Islamic Republic of Iran
Vice Presidency for Strategic Planning and Supervision

Guideline for Seismic Design of Natural Gas systems

No. 601

Office of Deputy for Strategic Supervision
Department of Technical Affairs

2012

Table of content Page number

Chapter 1-General

1-General 3

1-1-Objective 3

1-2-Scope 3

1-2-1-Orgaization of this guideline 3

1-2-2-Application notes 4

1-3-Related codes and regulations 4

1-3-1-Normative references 4

1-3-2-Code Abbreviations 5

Chapter 2- Principles

2-Principles 9

2-1-Design earthquake 9

2-1-1-Risk levels of earthquake and their return period 9

2-1-2-Seismic design spectra 10

2-1-3-Distribution of seismic intensity in stories 11

2-1-4-Vertical seismic intensity of design 11

2-2-Seismic safety control (design methods) 11

2-2-1-Seismic safety control 11

2-3-Anticipated functions in this guideline 12

Chapter 3- Seismic Loading

3-1- Types of Loads 17

3-2- Weights calculations 17

3-3- Loads combination 17

3-4- Types of equipments by their location 17

3-5- Seismic loads calculation methods 17

3-6-The effects of earthquake on gas supply facilities 18

3-7-The method of imposing earthquake effects on gas equipments 18

3-8-Methods of measuring the loads from earthquake 18

3-8-1-Gas supply components importance factor 19

3-8-2-Considerations related to determining the importance factor of gas supply facilities 20

3-9-Displacement response method (for buried structures) 28

3-10-Dynamic method 28

3-11-Loading caused by earthquake's geotechnical hazards on equipments 29

3-11-1-Liquefaction 29

3-11-2-Landslide 29

3-11-3-Faulting ... 29
3-12-Soil classification ... 30
Chapter 4-Seismic Design and Safety Control Methods
4-1-Target components ... 33
4-2- Seismic design procedure ... 33
4-2-1-Seismic design necessity ... 33
4-2-2-Principles of seismic design steps ... 34
4-2-3-Allowable stress design method ... 34
4-2-3-1-Stress calculation ... 34
4-2-3-2-Allowable stresses ... 34
4-2-3-3-Evaluation of the calculated stress ... 34
4-2-4-Ductility design method ... 34
4.2.4.1 Earthquake hazard level for designing ... 34
4-2-4-2-Seismic response analysis ... 34
4-2-4-3-Ductility factor ... 37
4-2-4-4-Allowable ductility factor ... 38
4-2-4-5-Ductility factor evaluation ... 38
4-3-Materials ... 38
4-3-1-Allowable values for allowable stress design method ... 38
4-3-1-1-Allowable stress for compression materials ... 40
4-3-1-2-Seismic design allowable stress of support structure materials ... 40
4-3-1-3-Piping materials allowable stress ... 42
4-3-2-Allowable value for ductile design method ... 43
4-4-Acceptable performance level of component and seismic input ... 43
4-4-1-Required seismic performance ... 43
4-4-2-Seismic performance evaluation ... 43
Chapter 5 -Seismic Design and Safety Control of Piping and Pipe Rack
5-1-Piping seismic evaluation steps ... 47
5-2-Simplified evaluation method (allowable span method) ... 47
5-2-1-Allowable span length ... 47
5-2-2-Displacement capacity ... 48
5-3-Allowable stress design method ... 48
5-3-1-Piping’s support structure response analysis ... 48
5-3-2-Piping’s support structure seismic performance evaluation ... 48
5-3-3-Piping system response analysis ... 48
5-3-4-Piping stress calculation ... 49
5-3-5-Piping tension evaluation ... 50

5-3-6-Flange connection seismic performance evaluation ... 50
5-3-7-Valve's seismic performance evaluation ... 53
5-3-8-Seismic performance evaluation of expansion connection ... 55
5-3-9-Seismic performance evaluation of towers and tanks nozzle ... 57
5-4-Ductile design method ... 58
5-4-1-Evaluation of seismic design of inertia force and response displacement ... 58
5-4-1-1-Pipe's support structure response analysis ... 58
5-4-1-2-Piping system's response analysis ... 58
5-4-1-3-Damage mode ... 59
5-4-1-4- Piping seismic evaluation ... 59
5-4-1-5-Seismic evaluation of flange connection ... 60
5-4-1-6-Expansion connection seismic evaluation ... 61
5-4-1-7- Seismic evaluation of towers and tanks nozzle ... 61
5-4-1-8- Seismic evaluation of pipe's support ... 61
5-4-2- Seismic design evaluation to ground movement ... 61
5-4-2-1- Piping system design caused by foundation displacement due to ground movement ... 61
5-4-2-2-Foundation displacement due to ground movement ... 62
5-4-2-3-Response analysis method ... 62
5-4-2-4- Failure mode ... 62
5-4-2-5- Seismic evaluation of pipelines ... 62
5-4-2-6-Seismic evaluation of flange connection ... 63
5-4-2-7-Expansion connection seismic evaluation ... 63
5-4-2-8-Seismic evaluation of towers and tanks nozzle ... 63
5-4-2-9-Pipe's support seismic evaluation ... 63
Chapter 6- Seismic design and Safety Control of Horizontal Vessel
6-1-Steps of seismic design of horizontal vessel ... 67
6-2-Allowable stress design method ... 67
6-2-1-Response analysis method ... 67
6-2-1-1-Pseudo-static method ... 67
6-2-1-2-Modified pseudo-static method ... 68
6-2-2-Stress measurement ... 69
6-2-3-Allowable stress ... 75
6.2.4 Acceptance criteria ... 75
6-3-Ductile Design ... 75
6-3-1-Damage mode ... 75
6-3-2-Yield earthquake coefficient ... 76
6-3-3-Ductility coefficient ... 81

