe body strain in allowable stress method

Calculation of continuous pipe body strain in allowable stress method is performed through following relations:

$$\varepsilon_{1L} = \alpha_1 \cdot \varepsilon_G \qquad 6\text{-}22$$

$$\varepsilon_{1B} = \alpha_2 \cdot \frac{2\pi D}{L} \varepsilon_G \qquad 6\text{-}23$$

$$\varepsilon_{1x} = \sqrt{\varepsilon_{1L}^{\ 2} + \varepsilon_{1B}^{\ 2}} \qquad 6\text{-}24$$

Where

ε_{1L} axial strain of pipe

ε_{1B} bending strain of pipe

ε_{1x} combinational axial and bending strain of pipe

ε_G axial strain of ground

α_1 coefficient of axial strain transmission of ground

$$\alpha_1 = \frac{1}{1 + \left\{2\pi / \left(\lambda_1 L'\right)\right\}^2} \qquad 6\text{-}25$$

$$\lambda_1 = \left\{K_{g1} / (EA)\right\}^{1/2} \qquad 6\text{-}26$$

where

α_2 coefficient of transversal strain transmission of ground similar to relation 6-19

ε_{1L} Value in elastic mode is calculated through above-mentioned method. However, when strain is exceeding from yield strain of pipe ε_y, α_1 value is modified with coefficient $\lambda_1 = \left[\left(K_{g1} / \left\{\varepsilon_y / \left(2\varepsilon_{1L}\right) EA\right\}\right)\right]^{1/2}$ and strain of pipe ε_{1L} is obtained.

Relations 6-16 to 6-18 can be used to control stress sufficiency.

6-3-5-3-5-Calculation of continuous pipe stress in ductile design

1-Axial strain ε_{2L} is given by

$$\left.\begin{aligned} \varepsilon_{2L} &= L/\zeta \quad (L \le L_1) \\ \varepsilon_{2L} &= L/(\kappa\zeta) \quad (L_1 < L < L_2) \end{aligned}\right\} \qquad 6\text{-}27$$

$$\left.\begin{aligned} \varepsilon_{2L} &= \varepsilon_{G\max} \quad (L_2 = L) \\ L_2 &= \kappa\zeta\left\{\varepsilon_{G\max} - (1 + 1/\kappa)\varepsilon_y\right\} \end{aligned}\right\} \qquad 6\text{-}28$$

where

ε_{2L} pipe axis strain and $\zeta = 2\sqrt{2}\,Et/\tau$
t pipe thickness (m)
k pipe strain hardening specification (k=0.1)
ε_y pipe yield strain
$\varepsilon_{G\max}$ ground strain corresponding to $S_{V\max}$ (if $0.7 \le T_G$, then S_V' is used)
2-Bending and combinational strain is computed similar to relations 6-24 and 6-25.

6-3-5-4-Calculation of discontinued pipe through displacement response

6-3-5-4-1-General

1-Discontiued pipes are generally designed through ductile design method. In seismic hazard level 2, sliding between pipe and soil is considered.
2-conncetion displacement and bending angle of pipe must be computed according to maximum horizontal displacement of ground.

6-3-5-4-2-Calculation of discontinued pipe stress without consideration of sliding (allowable stress method)

With neglecting relative sliding between pipe and soil, pipe body stress is obtained as following:

$$\sigma'_{1L}(x) = \zeta_1(x)\cdot\sigma_{1L} \quad 6\text{-}29$$

$$\sigma'_{1B}(x) = \zeta_2(x)\cdot\sigma_{1B} \quad 6\text{-}30$$

$$\sigma'_{1x}(x) = \sqrt{\{\sigma'_{1L}(x)\}^2 + \{\sigma'_{1B}(x)\}^2} \quad 6\text{-}31$$

Where
$\sigma'_{1L}(x)$, $\sigma'_{1B}(x)$ bending and axial stresses of pipe
$\sigma_{1L}(x)$, $\sigma_{1B}(x)$ axial and bending stresses
$\sigma'_{1x}(x)$ combinational stresses in distance X from flexible connection
$\zeta_1(x), \zeta_2(x)$ modification factor of buried pipe stress with assumption of connection continuation (as it can be considered discontinuous pipe as continuous with assumption of solidity of connection)
It is preferred that term $[\sigma'_{1L}(x)]^2$ in relation 6-31 to be multiplied with a number between 1 and 3.12 according to importance level

6-3-5-4-3-Calculation of discontinuous pipe body stress with consideration of sliding (ductile method)

Axial stress σ'_{2L} is calculated with consideration of sliding through simplification of non-linear response analysis results. In the case of soft metallic pipe under intermediate or intense earthquake, axial stress is given by relation 6-32.

$$\sigma_L = \frac{\pi\cdot D\cdot\tau\cdot l}{2A} \quad 6\text{-}32$$

Where
σ_L axial stress (Pa)
l pipe length (m)

A pipe cross section (m^2)
Bending and combinational stress are obtained similar to relation 6-30 and 6-31.

6-3-5-4-4-Calculation of discontinuous pipe strain without consideration sliding

Axial strain of pipe with flexible connection under earthquake of hazard level 1 is given by

$$\varepsilon'_{1L}(x) = \zeta_1(x)\cdot\varepsilon_{1L} \quad 6\text{-}33$$

$$\varepsilon'_{1B}(x) = \zeta_2(x)\cdot\varepsilon_{1B} \quad 6\text{-}34$$

$$\varepsilon'_{1x}(x) = \sqrt{\{\varepsilon'_{1L}(x)\}^2 + \{\varepsilon'_{1B}(x)\}^2} \quad 6\text{-}35$$

where

$\varepsilon'_{1L}(x), \varepsilon'_{1B}(x)$ axial and bending strain in distance X from connection

$\varepsilon_{1L}, \varepsilon_{1B}$ bending and axial strain from relations 6-29 and 6-30

$\varepsilon'_{1x}(x)$ combinational bending and axial strain

$\zeta_1(x), \zeta_2(x)$ modification coefficient of stress with assumption of continuous pipe

6-3-5-4-5-Calculation of discontinuous pipe body with consideration of sliding

axial strain ε'_{2L} is given by

$$\varepsilon'_{2L} = \frac{\tau L_e}{2Et} \quad 6\text{-}36$$

Where

L_e distance of flexible connections (m)

t pipe thickness (m)

Bending and combinational strain are computed similar to relation 6-34 and 6-35.

6-3-5-4-6-Calculation of axial displacement of connection

Axial displacement of connection is given by:

$$|u_J| = u_0\overline{u_J} \quad 6\text{-}37$$

Where

$|u_J|$ axial expansion of connection (m)

u_0 axial relative displacement with assumption of infinite length of pipe

$$\overline{u_J} = \frac{2\gamma_1|\cosh\beta_1 - \cos\gamma_1|}{\beta_1\sinh\beta_1} \quad 6\text{-}38$$

$$u_0 = \alpha_1 U_a \quad 6\text{-}39$$

$$\alpha_1 = \frac{1}{1+(\gamma_1/\beta_1)^2} \quad 6\text{-}40$$

$$\beta_1(=\lambda_1 l) = \sqrt{\frac{K_{g1}}{EA}}\cdot l \quad 6\text{-}41$$

$$\gamma_1 = \frac{2\pi l}{L'} \qquad 6\text{-}42$$

Where

EA axial hardness (N)

l distance of connection (m)

K_{gl} coefficient of soil hardness in axial direction (Pa)

L´ resultant of wavelength (=√2L)(m)

L wavelength (m)

Ua horizontal displacement of ground in the direction of pipe axis (m)

$$U_a = \frac{1}{\sqrt{2}} U_h \qquad 6\text{-}43$$

Where

U_h horizontal displacement in depth *x(m)* from ground level

Relation 6-44 is used for cast iron pipe under intermediate to intense earthquake:

$$e_p = \varepsilon_G l \qquad 6\text{-}44$$

e_p expansion of connection in axial direction

ε_G ground strain

6-3-5-4-7-Calculation of connection bending angle

Connection bending angle is calculated as following:

$$\theta = \frac{4\pi^2 l U_h}{L^2} \qquad 6\text{-}45$$

Where

θ bending angle of connection (Rad)

U_h displacement

Above relations are obtained with assumption of harmonized movement of pipe with ground

6-3-6-Safety evaluation of pipe under continuous displacement of ground

6-3-6-1-General

Permanent displacement of ground is accounted as considerable hazard for pipelines. Maximum strains of ground that are formed in boundary areas are a criterion for determination of maximum displacement of ground. In liquefaction mode, this boundary is the contact of liquefactive and non- liquefactive layers. In faults, maximum displacement is happen in location of fault itself and ground fracture. In sliding mode, displacement is also occurring in slope margins and its maximum value in subsidence mode depends on subsidence pattern.

6-3-6-2-Measures against permanent ground displacement (PGD)

In direct pipes, extensional connections are applied to control pipe deformations and displacements.

In connection of pipe to structures which there is a possibility of occurring high bending, extensional connections must have capacity to absorb higher displacements and rotations.

Extensional and integral connections (extensional displacement: pipe length $n \pm \beta\%$) must be used to prevent displacement and rupture of direct pipelines.
For sections that bending force is applied on them (such as opening of buildings and connection box), extensional ball connection of high flexibility (expansion volume: pipe length M±α [bending angle deg±θ]) must be used.

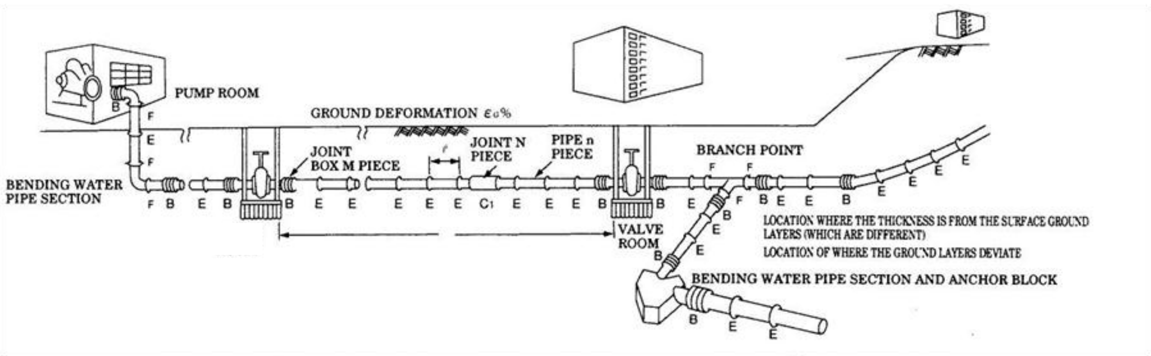


Figure 6-3-Conceptual design of connected pipeline

6-3-6-3-Calculations of discontinuous pipeline under liquefaction

To ensure safety of pipe with length L, axial displacement capacity of pipe must be higher than maximum ground displacement, i.e. $\varepsilon_G L$.

$$\varepsilon_G L < n\beta l + Ma \qquad 6\text{-}46$$

When above condition isn't satisfied, pipe flexibility must be increased with addition of extensional connections. number of these connections is obtained by following relation:

$$N = \frac{\varepsilon_G L - n\beta l + Ma}{b} \qquad 6\text{-}47$$

where

E expansion (β%±percent of pipe length)

b bending angle of connection $° \pm \theta, \pm \alpha$

C_1 expansion of long pipe

Endurable force of each connection must be more than transferred frictional force to pipe. Required controls must be made in perpendicular direction to pipe to endure deformations. Following figure shows function of extensional discontinuous connections.