6-3-4- Allowable ductility coefficient ... 81
6-3-5- Acceptance criteria ... 81
Chapter 7- Seismic Design and Safety Control of Spherical Tanks
7-1-Steps for seismic design of spherical tank ... 85
7-2-Allowable stress design method ... 85
7-2-1-Analysis methods ... 85
7-2-1-1-Pseudo-static method ... 85
7-2-1-2-Modified pseudo-static method ... 86
7-2-2-Stress measurement ... 88
7-2-3-Allowable stress ... 94
7-2-4-Acceptance criteria ... 94
7-3-Ductility design ... 94
7-3-1-Damage mode ... 94
7-3-2-Yield earthquake coefficient ... 95
7-3-3-Ductility factor ... 102
7-3-4-Allowable ductility factor ... 102
7-3-5-Acceptance criteria ... 103
Chapter 8- Seismic Design and Safety Control of Cylindrical Tanks
8-1-General procedure for seismic designing ... 107
8-2-Designing using allowable stress method ... 108
8-2-1-Response analysis method ... 108
8-2-2-Pseudo-static method ... 108
8-2-3-Modified pseudo-static method ... 109
8-2-4-Stress measurement ... 110
8-2-5-Allowable stress ... 112
8-2-6-Acceptance criteria ... 112
8-3-Ductility method design ... 112
8-3-1-Risk level-2 of earthquake ... 112
8-3-2-Damage modes ... 113
8-3-3-Yield earthquake coefficient ... 113
8-3-4-Ductility factor ... 116
8-3-5-Allowable ductility coefficient ... 117
8-3-6-Acceptance criteria ... 117
Chapter 9 - Seismic Design and Safety Control of Tower and Vertical Vessel
9-1-Seismic design procedure of tower and vertical vessel ... 121
9-2-Allowable stress design method ... 121
9-2-1-Response analysis ... 121

9-2-1-1-Pseudo-static method 121
9-2-1-2-Modified pseudo-static method 121
9-2-1-3-Modal analysis 123
9-2-2-Stress measurement 123
9-2-3-Allowable stress 127
9-2-4-Acceptance criteria 127
9-3-Ductility method designing 127
9-3-1-Towers with shell support.... 127
9-3-1-1-Damage modes 127
9-3-1-2-Yield earthquake coefficient 127
9-3-1-3-Response ductility factor.... 131
9-3-1-4-Allowable ductility factor.... 131
9-3-1-5-Acceptance criteria 132
9-3-2-Towers with leg and ring supports 132
9-3-2-1Damage mode 132
9-3-2-2-Yield earthquake coefficient of towers with leg support 134
9-3-2-2-1Yield earthquake coefficient in damage mode of shell plate 134
9-3-2-2-2Yield earthquake coefficient of leg's damage mode 135
9-3-2-2-3-Yield earthquake coefficient in damage mode of anchor-bolt 136
9-3-2-2-4-Yield earthquake coefficient in damage mode of leg attachments 137
9-3-2-3-Yield earthquake coefficient of towers with ring support 141
9-3-2-3-1-Yield earthquake coefficient in damage mode of shell plate 141
9-3-2-3-2-Yield earthquake coefficient in damage mode of bolt 141
9-3-2-4-Response ductility factor 141
9-3-2-5-Allowable ductility factor.... 142
9-3-2-6-Criteria 143
Chapter 10- Seismic Design and Safety Control of Pseudo-Building Structures
10-1-Cover structures of equipments 147
10-1-1-Seismic design methods for cover and frame structures 147
10-1-2-Allowable stress method 147
10-1-2-1-Calculating horizontal seismic force imposed on cover and frame structures 147
10-1-2-2-Calculated stress of cover and frame structures 151
10-1-2-2-3-Evaluation of calculated stresses 151
10-1-3-Ductility design method 151
10-1-3-1-Design's horizontal seismic force calculation imposed on cover and frame structures 153
10-1-3-2-Damage mode 153
10-1-3-3-Current seismic demand 153

10-1-3-4-Ultimate seismic demand 154
10-1-3-5-Yield strength evaluation 154
10-1-3-6-Pedestal yield strength evaluation..154
10-2-Chimney..155
10-2-1-Chimney seismic design procedure..155
10-2-2-Design notes..155
10-3-Furnace 155
10-3-1-Furnace seismic design procedure 155
10-3-2-Furnace response analysis..155
10-3-2-1-Analytic modeling 155
10-3-2-2-Mass 156
10-3-2-3-Damping constant..156
Chapter 11- Seismic Design and Safety Control of Pipelines
11-1-Target Components 159
11-2-Seismic calculations of on-ground pipeline..159
11-2-1-Loads and their combinations 159
11-2-2-Design seismic force of load bearing supports 162
11-2-3-Safety control of load-bearing supports 162
11-2-4-Length of seat..163
11-2-5-Bracing constraint and clamp..163
11-2-5-1-Design force of constraints and clamps..164
11-2-5-2-Displacement limitation of constraints and clamps 164
11-3-Buried pipelines..165
11-3-1-Straight pipelines..165
11-3-1-1-Design for seismic wave propagation 165
11-3-1-1-2-Allowable strain 166
11-3-1-1-3-Maximum strain of pipe on ground's transmission region (special case of crossing of pipes from boundary of two grounds)..166
11-3-1-1-3-1-Soil's resilience at longitudinal direction..167
11-3-1-1-3-2-Soil rigidity at transverse direction..167
11-3-1-2-Location of maximum pipe's strain caused by permanent ground deformation (PGD) 168
11-3-1-2-1-Design for horizontal displacement caused by liquefaction..168
11-3-1-2-2-Design in crossing with fault..169
11-3-1-2-3-Design in landslip 170
11-3-1-2-3-1-Response of buried pipe to longitudinal PGD..170
11-3-1-2-3-2-Buried pipe response to transverse PGD..171
11-3-2-Buried straight pipes in local displacements of ground (gap local settlement).........................172

11-3-2-1-Axial displacement capacity, Δ_l, in straight pipeline 172
11-3-2-1-2-Transverse displacement capacity, Δ_t, of straight pipeline 174
11-3-3-Curve pipes (buried lines) 176
11-3-3-1-Design for seismic wave propagation 176
11-3-3-2-Seismic design in liquefaction 177
11-3-3-2-1-Deformation of pipe's curve in sloped areas 177
11-3-3-2-2-Pipe's curve deformation in shore areas 178
11-3-4-Design of branches with T-shaped connection 179
11-3-5-Riser pipe's design 179
11-4-Amount of allowable stress in pipelines 180
11-5-Acceptance strain criteria proportionate with seismic damage modes 181
11-5-1-High-pressure pipelines 181
11-5-2-Seismic design criterion in medium and low pressure pipelines 183
11-5-3-Critical angles in riser pipes 183
Appendix
1-Flowchart of allowable span in seismic design of piping 187
1-1-Method of span length calculation 187
1-2-Calculation of allowable span length 188
1-3-Calculation of piping displacement capacity 190
1-4-Calculation of relative displacement 191
2-Standard structure of seismic design of piping system 193
3-Analysis of structure response for piping support 195
4-Analytical model and corrected seismic force for designing piping system 197
5-calculation of piping tension 198
6-Allowable stress of piping seismic design 204
7-Step of seismic function evaluation of flengic connection 204
8-Seismic evaluation of valve 208
9-Method of seismic evaluation for expansional connection 210
10-Method of seismic evaluation of spray in towers and containers 211
11-11-Seismic function evaluation of pipe support 213
12-Flowchart of piping seismic design through ductile method 219
13-Evaluation of pipe bending in big deformations 219
14-Flowchart of inertia force and response displacement 222
14-1-Method of derivation response displacement of pipe support point through corrected semi-static approach 223
14-2-equivalent linear analysis, detailed analysis and response magnification factor 224
14-3-Evaluation of rupture modes of bent pipe, junction pipe and direct pipes and details of undulatory deformation of pipe 227

14-4-Details of design procedure for flange connection and required contact pressure of washer 228
14-5-Details of expansional connection evaluation 230
14-6-Details of spray evaluation of towers and containers 233
14-7-Evaluation of required seismic function in pipe support 234
15-Flexibility of piping system and seismic design procedure due to ground liquefaction....................240
16-Displacement and relative displacement among foundation 246
17-Flexibility factor and design procedure of embowed pipe....................247
18-Details of allowable angle of embowed pipe 248
19-Procedure of seismic function evaluation of flange connection due to ground movement...............249
20-Details of procedure of expansional connection evaluation due to ground movement 251
21-Details of evaluation procedure of spray of containers and towers due to ground displacement.......253
22-Seismic function evaluation of pipe support due to ground movement....................254
23-Yeild strength evaluation of stud....................257

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 natural gas systems due to earthquake

- Seismic design of natural gas systems is presented in this guideline
- Vulnerability against earthquake is very critical, so 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

Intended installations of this guideline are installations of gas system including refinery components, pressure reducing stations, pipelines and distribution networks.

– 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, chapter 4, issue 360 and issue 123 can be used for seismic design for foundation of equipment together with results extracted from seismic design of related instrument from this guideline.

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 piping and pipe rack
Chapter 6: seismic design and safety control of horizontal storage tank
Chapter 7: seismic design and safety control of spherical storage tank
Chapter 8: seismic design and safety control of cylindrical storage tank
Chapter 9: seismic design and safety control of vertical tower and storage tank
Chapter 10: seismic design and safety control of semi-building structures
Chapter 11: seismic design and safety control of pipe lines
Appendix

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 of intended gas distribution facilities in this guideline, especially concrete material characteristics, issue 123 and 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 electrical facilities.

5-Other similar guidelines and documents which occasionally were prepared and developed by internal or external qualified bodies for seismic design of gas distribution systems can be implemented in coordination with this guideline.

6-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

BCJ1997 ,ifcations of seismic design for building componentsspec :Building Center of Japan, 1997

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

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

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

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

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

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

Technical Journal of Telegraph and Telephone Corporation (NTT): design technology of spatial structure (1-3), August, September and October 2007

1-3-2-Code Abbreviations

Abbreviation	Full Name
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 Works Association, 1997
KHK	Technical Seismic Design Code for High Pressure Gas Facilities, High Pressure Gas Safety

Abbreviation	Full Name
	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 following Iran 2800 code 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 and ductility method in non-elastic of material behavior for risk level 2
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 gas 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)

2-In Iran, magnitude (magnitude of momentum) and fault distance must be considered on the basis of occurrence probability of MOE and MCE in the facility sites.

1-In this guideline, useful service life of lifelines including desired gas facilities is considered relatively around 50 years. Maximum operational earthquake may be occurred once or twice during the service of gas facilities. Unacceptable failure modes during operation of facilities are confined to risk level 1 and operation of gas system continues reliably. In this risk level, occurrence probability of 50% during 50 years is in accordance with return period of 72 years. In standard no. 2800, overrun probability is considered around 99.5% which gives return period of about 10 years. 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. For some structures such as bridges which their life are considered up to about 250 years, this overrun probability in risk gives return period of 2475 years that gives overrun probability of about 2% for structures of useful service life of 50years such as buildings and lifelines that is not economic.

In some fields such as dam construction, the term MCE means Maximum Credible Earthquake. In this guideline, this term is adopted from chapter 12 of ASCE-7-05 and used with the same meaning.

2-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 gas 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 natural gas 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-In the application these spectra, following notes must be considered:

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-Analyse of dynamic response is a method for seismic safety control of structure, especially structures with complex behavior under earthquake. These analyses are expensive and time-consuming and only used when there is difficulty and uncertainty in application of response spectra.

3-Following points must be considered regarding to spectrum

3-1-acceleration response spectrum is used for above-ground structures. Also, these spectra are convenient for systems with several degrees of freedom with application of modal analysis method.

In this guideline, acceleration spectrum available in the valid and current version of standard no. 2800 is used for computations relied on acceleration spectrum.