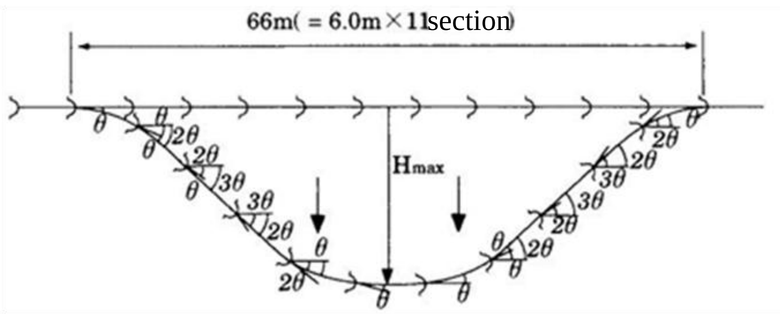


Figure 6-4-Behaviour of connections in perpendicular direction to pipe axis

$$H_{max} = l(\tan\theta + \tan 2\theta + \tan 3\theta + ... + \tan 3\theta + \tan 2\theta + \tan\theta) \qquad \text{6-48}$$

Where:

θ maximum bending angle of each connection in terms of degree

6-3-6-4-Computation of continuous pipe under liquefaction

6-3-6-4-1-Lateral deformation of ground adjacent to retaining walls

In the case of occurring lateral ground deformation and axial deformation in pipe, maximum strain of pipe is given by relation 6-49:

$$\varepsilon_P = \frac{\tau' L}{Et} \qquad \text{6-49}$$

Where

ε_P axial strain of pipe

τ' frictional stress of ground in liquefaction mode (Pa)

L displaced length

E elastic modulus steel (Pa)

t pipe thickness (m)

When ε_P is higher than following value, its value must be recalculated by relation 6-50.

$$\varepsilon_P = \frac{\tau' L}{\kappa Et} + \left(1 - \frac{1}{\kappa}\right)\varepsilon_y \qquad \text{6-50}$$

Where

k hardening of strain in extension (k=0.01)

6-3-6-4-2-Lateral ground deformation in slopes

In shelvy grounds in which pipe is located in perpendicular direction to slope, bending angle of direct pipe is given from relation 6-51 in terms of degree:

$P_1 = D_i \cdot \gamma_k \cdot \sigma_c$ for

$$\omega_s = \frac{180}{\pi} \cdot 127 \cdot D_i \cdot \sqrt{\frac{P_1 \cdot \gamma_\delta \cdot \delta_h}{EI}} \qquad \text{6-51}$$

Where

γ_δ ‘γ_k ‘σ_c partial safety factors for permanent ground deformation δ_h, soil springy K_1 and compression stress of soil, respectively

ω_s bending angle of pipe in degree

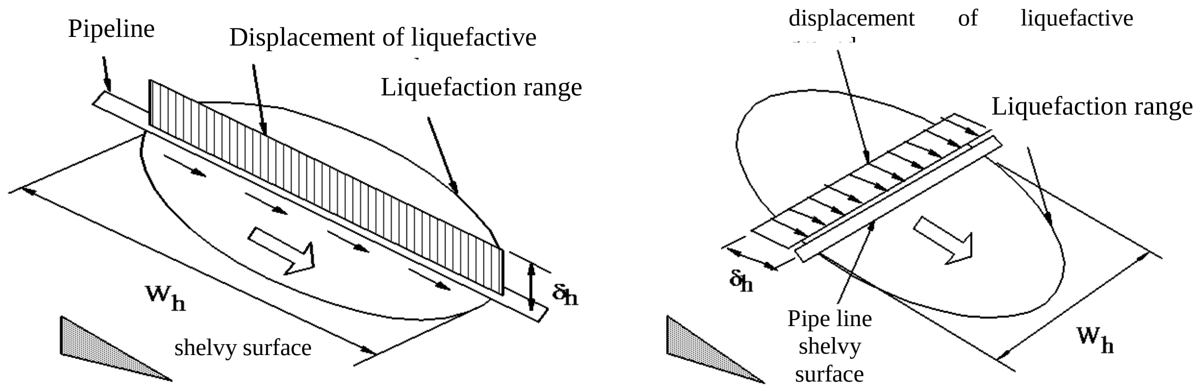


Figure 6-5-Analytical model of direct pipe in liquefactive shelvy ground

6-3-6-4-3-Evaluation of seismic design under permanent displacement due to liquefaction

1-In this condition, seismic safety is examined with probabilistic estimation of pipe limit state under permanent displacement due to liquefaction.

2-This evaluation method is used for direct pipes and bents.

3-Partial load safety factor, structural analysis, material specifications and structural components are applied separately.

Usual values of these partial safety factors are presented in table 6-2.

Table 6-2-Partial safety factor

<table>
<tr><th colspan="3">Partial safety factor</th><th>standard value</th><th>displacement mode</th><th>components</th></tr>
<tr><td rowspan="6">Load actor</td><td rowspan="3">γ_d for displacement</td><td>ramp</td><td>1.8</td><td rowspan="3">all modes</td><td rowspan="3">pipe and bent</td></tr>
<tr><td>bulwark</td><td>1.2</td></tr>
<tr><td>subsidence</td><td>1</td></tr>
<tr><td rowspan="3">γ_k for ground resistance force</td><td>axial</td><td>1.2</td><td rowspan="3">all modes</td><td rowspan="3">pipe and bent</td></tr>
<tr><td>transversal</td><td>1.2</td></tr>
<tr><td>vertical</td><td>1.1</td></tr>
<tr><td rowspan="4">factors related to structural analysis</td><td rowspan="4">γ_a for simplification of calculative relations</td><td>analytical relation</td><td>1</td><td>axial and bending buckling</td><td>pipe and bent</td></tr>
<tr><td rowspan="2">non-elastic beam analysis by FEM method</td><td>1.1</td><td>internal bending and external tension</td><td>pipe</td></tr>
<tr><td>1</td><td>internal bending and external elongation</td><td>bent</td></tr>
<tr><td>combinational analysis of beam and shell</td><td>1</td><td>all modes</td><td>pipe and bent</td></tr>
<tr><td>material factors</td><td colspan="2">γ_m for limit state strain of pipe material</td><td>1</td><td>all modes</td><td>pipe and bent</td></tr>
<tr><td rowspan="2">component factors</td><td colspan="2" rowspan="2">γ_b for formulating critical displacement of pipe</td><td>1</td><td>axial and bending buckling</td><td>pipe</td></tr>
<tr><td>1</td><td>internal bending and external tension</td><td>bent</td></tr>
</table>

6-3-6-5-Calculation of pipe strain under fault displacement effect

pipe strain under permanent displacement due to fault is given by relation 6-52:

$$\varepsilon_{pipe} = 2\left[\frac{PGD}{2L_a}\cos\beta + \frac{1}{2}\left(\frac{PGD}{2L_a}\sin\beta\right)^2\right] \qquad 6\text{-}52$$

where, β and L_a are applied displacement angle with orientation of pipe axis and pipe effective length under displacement

In the case of crossing communication pipeline from fault, following consideration must be applied:

1-If calculations prove that designed pipe can endure fault displacement securely; pipe standard design can be applied.

2-Unless location-specific studies must be performed that can be lead to increment of pipe thickness, toughness increment, modifications of welding, modification of embanking method, burial of underground pipe with making embankment, designing supporting structure and modification of supports.

Ground displacement, surficial topography, wide of faulted area, soil conditions, status of erosion and drainage, environmental effects, contiguity with other structures and related costs must be considered in design.

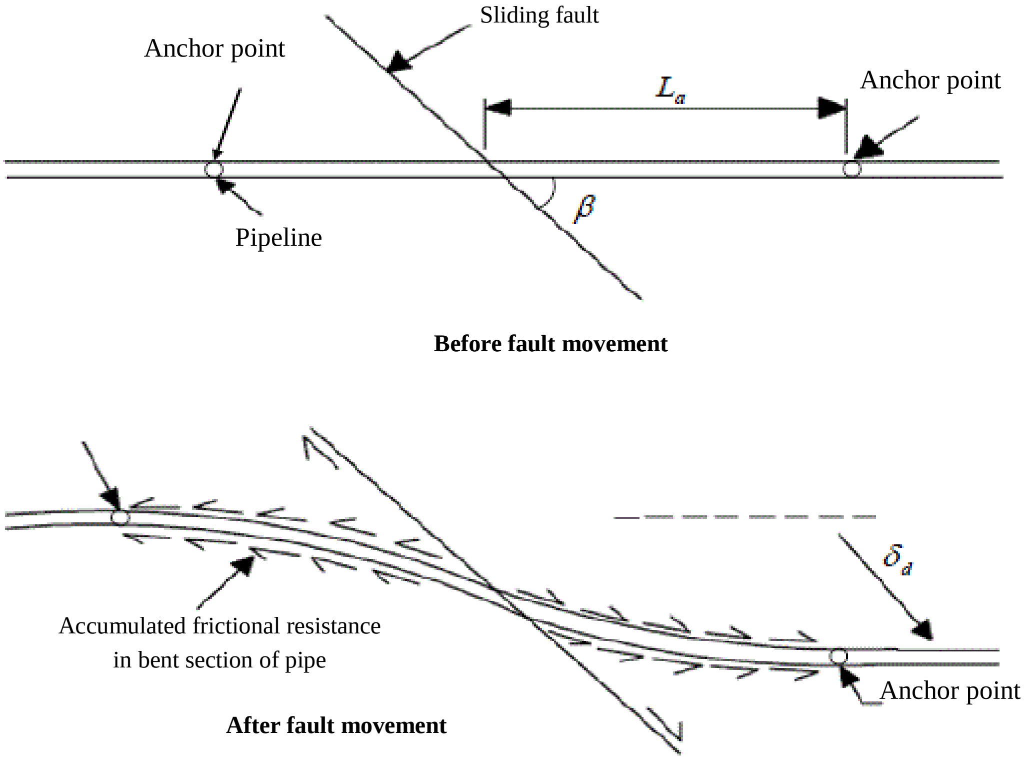


Figure 6-6-Intersection of pipe and fault

6-3-6-6-Calculations related to slope sliding

Permanent ground displacement (PGD) along slope is considered and pipe behavior depends on its position angle. Pipe which is positioned along PGD displacement is undergone axial deformation. Diagonal pipe is undergone bending.

Equivalent static method (ESM) cannot be used in these conditions. As usual, PGD inflict more damage to pipe axially and its value is experimentally, 5 to 10 times higher, because pipe under bending show more ductility under compression or extension.

ESM is applicable for pipes under small displacements but in big displacements, e.g. about 30 cm and higher, finite element analysis must be performed more accurately.

6-3-6-6-1-Pipe response to longitudinal permanent displacements

Axial displacement in pipe can make extension in upper parts of slope and compression in its lower part (points A and B in figure (6-7)).

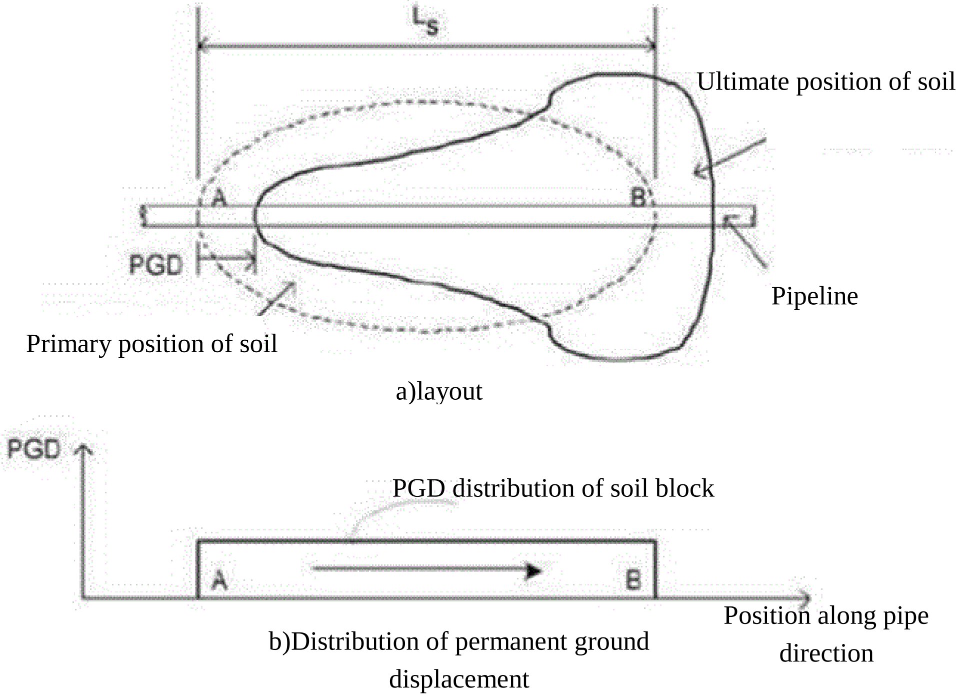


Figure 6-7-Buried pipe response to longitudinal PGD

Minimum value among F1 and F2 is considered to be design force in which F1 is in elastic range of pipe and fully embraced by soil while F_2 is ultimate transferable force between pipe and soil.

$$F = \min(F_1, F_2) \qquad 6\text{-}53$$

$$F_1 = \sqrt{A_r E t_u \delta} \qquad 6\text{-}54$$

where

δ Is PGD displacement, A_r is pipe area, E is elastic module and t_u is thickness.