3-2-Velocity response spectra are used for seismic analysis and design of underground structures such as pipelines, shielded tunnels and 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.

3-3-Velocity and acceleration response spectra must be compatible with seismic design of gas distribution 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 gas system components located on other facilities or in the upper stories. in stories simplified coefficient distribution (Ai) use for distribution of seismic intensity(K_H), that is given by equation (2-1). This coefficient multiplied at K_H 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. Vertical seismic intensity of design K_v is given by equation (2-3):

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

Which, K_H 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-Horizontal force capacity method: this method can be applied for risk level 2 with consideration of

maximum lateral forces due to structural deformation in the ultimate mode.
3-Equivalent ductile design method (alternative method) in which the obtained coefficient for earthquake design in risk level 2 is reduced to half of its value and design is performed with allowable stress method.
4-Ductile design method which compare existing ductile ratio of structure with allowable ductile ratio for risk level 2.
Note:
-In the cases which structure is controlled by ductile method, it is required that seismic intensity of 0.3 is controlled with allowable stress.
-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-Generally, allowable stress design method is used in risk level 1 for operation earthquake
2-Horizontal (lateral) force capacity method is one of the control methods. This method has been used in the seismic design of some components of gas distribution systems in this guideline and applied appropriately with related details.
3- Equivalent ductile design method (alternative method) is a method in which component stress is computed using one equivalent elastic design spectrum. This spectrum is obtained from multiplication of factor in one spectrum of elastic seismic design. This factor is determined according to ductility coefficient or structures' capacity for absorb energy. In this guideline, this coefficient is considered to be 0.5.
4-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. Sometimes, ductility coefficient can be described in terms of structural strains.

2-3-Anticipated functions in this guideline

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

- Unceasing usage function (until before material yield)
  Risk level 1: designed components must not damage gas system function effectively and their function must be continued unceasingly.
- Minimum interruption 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.

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.
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.
In general, anticipated classification of function in two limit modes has been considered in other codes and guidelines but in detail, they are different. In this guideline, implementation procedure of these limits according to designed components are presented in chapter 5 and afterward chapters based on allowable

design stress method (for limit mode of damage) and ductile methods of design (for ultimate limit mode) and needed computations and criteria are given.

Chapter 3

Seismic Loading

3-1- Types of Loads

Calculation of loads in gas supply facilities are as follows:

- Dead weight from the equipment and its accessories
- Weight from some materials inside the equipments
- Content's internal pressure (especially in storages, tanks, pipings, and pipeline)
- Water's hydrostatic and hydrodynamic pressures
- Soil's pressure on buried components
- Thermal pressure
- Lateral and vertical pressures caused by earthquake
- Wind's pressure

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

- The wind's pressure is not effective on buried components.
- Unlike buildings, the structures in gas facilities do not have any live human loads.
- Components such as tanks, pipelines and inter-facilities pipelines are greatly under the pressure of their internal materials and products.
- Vents and inter-facilities pipings are under high thermal pressures.

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 forward.

3-4- Types of equipments by their location

The gas supply facilities are generally located as follows:

- Aerial equipments
- On-ground equipments
- Underground and buried equipments

3-5- Seismic loads calculation methods

Earthquake-imposed loads on gas supply 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 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- In some cases for the buried structures which their mass and their internal materials could create inertia due to earthquakes, both the inertia and the force from displacement should be considered.

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 (for on-ground and stationary structures creates inertia force).

b. Velocity (it is more effective than the acceleration in the buried structures, especially in transmission and distribution lines).

c. Displacement (causes serious damages in all structures, especially in the buried lines).

2- The static effect or the so-called geotechnical hazards which results in permanent displacements in soil, including:

a. Liquefaction (and the lateral spread, especially in seas and rivers' shores).

b. Earthquake (in foothills with sharp slopes).

c. Fault (for stationary structures located on faults area and or the buried pipelines through them).

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 gas equipments

1- In order to calculate the imposed load on 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.

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.

3- 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.

4- 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.

3-8-Methods of measuring the loads from earthquake

In order to determine the seismic load imposed on aerial and on-ground gas facilities, usually the "pseudo-static method" is used. Moreover, in necessary cases, the following methods could be applied considering the structure's shape, seismic characteristics, facilities' importance, and destruction's mode:

- Modified pseudo-static method

- Dynamic analysis method (spectral or time-history)

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.

Table 3.1: The qualified equipments for using the pseudo-static method

Type of Equipment	**Applicable Traits**
Tower and vertical vessel	Height from the base plate less than 20m
Spherical tank	Tank's capacity less than 80 tons
Cylindrical tank	Outer diameter and lateral wall height less than 10m
Horizontal tank	Tank's capacity less than 100 tons

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. 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.

3-8-1-Gas supply components importance factor

The structure's importance factor is denoted by β_1 based on its importance and is determined using table 3.2.

Table 3.2: 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 gas supply system. The definition for different categories in table 3.3 and also the importance classification of different equipments are presented in table 3.4.

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

Table 3.3: 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 power supply system and would not lead to casualty and financial and environmental damages.

Table 3.4: Classification of different facilities' importance

Importance \ Equipments	LNG equipments	High-pressure gas equipments	Control and protection equipments
Very high	×	×	×
High	×	×	
Medium and low	×	×	×

3-8-2-Considerations related to determining the importance factor of gas supply facilities

Determining the structures' importance factor is done based on tables 3.5 and 3.6. This classification depends on factors such as the lowest distance from tower's outer surface, tanks and/or piping to the structure's building site boundary (where structures are placed based on towers' capacity, tanks, and/or piping), and type and mass of gas contents in utilization mode. Among the piping system, pipe connections to towers and tanks are of utmost importance. The importance factor determination stages are depicted in figure 3.1.