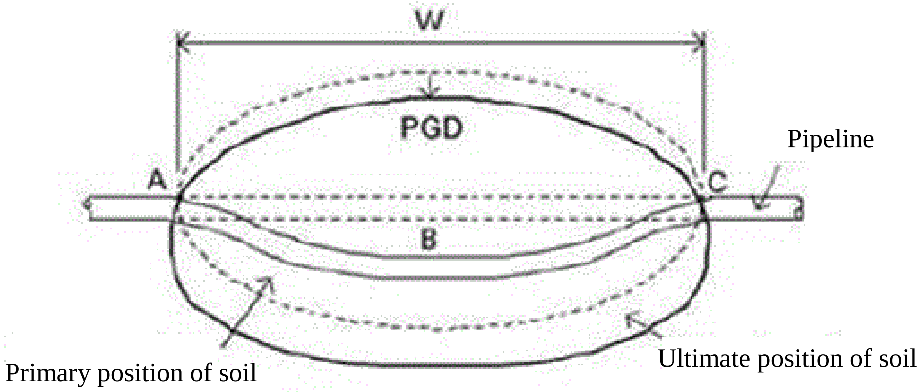
$$F_2 = \frac{t_u L_{sp}}{2} \qquad 6\text{-}55$$

Where L_{sp} is pipe length under displacement.

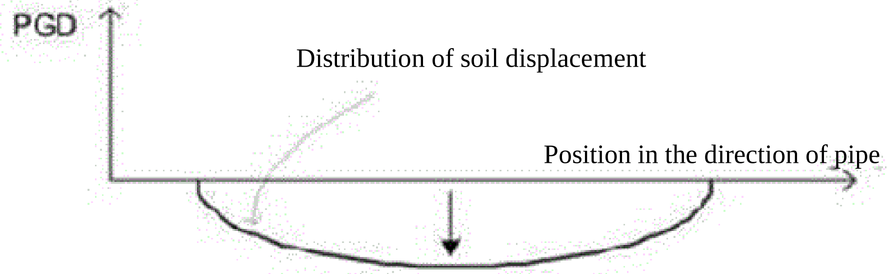
When elastic design isn't applicable, plastic method can be used. In this condition, pipe must be made of ductile materials and their connections must have capability to endure applied force.

6-3-6-6-2-Pipe response to transversal displacement

in this case, pipe act as a beam under maximum displacement in the middle of span. Distributed PGD involve maximum displacement in nearby of center and minimum displacement in adjacent of soil mass boundaries. The worst case is in nearby of soil mass boundaries such as a pipe passing across a fault. Effect point of maximum displacement is gravity center of displaced soil mass. Determination of boundary positions in effective design and in cases where PGD occur in nearby of soil mass boundaries is important and it is needed that investigation site-specific hazards to be performed.



a)layout



b) Distribution of permanent ground displacement

Figure 6-8-Pipe response to transversal displacement

maximum strain of pipe is produced in the middle or in the end of bended span and its minimum value is given conservatively by relation 6-56:

$$\varepsilon_b = \pm \frac{\pi D_o \delta}{W_G{}^2} \qquad 6\text{-}56$$

Where, W_G is width of soil mass, D_0 is external diameter of pipe and δ is ground displacement.

When additional strain may lead to squeezing or buckling, it is recommended to perform finite element analysis more accurately.

So, W and δ values must be obtained according to location conditions.

This method allows pipe to endure plastic deformation, however pipe material must have capability of its endurance.

There is at least 4 to 5% of this capacity in extension and 1% of it in compression and so, connections must have sufficient strength against it.

6-3-6-6-3-Specefic condition for pipes

Finite element analysis must be used for complex systems

One instance of these systems in equivalent spring model in longitudinal, transversal and vertical directions.

6-3-7-Allowable values related to buried pipeline

1-Allowable pipe displacement or strain is considered to be maximum tolerable connection.

2-In earthquake of hazard level 1 for wave effect, critical strain in fatigue mode with low cycle is considered to be minimum two values of 0.11% and buckling limit strain 23t/D.

3- In earthquake of hazard level 2 for wave effect, critical strain in fatigue mode with low cycle is considered to 46t/D.

4-Endurable displacement of mechanical connections such as extensional, spiral and male and female connections is obtained through test.

Above-mentioned allowable critical values are presented in table 6-3.

Table 6-3- Allowable critical values

Earthquake	Seismic load	Unit	Rupture mode	Components	Criterion
Hazard level 1	Wave effect	strain	rupture due to fatigue with low cycle	Continuous pipe	0.11%
			buckling		$23t/D$
Hazard level 2	Wave effect		buckling	Continuous pipe	$46t/D$

6-3-8-Acceptance criterion

6-3-8-1-Acceptance criterion of underground pipe

Acceptance criterion of bridge pipe is given by:

$$Q_p \le Q_{pa} \quad \text{6-57}$$

Where

Q_p response load related to failure modes

Q_{pa} allowable load related to failure modes

6-3-8-2-Acceptance criterion of buried pipeline

6-3-8-2-1- Discontinuous pipe in design through ductile method without consideration of relative sliding between pipe and soil

1-Pipe body stress must be lower than or equal to allowable stress of yield or shrinkage

2-Displacement must be lower than or equal to allowable displacement of primary design

6-3-8-2-2- Discontinuous pipe in design through ductile method with consideration of relative sliding between pipe and soil

1-Stress: Pipe body stress must be lower than or equal to allowable stress of yield or squeezing.

2-Connection displacement: displacement must be lower than or equal to allowable displacement of primary design

6-3-8-2-3-Acceptance criterion of continuous pipe in allowable stress method

Stress: Pipe body stress must be lower than or equal to allowable stress.

6-3-8-2-4-Acceptance criterion of continuous pipe in ductile method

Strain: pipe strain must be lower than or equal to allowable strain.

6-4-Seismic design consideration of communication anchored flexible conduits (tunnels)

In loose grounds in which drilling is difficult, flexible anchored tunnels is used in depth.

In these cases, flexible anchored tunnels are used for transferring main communication lines, especially as common conduits.

Modeling and analysis method is very important in seismic designing of these conduits.

Analytical models of flexible anchored conduits are introduced in the following.

These models are considered as a beam on continuous spring with monotonous hardness in length. However, there is no comprehensive method for tunnels with blocky coating.

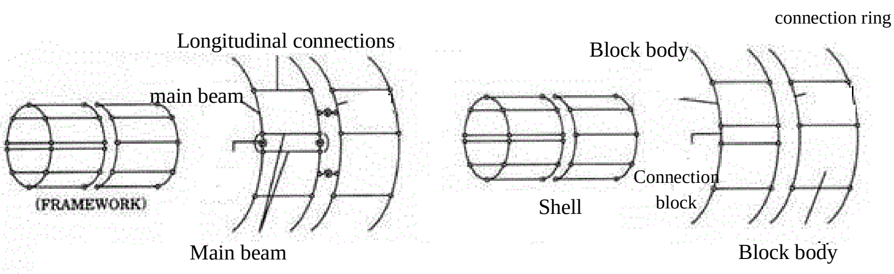
1-First model (the Koizumi model)

Joints are modeled in this method and model parameters are obtained on the basis of test.

Joint of blocks are considered with rotary springs and joints of rings are considered with similar materials.

Three-dimensional frame elements are used for metallic conduits according to the following figure.

Shell elements are used for concrete conduits.



Spatial frame for metallic coating **Spatial shell for concrete coating**

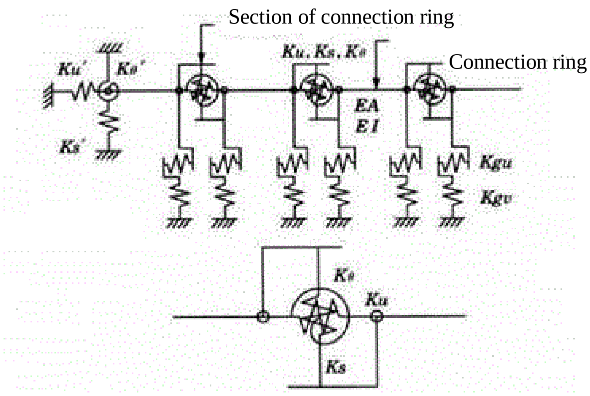


Figure 6-9-Cylindrical model of tunnel coating

2-Second model (the Shiba model)

In this model, coating is considered as continuous beam (with hardness equivalent with connection springs) that is monotonous with strain load relation.

According to the above figure, bending and extensional equivalent hardness is considered for block connection.

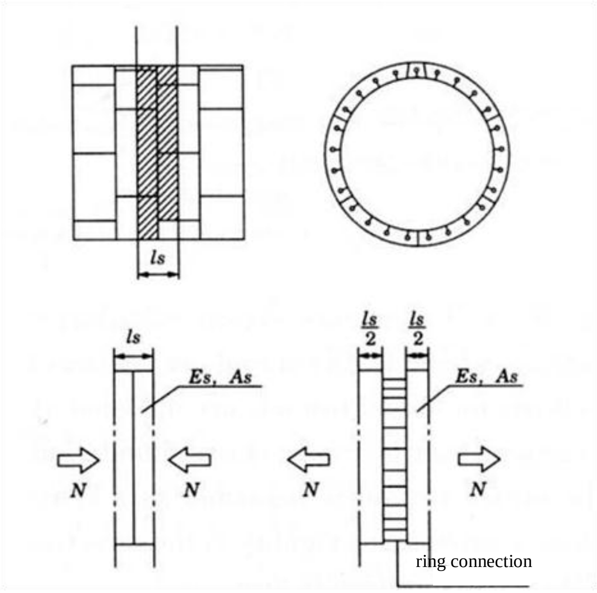


Figure 6-10-Calculation of axial equivalent hardness

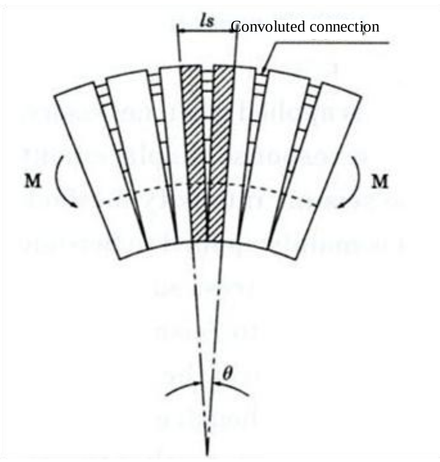


Figure 6-11-Mechanism of connection deformation of blocks under bending

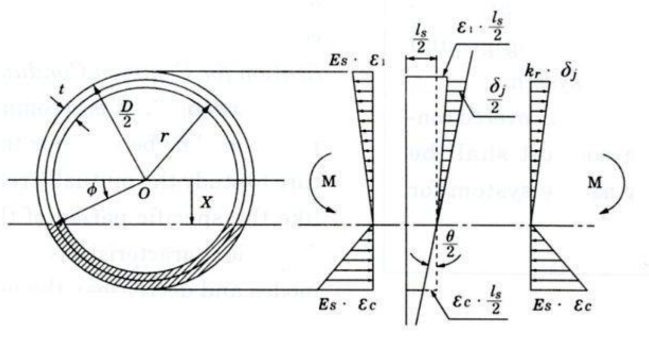


Figure 6-12-Combination of stresses and displacements in blocks and connections (extensional force is in elastic range)

6-5-Calculations of common and anchored flexible conduits through dynamic method

– Linear dynamic analysis is done for earthquake of hazard level 1 and non-linear analysis is done for hazard level 2.
– -in total analytical model, structure, relevant parts and surrounding soils must be modeled altogether and in partial analytical model, flexible anchored conduits must be modelled as separate systems.

1-if application of displacement response method and semi-static method isn't possible, dynamic analysis method must be used.

There is two dynamic analysis method including total analysis of a collection and its partial analysis.

In total analytic model, related parts and surroundings soils are modeled together and obtained results are used in total design of structure.

In second method, movements of various parts of structure due to ground movement are investigated separately. Linear analysis is usually used for safety control in hazard level 1 and non-linear analysis is used in hazard level 2 with consideration of functional range of structure.