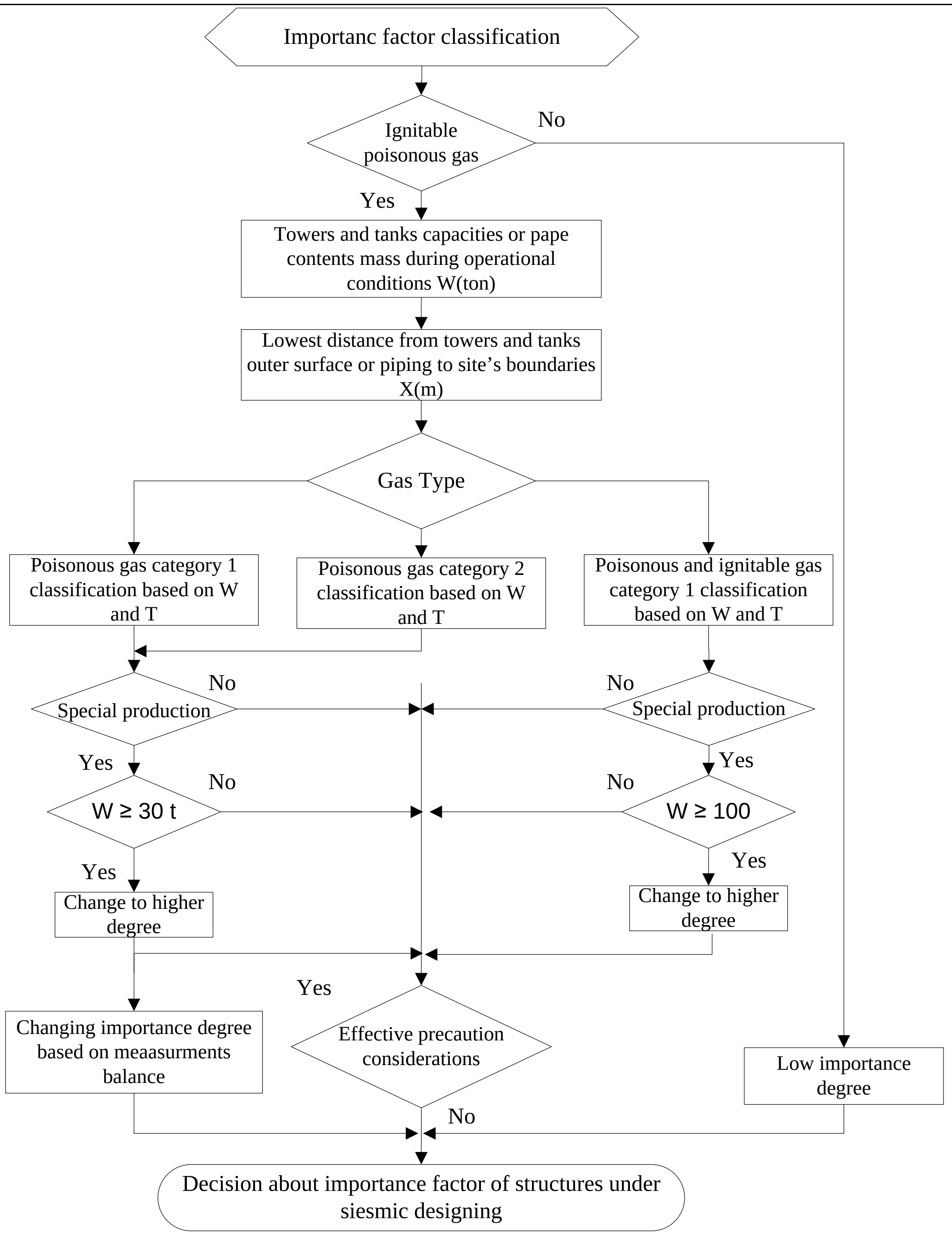


Figure 3.1: Steps of determining the importance factor of gas supply system components

Table 3.5: Determining the (general) importance factor

Gas Type	Tank Capacity W(ton) / Distance X(m)	200 or more	From 100 to 200	From 20 to 100	From 5 to 20	Less than 5
Category 1 of Toxic gases	Less than 100	High	High	High	High	High
	From 100 to 200	High	High	High	High	Medium
	From 200 to 500	High	High	High	Medium	Low
	From 500 to 1000	High	High	Medium	Low	Low
	1000 and more	High	Medium	Low	Low	Low
Category 1 of Toxic gases	Less than 50	High	High	High	High	High
	From 50 to 200	High	High	High	High	Medium
	From 200 to 500	High	High	High	Medium	Low
	From 500 to 1000	High	High	Medium	Low	Low
	1000 and more	High	Medium	Low	Low	Low

Table 3.6: Determining the (general) importance factor

Gas Type	Tank Capacity W(ton) / Distance X(m)	Less than 10	From 10 to 100	From 100 to 1000	From 1000 to 10000	10000 or more
Flammable Gas Category 3 of toxic gases	Less than 20	High	High	High	High	High
	From 20 to 40	Medium	High	High	High	High
	From 40 to 90	Medium	Medium	High	High	High
	From 90 to 200	Low	Medium	Medium	High	High
	From 200 to 400	Low	Low	Medium	Medium	High
	From 400 to 900	Low	Low	Low	Medium	Medium
	From 900 to 2000	Low	Low	Low	Low	Medium
	2000 and more	Low	Low	Low	Low	Low
Regardless of category and distance, other gases are always in category 3.						