Both spectrum analysis and time history methods can be used.

2-Since accessories of structure have complex behavior during earthquake, it is recommend to use dynamic analysis in order to examine effects of ground predominant period, dynamic specifications (such as modes and damping) of main structure and its accessory.

Since structure movement and surroundings soils in major earthquake such as hazard level 2 have non-linear behavior, relevant analysis method must be considered.

Often, two-dimensional spring model is used.

If natural setting of ground surroundings structure is complex, three-dimensional finite element analysis model must be used.

In these analyses, non-linear behavior must be considered in model.

6-6-Seismic design consideration of communication shaft (vertical conduit)

1-Seismic calculations of shaft are performed through semi-static and displacement response methods.
2-If needed, dynamic analysis is applicable.
3-If hardness of structure connected to shaft is approximately same as its hardness, specification of that structure also must be considered in seismic calculations.

Elastic method is used in designing shaft section for hazard level 1.

A method that can be secure sufficient safety against earthquake stresses is used in designing lateral faces of shaft.

All three mentioned methods can be used for hazard level 2.

If hardness of shaft connections is negligible, there aren't considered in design.

Lateral load of shaft is given by relation 6-5.

$$q_H(x) = k_H(x)\{u_H(x) - u_H(h_B)\} \qquad 6\text{-}58$$

Where

$q_H(x)$ Horizontal pressure in depth x, (N/cm2)

$k_H(x)$	Factor of horizontal reaction, (N/cm3)
$u_H(x)$	Horizontal displacement of ground in depth x (cm)
$u_H(h_B)$	Horizontal displacement of ground in depth hB (cm)

This force is applied when ground impose pressure on shaft.

When ground pull shaft, there are three methods for calculation of load depends on condition of soil and shaft interface.

In the first method, displacement is assumed to be negligible and above-mentioned relation is applicable.

In the second method, soil tension is neglected.

In the third method, parts of soil pressure impose on shaft as reaction.

It is designer that selects the safest method.

When connection components of shaft have considerable hardness, dynamic analysis method is used to examine their behavior.

6-7-Seismic design considerations of buried cable (direct burial without conduit)

Cable seismic analysis (cables that directly or via pipe medium, are buried on soil) is performed on the basis of pipe displacement or applied strain of ground on cable.

6-8-Seismic design consideration of communication cables

Communication cables that are buried directly are more venerable against permanent ground displacements especially ones due to earthquake than similar type in conduits.

For seismic designing of these cables, it is required to note following consideration in order to seismic design being more conformable to cable behavior:

In occurrence of high bending, cable may be fractured.

In the case of high tension, cable pullout is possible.

In the intersection with fault, high tension may be imposed to cable.

In liquefaction, especially when cable is connected to structure, high tensile strain may be produced in cable.

Relative movement of adjacent structures may also deform communication cables, dramatically.

In occurrence of sliding, cable extrusion happens as well.

Conditions as following figure may be occurred for cables.

Rising and rotation of liquefied ground in manholes lead to imposition of high strain in the cable that is connected to it.

6-9-Steps of designing buried cables

6-9-1-Steps of designing buried cables against ground deformation

1-Elongation

Since buried cable has low bending hardness and its failure mode is predominantly occur under tension, critical cable elongation is obtained as following:

$$\Delta L = \int_0^W \sqrt{1+\left(\frac{df}{dx}\right)^2}\,dx - W \qquad 6\text{-}59$$

Where, $f(x)$ is the cable shape figure and W is its primary length.

$$\varepsilon = \frac{\Delta L}{W} \qquad 6\text{-}60$$

2-Wave propagation

Wave propagation in ground surface cause deformation in buried structures.

Anyway, cable strain is lower than ground strain and so can be used to control cable behavior to provide safety against ground strain.

$$\varepsilon_{cable} \approx \varepsilon_G \qquad 6\text{-}61$$

3-Intersection with fault

cable strain due to fault movement is obtained as following:

$$\varepsilon_F = \frac{d}{L} \qquad 6\text{-}62$$

Where, *d* and *L* are fault displacement and cable effective length respectively.

$$d = \frac{h}{2\sin\left(\frac{\theta}{2}\right)} \qquad 6\text{-}63$$

$$L = \sqrt{\frac{2E_2 d}{q} + \left(\frac{\sigma_1 - \sigma_o}{q}\right)^2} - \frac{\sigma_1 - \sigma_o}{q} \qquad 6\text{-}64$$

Where, q is sliding strength in length unit and E_2, σ_0 and σ_1 are secondary module and critical stresses according to figure 6-13.

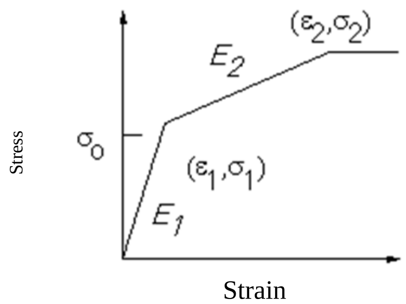


Figure 6-13-Stress-strain curve of cable element

4-Liquefaction

Maximum ground strain due to liquefaction can be considered as maximum strain in cable:

$$\varepsilon_L = \varepsilon_{axial} \qquad 6\text{-}65$$

Where, ε_{axial} is maximum ground strain.

$$\varepsilon_{axial} = \max\{\varepsilon_x, \varepsilon_y\} \qquad 6\text{-}66$$

$$\begin{pmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{pmatrix} =$$

$$\frac{1}{2ab}\begin{bmatrix} b(1+\eta) & -b(1+\eta) & -b(1-\eta) & b(1-\eta) & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & a(1+\xi) & a(1-\xi) & -a(1-\xi) & -a(1+\xi) \\ a(1+\xi) & a(1-\xi) & -a(1-\xi) & -a(1+\xi) & b(1+\eta) & -b(1+\eta) & -b(1-\eta) & b(1-\eta) \end{bmatrix}\begin{pmatrix} u_1 \\ u_2 \\ u_3 \\ u_4 \\ v_1 \\ v_2 \\ v_3 \\ v_4 \end{pmatrix} \qquad 6\text{-}67$$

Where a, b, x and h are defined as following.

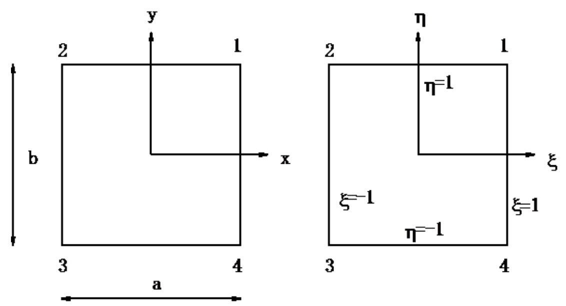


Figure 6-14- Coordinate System of quadrangle element

When cables connected to structures are moving together with ground displacement, they are undergone pure tension and compression. In this case, cable strain is obtained from division of relative displacement among structure on its primary length according to figure 6-15.

$$\varepsilon_L = \frac{\Delta L}{W} \qquad 6\text{-}68$$

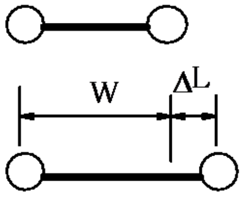


Figure 6-15-Cable tension among structures

5-Ground sliding

Sliding due to earthquake make strain through following relation:

$$\varepsilon_{LS} = \frac{\Delta L}{W} \qquad 6\text{-}69$$

Where, ΔL is elongation along cable direction.

6-9-2-Deformation of cable connected to structure

1-In conduit

if cable behavior in conduit is elastic, maximum strain is given by:

$$\varepsilon_{D} = \frac{D}{2EI}\left(\frac{7}{12} qW_2^2 + \frac{2}{3} C_1 W_2\right) \qquad 6\text{-}70$$

where D, q and W_2 are cable diameter, weight and length unit of cable and horizontal effective length, respectively:

$$C_1 = \frac{q\left(W_2^3 - W_1^3 + 2W_2W_1^2\right) + \dfrac{12hEI}{W_1}}{2\left(W_1^2 + W_2^2\right)} \qquad 6\text{-}71$$

Where, h, w_1 and EI are fault vertical displacement, horizontal length and cable bending hardness, respectively.

2-Manhole and building

in this case, maximum strain is obtained from relations 6-72 and 6-73:

$$\varepsilon_{MH} = \frac{\sqrt{W^2 + h^2} - W}{W} \qquad 6\text{-}72$$

$$W = \frac{2\pi}{\beta}, \beta = \sqrt[4]{\frac{K}{4EI}} \qquad 6\text{-}73$$

Where K and EI are spring module between cable and soil and cable hardness respectively.

Chapter 7

Seismic design safety control of communication network manhole

7-1-Principal components

This chapter, addresses seismic design of buried manholes in communication network through following classification:

1-Insitu manholes
- circular section
- rectangular section

2-prefacbricated manholes

7-2-Procedure of seismic design

Procedure of manhole seismic design is on the basis of its function after earthquake.
Functional levels for these structures are consisted of:

1-Unceased operational function (operational limit) in which manhole must maintain its function after earthquake immediately and show approximately elastic behavior.

2-Operation function with minimum suspension (ultimate limit mode), in which manhole function must be repairable and recyclable under earthquake of hazard level 2 in least possible time.

7-3-Required function level of manhole components and hazard level

7-3-1-Required function

Manholes are used in main and subsidiary communication lines.
Anticipated function of manhole is based on line function that is implemented on it.
It must be used unceasingly in hazard level 1 and with least suspension in hazard level 2.

Table 7-1 presents manhole status under earthquake of hazard level 2.

Table 7-1-Required status of various parts of manhole

Manhole components	Required status
connection between manhole and conduit	Bending angle and elongation length must be in allowable range and soil or sand mustn't penetrate or slump into it.
main body of manhole	Stress intensity must be in limit value range. if repair to be needed, excavation mustn't cause traffic jam. soil and sand mustn't penetrate inside from connection points of prefabricated and opening parts

7-4-Design procedure

1-Manhole design procedure is on the basis of its material type and performed according to following items:

1-1-Vertical section

1-2-Horizontal section

1-3-Connection between prefabricated parts

1-4-Soaking of main body due to liquefaction

2-Preparations against earthquake

Relations of this part are generally related on in situ manhole with circular or rectangular section and prefabricated manhole. In prefabricated manhole, connection part between blocks acts as connection. Table 7-2 presents manhole design methods in main lines.

Table 7-2-Design methods of manhole connected with main lines

<table>
<tr><th colspan="2" rowspan="2">item / structure type</th><th rowspan="2">judgment about liquefaction (FL-value)</th><th colspan="2">vertical section</th><th>horizontal section</th></tr>
<tr><th>opening wide</th><th>stress intensity</th><th>stress intensity</th></tr>
<tr><td rowspan="3">manhole type</td><td>in situ type with circular section</td><td>ductile</td><td></td><td>allowable stress-ductile</td><td>allowable stress-ductile</td></tr>
<tr><td>in situ type with rectangular section</td><td>ductile</td><td></td><td>allowable stress-ductile</td><td>allowable stress-ductile</td></tr>
<tr><td>prefabricated type</td><td>ductile</td><td>allowable stress-ductile</td><td>allowable stress-ductile</td><td>allowable stress-ductile</td></tr>
</table>

For manhole of other lines, only allowable stress method is applied.

Here are points that must be heeded in seismic designing of manhole:

1-Soil profile must be according to actual condition and it excavation is high and ground primary condition is changed, it must be considered in design.

2-In conversion of displacement to force and in displacement response method, coefficients of horizontal reaction K_h, vertical reaction K_v and shear reaction and rotatory spring constant of bed are required.

In the case of using these coefficients, short-term additional load will be negligible (α=1). However, reduction of soil characterization due to liquefaction is not required.

3-K_h is given by relation 7-1.

$$k_h = k_{h0}\left(\frac{B_h}{0.3}\right)^{-3/4} \quad 7\text{-}1$$

$$k_{h0} = \frac{1}{0.3}\alpha \cdot E_0 \quad 7\text{-}2$$

Where

K_h bed reaction factor in horizontal direction in terms of kN/m^3

K_{h0} horizontal reaction factor from circular plate test with diameter of 0.3 m

α short-time additional load factor

E_0 ground deformation module (kN/m^2)

In the case of E_0 calculation based on SPT number, E0 is calculated equal to 2800×N.

K_v is equal to:

$$k_v = k_{v0}\left(\frac{B_v}{0.3}\right)^{-3/4} \quad 7\text{-}3$$

$$k_{v0} = \frac{1}{0.3}\alpha \cdot E_0 \quad 7\text{-}4$$

Where

k_{v0} reaction factor corresponding to circular plate test with diameter of 0.3 m

α short-time additional load factor

E_0 ground deformation module (kN/m^2)

4- Equivalent width of foundation for k_h calculation is given by

$$B_h = \sqrt{A_h} \qquad 7\text{-}5$$

Where

B_h equivalent width of foundation in terms of m perpendicular to loading direction

A_h loading area in transversal direction (m^2)

For quadrangle manhole: D manhole width × H wall length=A_h

For circular manhole: 0.8D circular diameter × H wall length=A_h

B_v for vertical load factor is given by

$$B_v = \sqrt{A_v} \qquad 7\text{-}6$$

A_v is loading area in vertical direction in terms of m^2.

For quadrangle manhole, A_v is base area of manhole.

For circular manholes, B_v is equal to manhole diameter.

5-k_s and k_θ are given by relations 7-7 and 7-8.

$$k_s = \lambda \times k_v \qquad 7\text{-}7$$

Where

k_s shear spring constant of soil (kN/m^3)

λ Ratio of bed reaction factor in the transversal direction to its value in vertical direction that is between quadrant and third.

$$k_\theta = k_v \times I \qquad 7\text{-}8$$

Where

k_θ rotatory spring constant of soil (kN.m/rad)

I geometrical inertial moment of manhole base (m^4/m)

6-Seismic analysis is primarily performed through displacement response method for vertical section.

7-Then computations are performed in lateral direction using obtained factors of bed reaction.

8-Figure 7-1 shows suitable in suit and prefabricated manhole model.

9-In the case of in situ manhole, elastic manhole behavior is assumed to be elastic and its interaction with surroundings soils is modeled with addition of springs in boundaries.

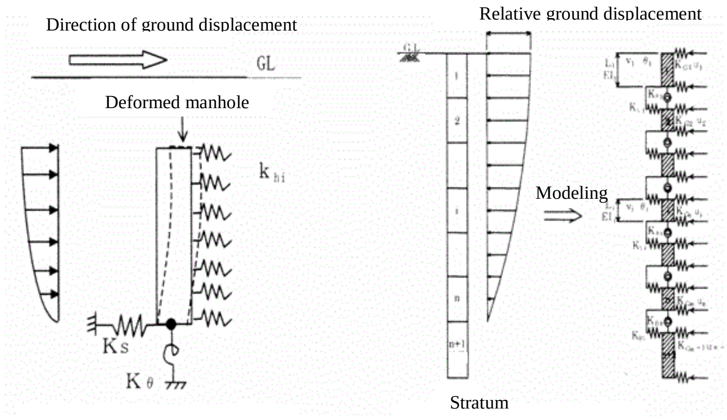
10-Part connections are modeled with spring in the case prefabricated mode.

11-Stress intensity under normal and seismic load is controlled in the case of lateral load.

Normal load involve soil pressure in the stationary state under water pressure.

In order to calculate earthquake load, bed reaction ω_b is used with consideration of manhole body as continuous body in the vertical direction.

ω_b is obtained with multiplication of relative ground displacement $\Delta U(Z)$ and component vertical displacement $\delta(z)$ in ith layer bed reaction factor in the transversal direction khi.



In situ type

Elastic body with boundary spring Blocks connected with spring

Figure 7-1-Sample of calculative manhole model

$$\omega_b = \{\Delta U(z) - \delta(z)\} \times k_{hi} \qquad 7\text{-}9$$

Where

ω_0 additional load due to earthquake (load reaction due to response displacement) (kN/m^2)

ω_0 is in terms of kN/m^2. If ω_0 is negative, force is applied in opposite direction and its absolute value is added to normal load, namely ω_0.

Where

$\Delta U(z)$ relative ground displacement in depth z(m)

$$\Delta U(z) = U_h(z) - U_h(h) \qquad 7\text{-}10$$

Where

$U_h(z)$ ground displacement in depth z(m)

$U_h(h)$ ground displacement in bottom level of manhole (m)

(zδ(component displacementin depth z(m)

k_{hi} bed reaction factor in transversal direction in node I (kN/m^3)

12- In the case of in situ manhole with rectangle section, calculations must be performed in longitudinal and transversal direction.

In previous earthquake, main failure pattern of manholes was due to sliding or rupture of block connections in soft grounds.

These connective blocks are usually connected together with mortar.

As a result of sliding blocks, soil and sand are collapsed and shut the path which is accounted as secondary damage. So, wall thickness and reinforcement must be increased to neutralize shear force of earthquake in big size in situ manholes that are located in main lines.

Unreinforced concrete is usually used in small in situ manholes with circular sections in low depths.

Seismic function of manholes that are connected to more than one manhole is decreased due to size increment of opening and forming crack or fracture so they must have sufficient strength in opening location under earthquake load.

Control must be exercised in connections among prefabricated blocks in prefabricated manholes to make sure of their integrity.
In whole, prefabricated manholes show higher strength.
If absorbing high displacement is needed, flexible connections are also required.

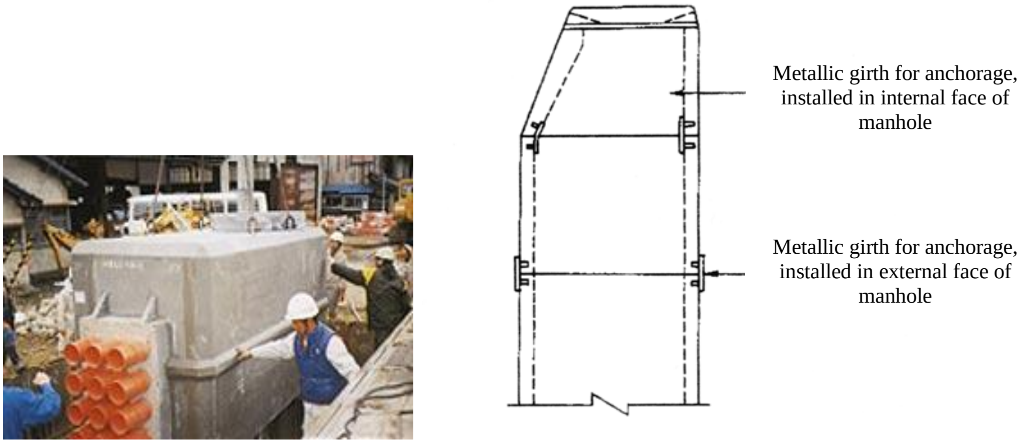


Figure 7-2-An example of earthquake-resistant measures in inside and prefabricated manhole

7-5-Procedure of manhole calculations

7-5-1-Seismic design of connection among blocks

In seismic design of connection among blocks, joint opening must be considered in designing vertical sections.

According to following figure, joint among prefabricated blocks opens during earthquake.

Amount of opening depends on joint type, block width and size of internal space and applied force and this opening is formed under effect of bending, shear and movement of rotatory springs.

These springs show permanent deformation through representation of nonlinear behavior.

Non-linear behavior of spring is considered usually on the basis of 3 lines model.

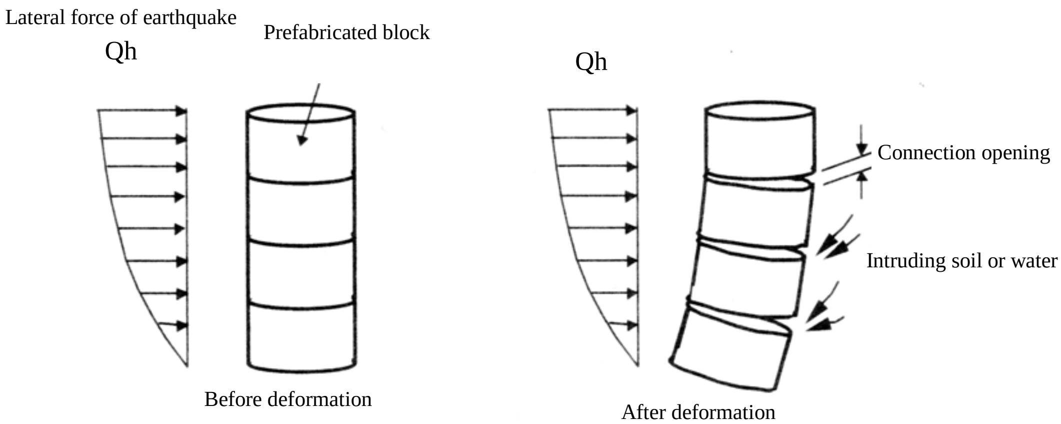


Figure 7-3-Systematic deformation of prefabricated manhole in earthquake

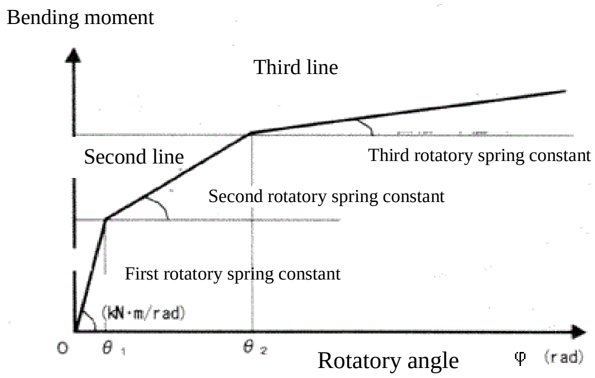


Figure 7-4-Relation between rotatory angle θ and bending moment M (three lines model)

7-5-2-Seismic design of manhole body

Seismic design of manhole body is performed in vertical and transversal directions.

Stress in vertical section of body, wall thickness and amount of steel in axial (vertical) direction must be controlled in designing in vertical direction.

Convoluted transversal rebars endure forces under normal loads such as water and soil pressure while vertical rebars play major role in earthquake.

Convoluted rebars must be controlled in designing in lateral section.

In order to compute shear force in manholes of circular section, cross section is converted into rectangular with equivalent area.

Rebars can be arrayed in similar ways as conduits or walls.

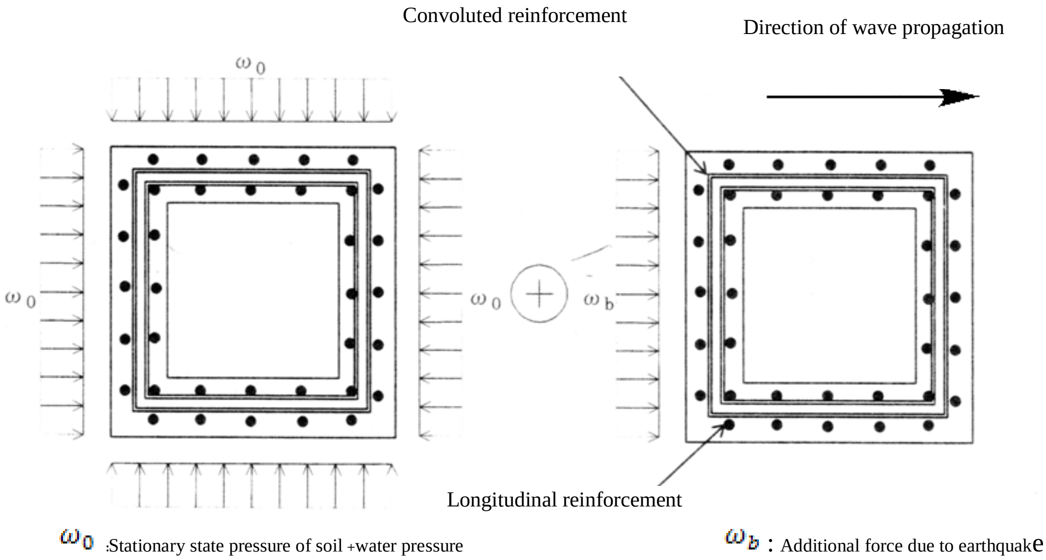


Figure 7-5-Loading and arraying rebars (insitu rectangular manhole)

7-6-Allowable value

Allowable response values are characterized according to technical structural specifications and manufacture of parts.

Current relevant Iranian guideline must be used for concrete and steel manhole.