Table 3.7: Determining the (special) importance factor

Gas Type	Tank Capacity W(ton) / Distance X(m)	Less than 5	From 5 to 20	From 20 to less than100	From 100 to less than 200	200 and more
Category 1 of Toxic gases	Less than 100	High	High	High	Very High	Very High
	From 100 to 200	Medium	High	High	Very High	Very High
	From 200 to 500	Low	Medium	High	Very High	Very High
	From 500 to 1000	Low	Low	Medium	High	Very High
	1000 and more	Low	Low	Low	Medium	High
Category 1 of Toxic gases	Less than 50	High	High	High	Very High	Very High
	From 50 to 200	Medium	High	High	Very High	Very High
	From 200 to 500	Low	Medium	High	Very High	Very High
	From 500 to 1000	Low	Low	Medium	High	Very High
	1000 and more	Low	Low	Low	Medium	High

Table 3.8: Determining the (special) importance factor

Gas Type	Tank Capacity W(ton) / Distance X(m)	Less than 10	From 10 to 100	From 100 to 1000	From 1000 to 10000	10000 or more
Flammable Gas Category 3 of toxic gases	Less than 20	High	High	Very High	Very High	Very High
	From 20 to 40	Medium	High	Very High	Very High	Very High
	From 40 to 90	Medium	Medium	Very High	Very High	Very High
	From 90 to 200	Low	Medium	High	Very High	Very High
	From 200 to 400	Low	Low	High	High	Very High
	From 400 to 900	Low	Low	Medium	High	High
	From 900 to 2000	Low	Low	Medium	Medium	High
	2000 and more	Low	Low	Medium	Medium	Medium
Regardless of category and distance, other gases are always in category 3.						

1-Changing importance factor

If in category 1 and 2 of toxic gases, the storage capacity is more than 30 tons, or in category 3 of toxic gases and flammable gases is more than 100 tons, the presented importance factor in the above tables should be considered one level higher for towers and pipings. In such cases the high importance factor changes to very high.

2- Gas classification

In order to determine the structures' importance factor, gases are divided into the following five types.

- Category 1 of toxic gases: highly toxic gas
- Category 2 of toxic gases: semi-toxic gas
- Category 3 of toxic gases: gas with low poisoning

- Flammable gas: gases like isobutene, ethane, ethylene, normal butane, propane, LPG, and the following gases:
  - Low explosion possibility (possibility of explosion during mixture with air): 10% or lower
  - Various explosion possibility between high and low: 20% or more

3- Storage capacity

3-1- Towers and tanks:

The storage capacity, W, is as following in determining the importance factor of towers and tanks.

Compressed gas tank

a. The capacity of compressed gas tank could be determined by converting the tank's volume (m^3) to mass (ton) under operational temperature and gas pressure. The conversion equation is as follows.

$$W = \frac{273}{T_t}(10P_0 + 1)\frac{V}{1000 \times 22.4} M_g \tag{3.1}$$

W: tank capacity (ton)

P_o: operational pressure

V_1: Tank's inner volume

M_g: Gas molecular mass (kg/kmol)

T_t: Operational absolute temperature (Kelvin)

b.Liquid gas tank

The liquid gas tank's capacity could be obtained from equation 3.2.

$$W = C_1 w V_1 \tag{3.2}$$

$C_1 = 0.9$

w = density (t/m^3) of liquid gas at tank's operational temperature

V_1 = tank's inner volume (m^3)

Considerations: the operational temperature in seismic design is the temperature in which the liquid gas density is at its maximum while it is stored normally.

c.Tower and intermediate receiving tank

In normal conditions, utilization of towers or tank with gas flow in them, such as tower and intermediate receiving tank (intermediate process tank), include startup to shutdown stages.

d.Piping

Mass, W, of pipe's contents are measured as follows in classification of piping importance factor.

i)Compressed gas piping

Mass, W, of compressed gas piping contents is determined by converting the piping volume (m^3) to mass (ton) in operational temperature and gas pressure. The conversion equation is as follows.

$$W = \frac{273}{T_t}(10P_o + 1)\frac{V_1}{100 \times 22.4} M_g \tag{3.3}$$

W: piping contents mass (ton)

P_o: operational pressure (MPa)

V_1: piping inner volume (m^3)

M_g: gas molecular weight (kg/mol)

T: Operational absolute temperature

ii)Liquid gas piping

Mass of liquid gas piping contents is a value obtained from equation 3.4.

$$W = wV_1 \tag{3.4}$$

W: piping contents mass (ton)

w: liquid gas density at tank's operational temperature (ton/m^3)

V_1: piping inner volume (m^3)

4- Distance

4-1-Towers and tanks

One of the used parameters in classifying the importance factor is the horizontal distance (closest X distance) of towers and tanks' outer surface from the outer boundary of the desired industrial region or the first security center against hazards. However, if the mentioned structures listed from (i) to (iv) which are adjacent to the site are existent, the horizontal distance, X(m), would be the nearest distance situation to the equipments' outer boundary.

(i) Sea, lake, marsh, river, and streams for industrial uses

(ii) Freight railway

(iii) Industrial regions

(iv) Adjacent road or railway to the site

An example is presented in figure 3.2.

b.Piping

Unlike towers and tanks, for piping, the distance from the site's boundary varies from one place to another. The scale of the assumed changes according to being occurred whether in a near or far position. A straight distance to boundaries is assumed and the piping importance factor is then decided. For example, the importance factor is determined based on a 10m distance in figure 3.3.