7-7-Acceptance criterion

results of seismic calculations are controlled as following:

- Control for hazard level 1 through allowable stress method or operation limit state method
- Control for hazard level 2 through ultimate limit state
- preparations related to earthquake

-Securing strength in wall and its rebars in vertical direction (strength in vertical sections)
-Securing wall strength and convoluted rebars in transversal direction (strength in lateral sections)
-Controlling joint openings in prefabricated manholes
-Controlling displacement absorption in connections

-Controlling operation limit state in hazard level 1 is performed as following:

1-Any leakage must be prevented through controlling allowable stresses

2-Opening of prefabricated joints must be lower than 2 mm and sealing must be maintained.

-Controlling ultimate limit state in hazard level 2 is performed as following:

1-Stresses mustn't be exceeding from yielding limit to prevent any inflow of materials, cracking and sliding of connections.

2- Opening of joints must be lower than 10 mm in order to prevent any inflow of materials.

-At first, wall thickness and reinforcing in axial direction must be examined in designing vertical section of manhole.

-If controlling in vertical direction isn't responsive, lowering elevation or changing arrays of opening may be exercised.

-If structure crosses hard layers, layout controlling of joints is required for prevention of stress concentration in connections of hard and soft layers.

-If capacity in transversal direction isn't responsive, regulations may be fulfilled through changing thickness of design in vertical direction.

-In each level of earthquake, connection type and block elevation must be conformed together in order to fulfill connection opening criterion in prefabricated manhole. If conformation isn't possible, insitu prefabricated manhole must be used.

-If required, flexible connection must be used in connection with conduit. If solid foundation is used on gravel embankment, manhole and solid foundation must be separated from each other in order to prevent conduit effect on manhole.

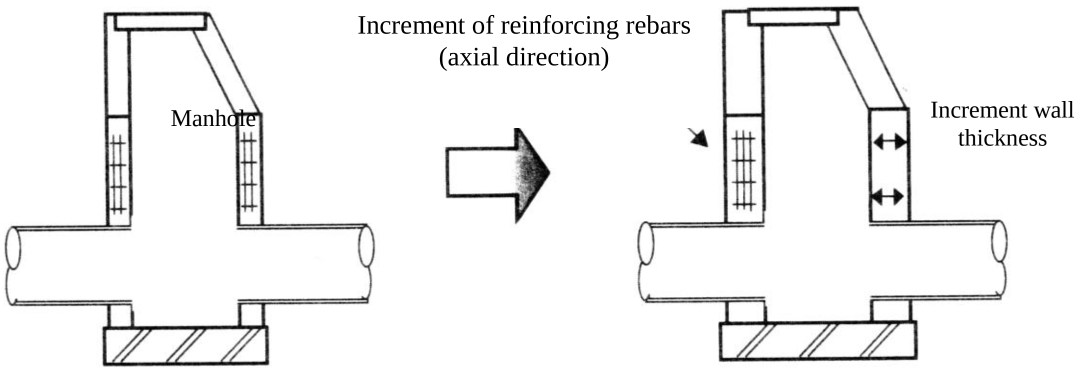


Figure 7-6-Method of changing manhole type

Chapter 8

Seismic design and safety control internal equipment bracing

8-1-Objective components

1-Internal equipment includes ventilation apparatus, power generator, battery, power panel, control panel, transformation, etc.
2-Seismic design of internal equipment bracing is performed for following items:
2-1-Bracing bolts
2-2-Support components
2-3-Stoppers
3-Internal seismic function of equipment must be supplied and secured.

This guideline involves internal equipment installed in steel and reinforced concrete buildings with elevation lower than 60 m.

8-2-Seismic design procedure

8-2-1-General

1-Design earthquake force for internal equipment is calculated using semi-static method (by means of localized earthquake factor) or dynamic method.
2-Static equivalent force method is a seismic evaluation method for installation of equipment with 100 kg or lower weight.

8-2-2-Semi-static method

Horizontal design earthquake force F_H, that is applied on gravity center of internal equipment is given by:

$$F_H = K_{H1} \cdot W \quad (8\text{-}1)$$

Where

K_{H1} horizontal design earthquake factor

W weight of equipment

If required, vertical force of earthquake, F_v is obtained from relation 8-2.

$$F_V = K_{V1} \cdot W \quad (8\text{-}2)$$

$$K_{V1} = (1/2)K_{H1}$$

Where

K_{V1} vertical design earthquake factor

Horizontal design earthquake factor, K_H for common building is given by relation 8-3.

$$K_{H1} = \beta_E \cdot A \quad (8\text{-}3)$$

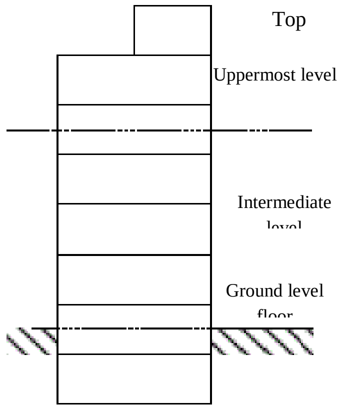
Where

A basic design acceleration

β_E localized earthquake factor that is given by table 8-1.

Table 8-1-Localized design earthquake factor for internal equipment

	Importance of internal equipment of building		
	Very important and important	Intermediate	low
Upper and uppermost level	5	3.75	2.5
Intermediate level	3.75	2.5	1.5
Floor and underground level	2.5	1.5	1



In buildings with two to 6 stories, uppermost level is the last level.
In buildings with 7 to 9 stories, uppermost level is the two last levels.
In buildings with 10 to 12 stories, uppermost level is the three last levels.
In buildings with more than13 stories, uppermost level is the four last levels.

8-2-3-Dynamic method

In buildings involving separator or seismic control, response acceleration in each story G_f (m/s^2) is given by dynamic analysis.

1-Horizontal design earthquake factor, K_H

1-1-$K_{H'}$ for equipment

$$K_H' = A \cdot B \cdot B_B \cdot B_E \cdot D_{ss} \cdot I_S \cdot I_K \qquad 8\text{-}4$$

Where

B Building response factor

B_B Response magnification factor

Table 8-2-Magnification factor of equipment response B_E

Equipment bracing	**Magnification factor B_E**
equipment with vibration control system	**2**
braced equipment	**1.5**

D_{ss} characteristic factor of equipment seating that is considered to be 2.3 if dynamic analysis of building isn’t performed.

I_S importance factor of equipment (I_S=1 to 1.5)

I_K importance factor of building (I_K=1 to 1.5)

1-2-Response

Building story response G_f is obtained using seismic response analysis as following:

$$G_f = A \cdot B \cdot I_K . g \qquad 8\text{-}5$$

$$K_H' = (G_f / g) \cdot B_E \cdot D_{ss} \cdot I_S \qquad 8\text{-}6$$

for equipment

where

G_f story response acceleration (m/s^2)

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

Horizontal design earthquake factor, K_H is given by table 8-3 along with K_H' value.

Table 8-3-Horizontal design earthquake factor K_H using dynamic analysis of building

horizontal design earthquake factor K_H	$K_{H'}$ Calculated in sections 1 or 2
0.4	less than 0.42 (for common building)
0.6	less than 0.63 (for building and equipment with high usage factor)
1	more than 0.63 and less than 1.1
1.5	more than 1.1 and less than 1.65
2	more than 1.65

2-Vertical design earthquake factor K_V

Vertical earthquake factor is assumed to be half of horizontal factor

Vertical earthquake factor in building with seismic separator is obtained from localized earthquake factor regarding seismic force of equipment.

8-3-Calculations and safety control

1-Seismic design of bracings must be performed on the basis of allowable stress method (ductile design method isn't used)

2-Stress in supports mustn't be more than allowable stress in order to avoid overturn, sliding and falling equipment.

8-3-1-Bracing bolt

In symmetrical array of bracing bolts, tensile stress σ and tensile force R_b of a bolt is given as following.

$$\sigma = M/Z \quad \text{8-7}$$

$$R_b = A_b\sigma \quad \text{8-8}$$

Where

M bending moment applying on equipment base

Z section module of group of bracing bolts

l_i distance from center of bracing bolt to neutral axis

A_b cross section of bracing bolt stem

Bracing bolts that are positioned as a rectangular matrix must be designed for the most critical load in both directions of section.

Bracing bolts that are positioned as a circle must be examined with the assumption that neutral axis passes from circle center and all bolts are undergone shear stress.

8-3-1-1-Tensile and shear force of bracing bolt

when seismic force is applied on equipment, tensile and shear force in bracing bolt must be calculated according to their installation point that is as following:

1-Positioning in floor or foundation (equipment with rectangular section)
2- Positioning in floor or foundation (equipment with circular section)
3-Hanging from wall
4-Hanging from ceiling

1-Equipment with rectangular cross section that is positioned on floor or foundation

1-1-Tensile and shear force of bracing bolt

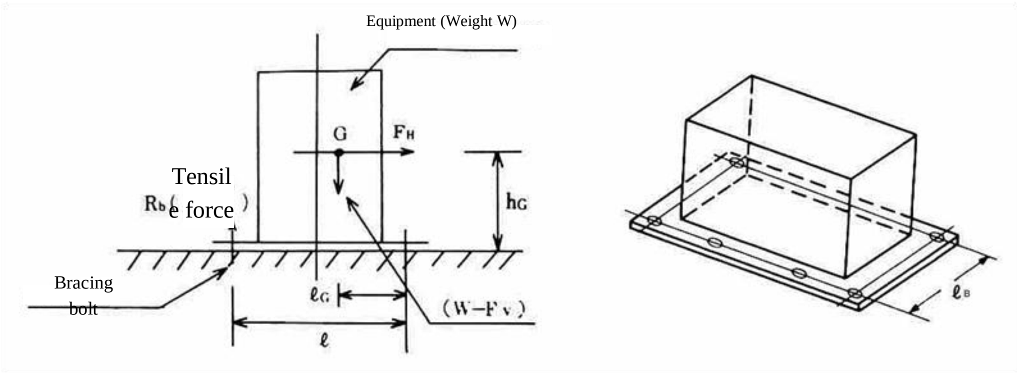


Figure 8-1-Equipment of rectangular cross section that is situated on floor

G gravity center of equipment
R_b tensile force of bracing bolt
n_a total number of bracing bolt
n_t total number of bracing bolt in a side that receive tensile force in the overturning condition of equipment
h_G distance from constant face to gravity center of equipment
l_B bolt span in the direction of target
l_G distance from bolt center to gravity center of equipment in the direction of target ($\ell_G \leq \ell/2$)
F_H horizontal design earthquake force ($F_H = K_H \cdot W$)
F_V vertical design earthquake force ($F_V = (1/2)F_H$)

1-2-Tensile force of bracing bolt

$$R_b = \frac{F_H \cdot h_G - (W - F_V) \cdot \ell_G}{\ell_B \cdot n_t} \qquad 8\text{-}9$$

1-3-Shear force of bracing bolt

$$\tau = \frac{F_H}{n_a \cdot A_b} \quad \text{or} \quad Q = \frac{F_H}{n_a} \qquad 8\text{-}10$$

τ shear stress applied on bolt
Q shear force applied on bolt
n_a total number of bracing bolt

2-Equipment of circular cross section that is positioned on floor or foundation

2-1-Tensile and shear force of bracing bolt

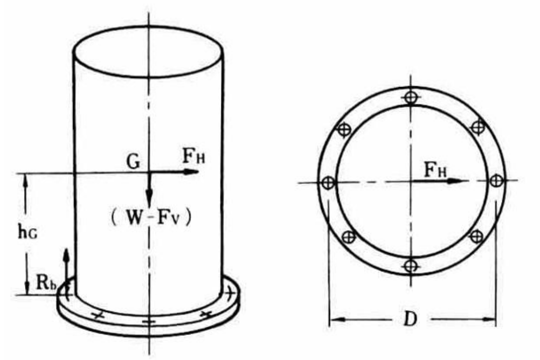


Figure 8-2- Equipment of circular cross section that is positioned on floor

G ،W ،F_V ،F_H ،R_b ،h_G are as defined before an indicated in figure 8-2

D bolt braced with circular cross section

2-2-Tensile force of bracing bolt

$$R_b = \frac{4}{n_a \cdot D} F_H \cdot h_G - \frac{W - F_V}{n_a} \qquad 8\text{-}11$$

2-3-Shear force of bracing bolt

$$\tau = \frac{F_H}{n_a \cdot A_b} \text{ یا } Q = \frac{F_H}{n_a} \qquad 8\text{-}12$$

3-Equipment that is situated on well

3-1-Tensile and shear force of bracing bolt

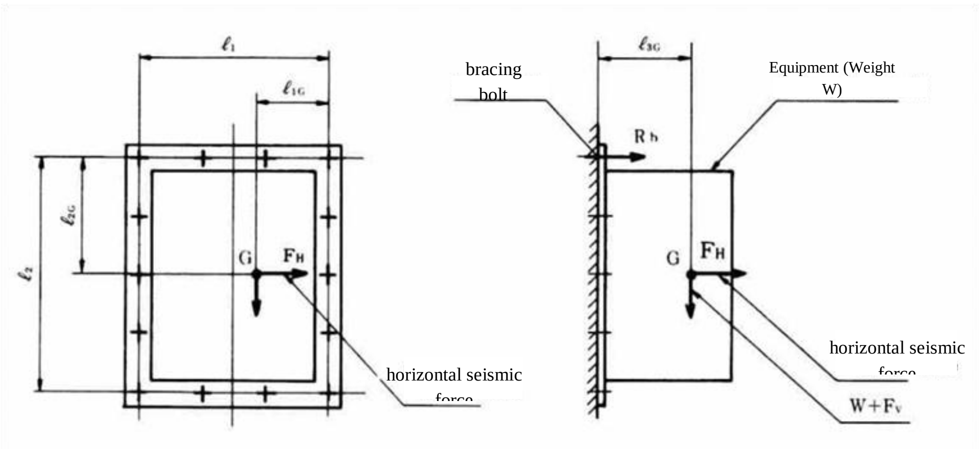


Figure 8-3-Equipment that is situated on wall

Where

ℓ_1 bolt span in horizontal direction

ℓ_2 bolt span in vertical direction

ℓ_{1G} horizontal distance between bolt center and gravity center of equipment ($\ell_{1G} \le \ell_1/2$)

ℓ_{2G} vertical distance between bolt center of upper section and gravity center of equipment

ℓ_{3G} distance between wall face and gravity center of equipment

n_{t1} number of bracing bolts in one side from upper or lower face (number of bracing bolt that is located on line l_1 in figure 8-3).

n_{t2} number of bracing bolt that is situated in one side of lateral face (number of bracing bolt that is located on line l_2 in figure 8-3).

3-2-Tensile force of bracing bolt

Tensile force of bracing bolt situated on the upper part is considered to be higher value obtained from relations 8-13 and 8-14.

$$R_b = \frac{F_H \cdot \ell_{3G}}{\ell_1 \cdot n_{t2}} + \frac{(W + F_V) \cdot \ell_{3G}}{\ell_2 \cdot n_{t1}} \qquad 8\text{-}13$$

$$R_b = \frac{F_H \cdot (\ell_2 - \ell_{2G})}{\ell_1 \cdot n_{t1}} + \frac{(W + F_V) \cdot \ell_{3G}}{\ell_2 \cdot n_{t1}} \qquad 8\text{-}14$$

3-3-Shear force of bracing bolt

$$\tau = \frac{\sqrt{F_H{}^2 + (W + F_V)^2}}{n_a \cdot A}$$

or 8-15

$$Q = \frac{\sqrt{F_H{}^2 + (W + F_V)^2}}{n_a}$$

4-Equipment situated on the ceiling

4-1-Tensile and shear force of bracing bolt

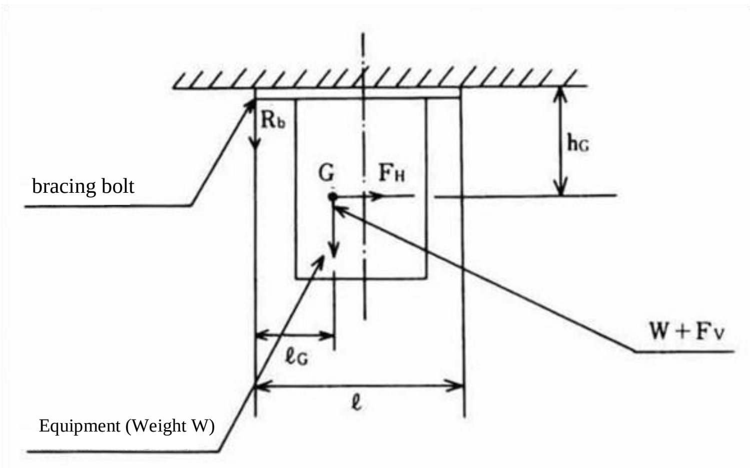


Figure 8-4-Equipment of the ceiling

4-2-Tensile force of bracing bolt

Tensile force of bracing bolt is determined from relation 8-16.

$$R_b = \frac{F_H \cdot h_G + (W + F_V) \cdot (\ell - \ell_G)}{\ell \cdot n_t} \qquad 8\text{-}16$$

4-3- Shear force of bracing bolt

Calculation method for shear force of bracing bolt is similar with relation 8-12.

8-3-1-2-Tensile force of bracing bolt

Reaching tensile force to yield stress in bolt means overturning equipment. Horizontal earthquake forces corresponding to installation condition are shown in figures 8-1 to 8-4. Tensile forces are obtained from relations 8-10 to 8-16.

8-3-1-3-Shear force of bracing bolt

Horizontal earthquake force leads to sliding equipment in horizontal direction. Shear force applied on each bolt is calculated and its size is determined in such a way that applied stress must be lower than allowable shear stress. Friction with floor, due to equipment weight and bolt bracing force mustn't be considered. Horizontal earthquake forces are shown in figures 8-1 to 8-4. Shear forces are shown in figures 8-9 to 8-15.

8-3-1-4-Cases that there is no need for calculations

There is no need for calculations, if equipment weight is lower than allowable weight obtained in figure 8-5 in terms of ratio of length to width and horizontal earthquake factor and equipment is installed with four or more bracing bolt with number M8 or higher on the floor.

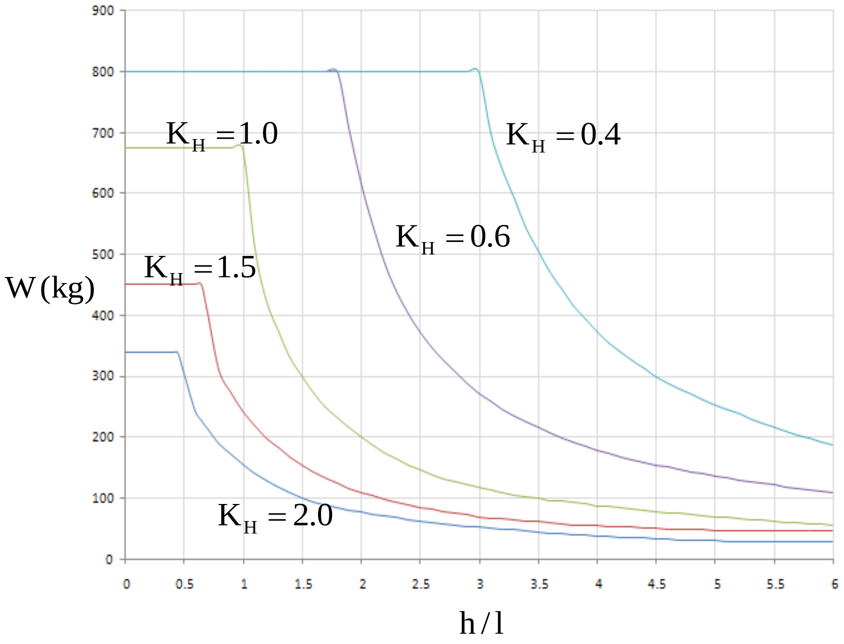


Figure 8-5-Allowble weight in terms of ratio of equipment precision and horizontal earthquake factor

8-4-Upper support

8-4-1-Design

When equipment brace isn't enough to control overturning and sliding or dumper or connection

equipment is used in floor, seismic stability can be secured through upper support.

Methods of upper support installation include:

1-Connection through connecting member to wall

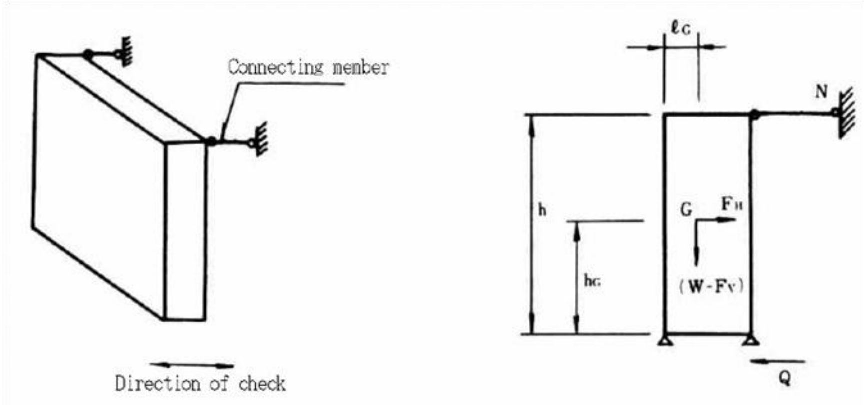


Figure 8-6-Method of installation and check of connecting member to wall

Axial force applied on support member is given by

$$N_O = \frac{F_H \cdot h_G}{m \cdot h} \qquad 8\text{-}17$$

Shear force applied on lower brace bolt is given by

$$Q_O = \frac{F_H (h - h_G)}{n \cdot h} \qquad 8\text{-}18$$

$$\tau = \frac{Q}{A_b} \qquad 8\text{-}19$$

Where

ℓ_G	horizontal length up to gravity center of equipment
h	height of equipment
h_G	elevation of fixed face up to gravity center of equipment
m	number of connecting members
n	total number of bracing bolt
A_b	cross section of bracing bolt

Condition of $N \le F_{CA}$ must be satisfied in connecting member (F_{CA} is short-term allowable compressive force). Diameter of bracing bolt in connecting member must be designed with consideration of $R_b = N/n_0$ (N is tensile force applied on each member and n_0 is the number of bracing bolt of each member). Diameter of bracing bolt must be designed with consideration of (N is tensile force that is applied on each member and n_0 is the number of bracing bolt of each member). Diameter of lower bracing bolt must be calculated using Q and τ in relations 8-18 and 8-19.

2-Connecting member to ceiling

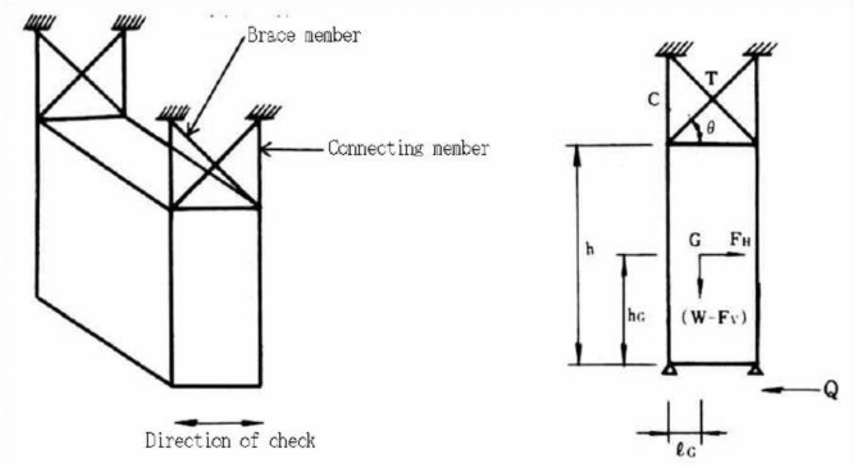


Figure 8-7-Method of installation and control of connecting member to ceiling

Applied axial force T_0 and C_0 on bracing member and connecting member is given by

$$T_O = \frac{F_H \cdot h_G}{m \cdot h} \cdot \frac{1}{\cos\theta_b} \qquad 8\text{-}20$$

$$C_O = T\sin\theta_b \qquad 8\text{-}21$$

Applied force on bracing bolt of upper support member is given by

$$R_b = \frac{T\sin\theta_b}{n_0} \text{ : Tensile force} \qquad 8\text{-}22$$

$$Q_b = \frac{T\cos\theta_b}{n_0} \text{ : Shear force} \qquad 8\text{-}23$$

Applied shear forces Q and τ on lower bracing bolt must be according to relations 8-18 and 8-19.