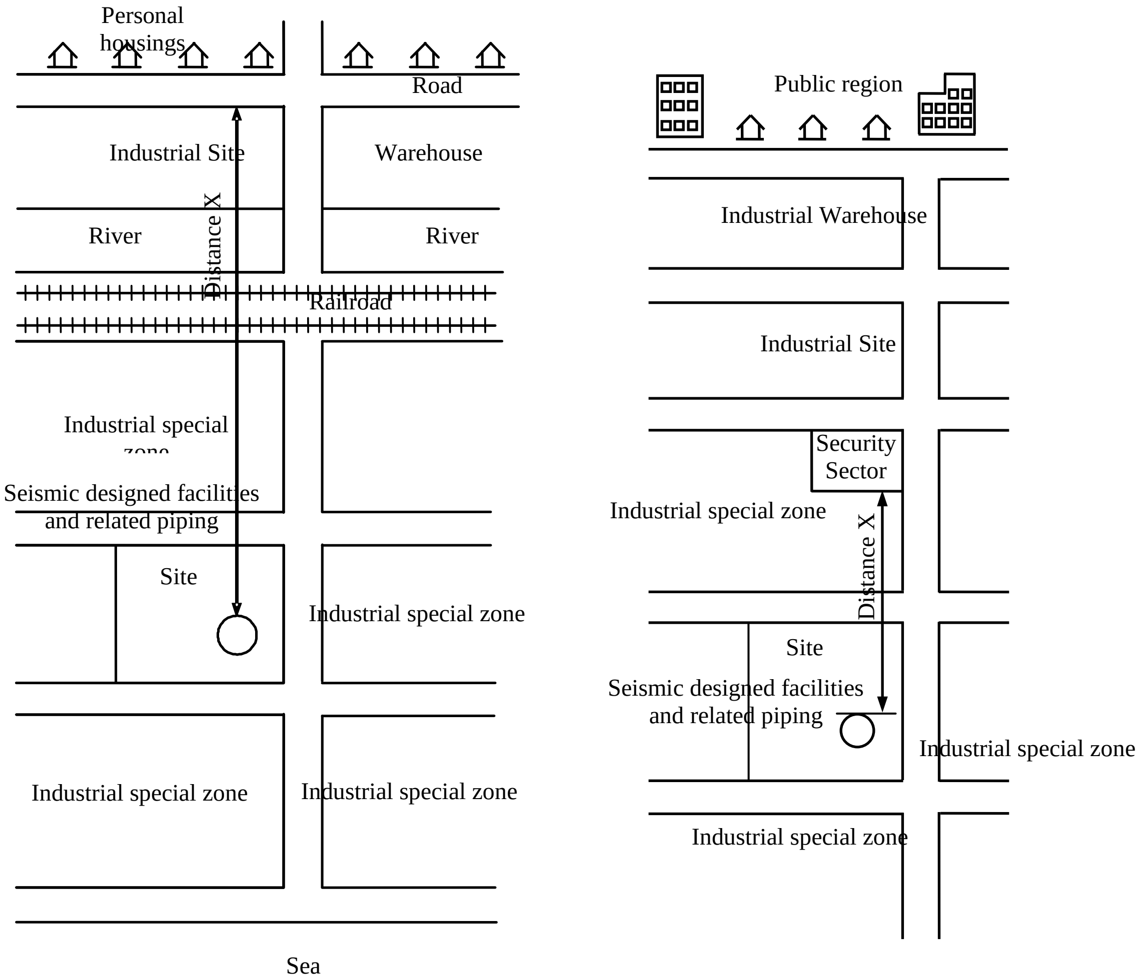


Figure 3.2: an example for determining the distance X

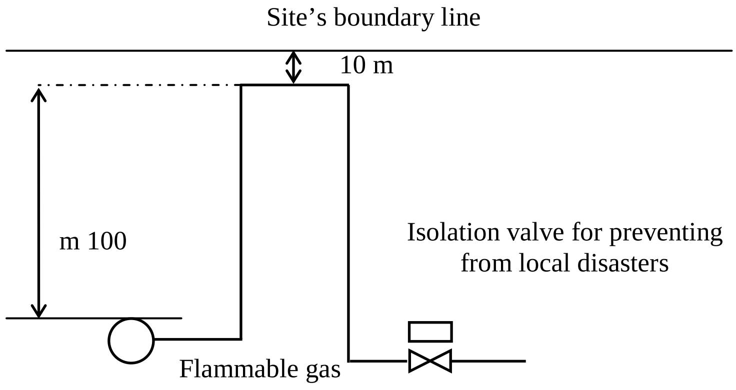


Figure 3.3: piping distance to site's boundary

5-Piping classification based on importance factor

If, due to an earthquake, the piping connection to towers and tanks is damaged, the whole contents of gas inside the connection would flow out. But if there is a separation valve between towers and tank, the leakage would be limited to the volume of gas surrounded by this valve. Therefore, according to the high-pressure gas mass surrounded by the earthquake, tanks, and towers shut-off valve, the piping importance factor could be determined according to figure 3.4.

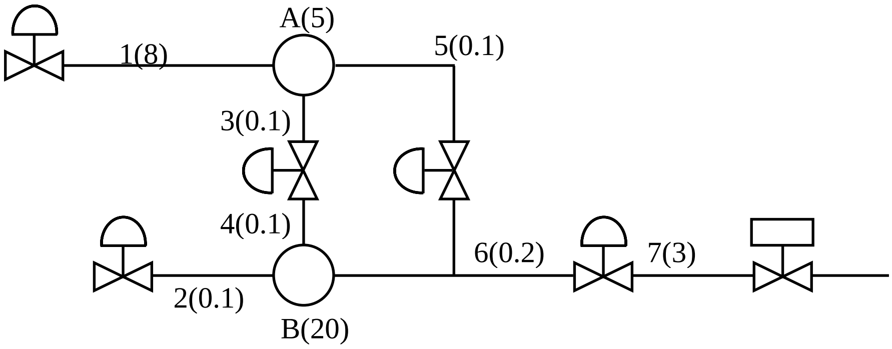
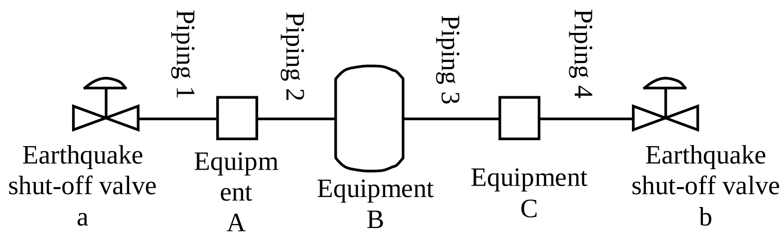


Figure 3.4: Separation of piping cross-section

Piping Number	1	3	5	2	4	6	7
Piping contents mass (t)	**8**	**0.1**	**0.1**	**0.1**	**0.1**	**0.2**	**3**
Capacity of tanks and towers (t)		**5**			**20**		**0**

An example for determining the separated volume of towers and tanks by earthquake shut-off valve is shown in section (a) to (f). In this regard, contents of pipe, expansion joints, and valves are summed up. Also, it is assumed that the piping boundary, towers and tanks are assumed to be the first weld line from the piping nozzle flange panel or the towers and tanks.