Θ_b angle of bracing member

n_0 number of bracing bolts of upper support member

Diameter of bracing bolts of upper support member must be designed using R_b and Q_b.

In the case of lower bracing bolt, it must be acted similar to connecting member to wall.

3-Connecting member to wall in two directions

3-1-Upper support

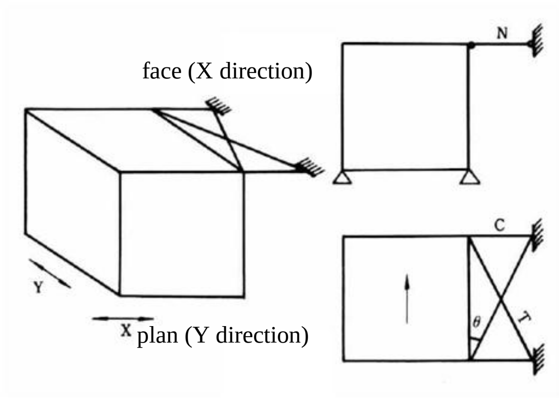


Figure 8-8-Method of examining C component

N in X direction is calculated from relation 8-17 and C and T in Y direction are calculated from relations 8-20 and 8-21.

Upper support member must be safe against axial force N, tensile force T and compressive force C.

Designing braced bolt must be according to designing connecting members of wall or ceiling.

8-4-2-Selection upper support member

Support member must be designed in such a way that satisfies following relations.

$C \le C_a$ for compressive force

$T \le T_a$ for tensile force

T_a and C_a are allowable tensile and compressive forces respectively.

With assumption of known N force, steps of design are as following:

1-Assumption of member cross section as A_r

2-Calculation of stress $\sigma = N/A_r$

3-Comparison of stress σ with allowable stress f_a

Support member with allowable stress f_a that is higher than N, is simply designed with following calculations:

$$N/N_a \le 1.0 \quad , N \le A \cdot f_a = N_a \qquad 8\text{-}24$$

Where

f_a is allowable stress and N_a is allowable force

When bending moment is exceeding yield limit, relation $N/N_a + M/M_a \le 1.0$ is applicable.

8-5-Stopper

8-5-1-General

When tightening bracing bolt isn't possible through support, seismic stopper may be used.

Stopper is installed in small distance from main equipment, usually together with damper and doesn't contact with equipment.

Stopper must be tighten to foundation or main body through bolts in order to prevent sliding due to seismic force

Stopper type must be selected after controlling overturning likelihood of equipment under applied seismic force on gravity center of equipment.

8-5-2-Selection of stopper type

Various parts of stoppers must be designed against earthquake force.
Stopper must prevent sliding or overturning equipment.
If damping with high deformations is required, spring stopper is used .

Here are various types of stopper including:

1-Sliding stopper

This type is used to prevent horizontal movement and is made from steel sheet or profile.

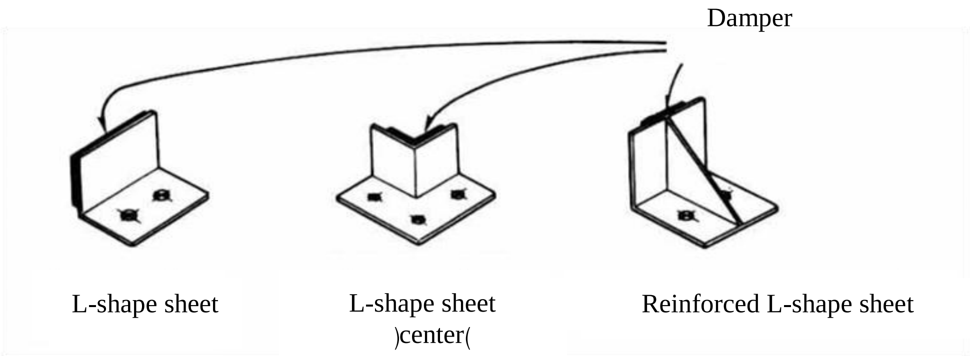


Figure 8-9-Types of L-shaped stopper

2-Overturning/sliding stopper

This stopper is used to prevent horizontal movement and overturning and is made from steel sheet or profile.

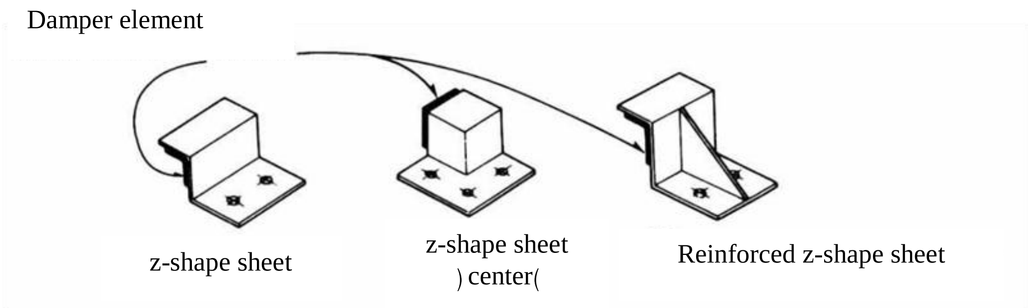


Figure 8-10-Z-shaped stopper

3-Other types

Figure 8-11 shows other types of stopper.

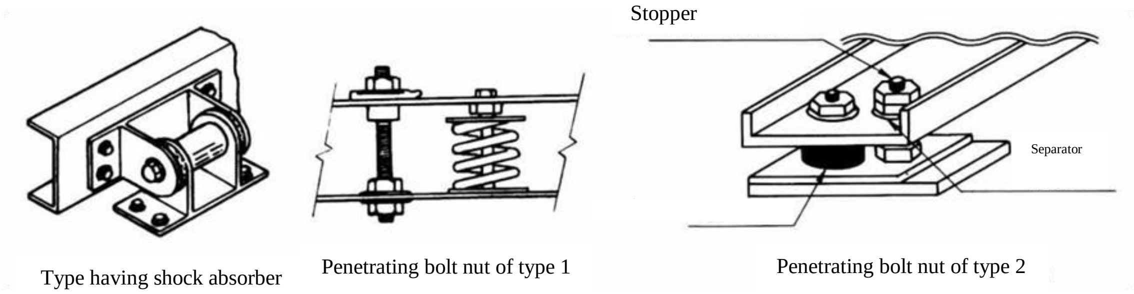


Figure 8-11-Other type of earthquake-resistant stopper (for sliding/overturning stopper)

8-5-3-Controlling stopper function

When bolt of Penetrating is selected, its tensile force must be controlled.

Overturning moment in point B must be obtained from relation 8-25 as indicated in figure 8-12.

$$T_O = \frac{W\{K_H h_G - \ell(1-K_V)\}}{\ell} \qquad 8\text{-}25$$

$$T_O \leq 0 \qquad (a)$$

$$h_G/\ell_G \leq (1-K_V)/K_H \qquad 8\text{-}26$$

Stopper of sliding type is used as long as force in damper doesn't reach yield strength.

$$T_O > 0 \qquad (b)$$

$$h_G/\ell_G > (1-K_V)/K_H \qquad 8\text{-}27$$

When tensile force in damper reach to yield strength, sliding and overturning stopper must be used.

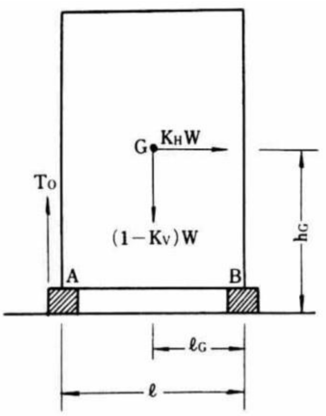


Figure 8-12-Effect point of applied forces on stopper

T_O : Tensile force of support A

Type and diameter of bracing bolt is selected according to section 5-5-4-1.

Designing bracing bolt and sheet of stopper is done as following:

1-Sliding stopper

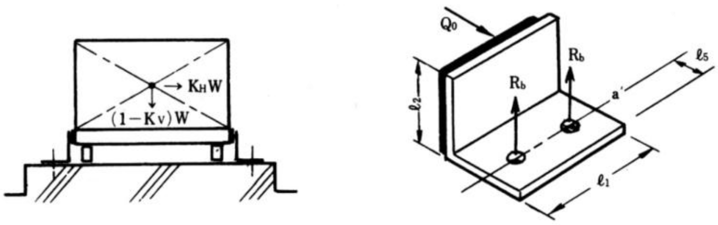


Figure 8-13- L-shaped sheet sliding stopper

λ_2 is elevation up to point of force application.

-Thickness of stopper sheet

$$t \geq \sqrt{6K_H W \ell_2 / \{f_b(\ell_1 - md_0)N_S\}} \qquad 8\text{-}28$$

Shear force of bracing bolt:

$$Q = \frac{K_H W}{mN_s} \qquad 8\text{-}29$$

Tensile force of bracing bolt:

$$R_b = \frac{\ell_2 K_H W}{\ell_5 \cdot m \cdot N_S} \qquad 8\text{-}30$$

f_b short term allowable bending stress of steel member

d_0 diameter of bolt bore

t thickness of stopper sheet

N_S number of stoppers in one side of equipment

2- Sliding and overturning stopper

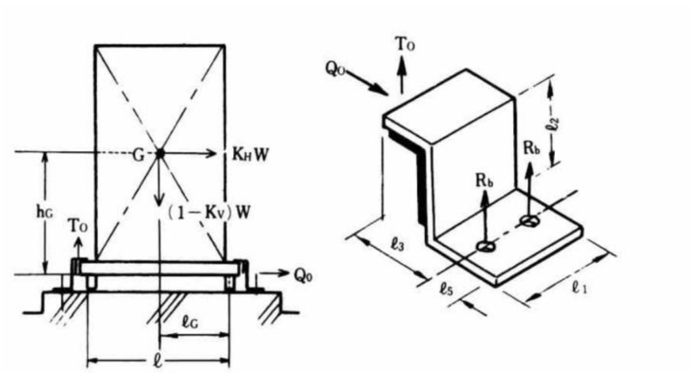


Figure 8-14-Z-shaped stopper

Thickness of stopper sheet is considered to be highest of following values:

$$t \geq \sqrt{\frac{6\{K_H h_G - \ell_G(1-K_V)\}W\ell_3}{f_b \ell(\ell_1 - md_0)N_S}} \quad \text{for } T_0 \qquad 8\text{-}31$$

$$\text{for } Q \quad t \geq \sqrt{\frac{6K_H W \ell_2}{f_b(\ell_1 - md_0)N_S}} \qquad 8\text{-}32$$

Shear force of bolt nut

$$Q = \frac{K_H W}{m \cdot N_S} \tag{8-33}$$

$$R_b = \frac{\{K_H h_G - \ell_G (1 - K_V)\} W}{\ell \cdot m \cdot N_S} \cdot \frac{\ell_3 + \ell_5}{\ell_5} \tag{8-34}$$

3-Controlling penetrative stopper

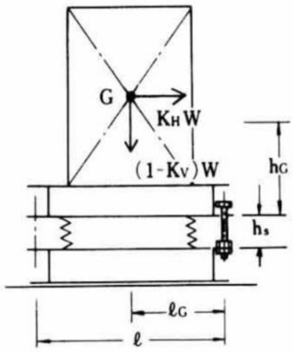


Figure 8-15-Penetrating stopper

Thickness of stopper sheet is considered to be highest of following values:

$$f_b \geq \sigma_{tb} = \frac{T}{A_e} + \frac{M}{Z} = \frac{W\{K_H \cdot h_G - (1 - K_V) \cdot \ell_G\}}{\ell \cdot n_t \cdot A_e} + \frac{K_H \cdot W \cdot h_S}{n_s \cdot Z} \tag{8-35}$$

$$f_S \geq \tau = \frac{K_H \cdot W}{{}_n A_e} \tag{8-36}$$

σ_{tb} member stress during receiving tensile and bending loads

T tensile force

A_e effective cross section (in the case of bolt, multiplication of axial cross section in 0.75)

M bending moment

Z cross module (in the case of nut bolt $Z = \frac{\pi \cdot (0.85d)^3}{32} = 0.06d^3$)

G_1 gravity center of equipment itself

G_2 convoluted gravity center of equipment and upper rack

h_S support point length of stopper up to upper rack

n_s total number of stopper nut

d bore diameter of bolt nut

n_t number of bolt nut located on communication line

f_S short term allowable shear stress of steel member