a.Existence of a high-pressure gas equipment between earthquake shut-off valves

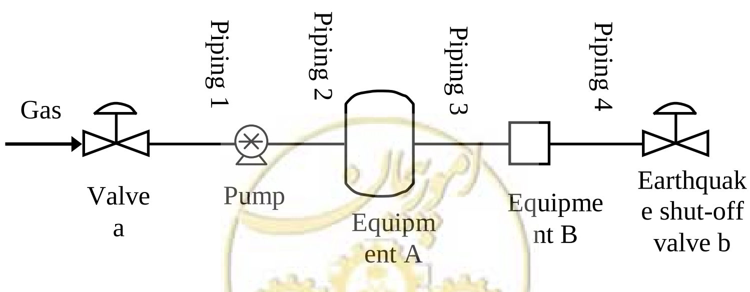


Contents volume = half of contents of earthquake shut-off valve *a* + pipe 1 content + pipe 2 contents + … + pipe 3 contents + pipe 4 contents + half of contents of earthquake shut-off valve *b*

Considerations: the separation location of equipment and pipe is the first flange or first weld line at both ends of equipment.

Figure 3.5: existence of high-pressure equipment between earthquake shut-off valves

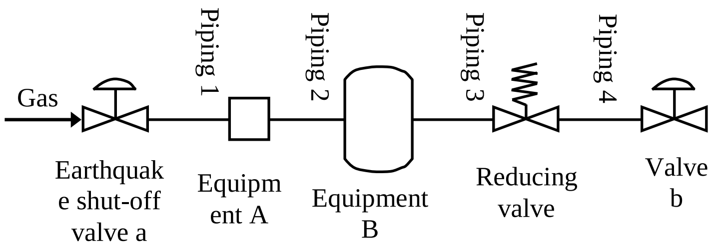
b.High-pressure gas equipments and pump at upstream of earthquake shut-off valves



Contents volume = pipe 2 contents + pipe 3 contents + half of contents of earthquake shut-off valve *b*
Considerations: the separation location of pump and output sector piping is the first flange or first weld line.

Figure 3.6: positioning of the reducing valve at the upstream of high-pressure gas earthquake shut-off valve

when a reducing valve is at downstream of high-pressure gas equipment of earthquake shut-off valve



Contents volume = half of contents volume of earthquake shut-off valve *a* + volume of piping 1 contents + volume of piping 2 contents + volume of piping 3 contents + half of contents volume of earthquake shut-off valve *b*

Figure 3.7: positioning of reducing valve at downstream of earthquake shut-off valve, high-pressure gas equipment

3-9-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.
In the next chapter, the seismic loading of each equipment is presented using the above-mentioned methods.

3-10-Dynamic method

In this method, the mathematical model of equipment is exposed to spectral and/or time-history loading, and is solved by solving the dynamic balance equations. Usually, the dynamic method is used for controlling the static or pseudo-dynamic methods and/or analyzing very important or complex structures.

3-11-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.

3-11-1-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.

3-11-2-Landslide

In mountains regions with high ground slope and weak layers, there is a possibility for landslides and may cause damages to different gas 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

3-11-3-Faulting

It is impossible to avoid regions with faulting potential in gas supply 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.

3-12-Soil classification

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

Chapter 4

Seismic Design and Safety Control Methods

4-1-Target components

The target components in this guideline are as follows:

1-Refinery components
- 1-1-Piping and pipe support
- 1-2-Vessels (lateral and vertical)
- 1-3-Tower
- 1-4-Spherical tank
- 1-5-Large dimensions cylindrical tank (storage tank, storage foundation, tank related equipments, etc.)
- 1-6-Regulators
- 1-7-High-pressure valves, etc.

2-Transport line (high-pressure, 1000 lb/in^2)

3-Basic distribution network lines (medium-pressure, 250 lb/in^2)

4-Distribution network lines (low-pressure, 60 lb/in^2)

5-Gas supply buried lines

6-Accessibilities and manholes

City gas stations (CGS) which reduce the gas pressure from 1000 to 250 lb/in^2, do not have building structures and their designing is stable so that no damage could be inflicted during earthquakes. However, against geotechnical hazards, it should be strictly avoided to build them in grounds susceptible to liquefaction, landslide, and faulting. Town border stations (TBS) and district reduction stations (DRS), in which the gas pressure is reduced from 250 to 60 lb/in^2, are in two types:

1-Indoor *building stations:* these stations are also stable mechanically and ground vibrations have no destructive effect on them. But, any damage to their building may lead to damages in weaker components inside the station, such as reduced diameter pipes, and may result in gas leakage. Hence, it is necessary to design their buildings against earthquake based on 2800-standard.

2-*Cabinet stations:* these stations, which are smaller than indoor stations, are usually placed inside a cabinet-like chamber. According to dimensions and stability of such stations, no damage would be inflicted during earthquakes due to ground shakes.

None of the above stations should be built on grounds susceptible to geotechnical hazards.

4-2- Seismic design procedure

4-2-1-Seismic design necessity

The following structures should be designed against earthquake in case of the given conditions:

1-Tower (height more than 5m)

2-Tank (capacity more than 3 tons or more than 300m^3)

3-Support structure and foundation

4-Piping: pipes with diameters more than 45mm, pipes with volume more than 3m^3 between earthquake shut-off valves, and pipes from chamber to valve.

4-2-2-Principles of seismic design steps

The refinery components, based on the used hazard level type, are designed by either the allowable stress or ductility design methods.
The allowable stress design is used when using hazard level-1.
For cases with hazard level-2, the seismic design is performed using the ductility method.

1- In the allowable stress level method, the members' stresses should not exceed the allowable stress values. Otherwise, permanent deformations would emerge after earthquake.
2- In ductility design method, the plastic deformations which occur in members should be smaller than the allowable plastic deformations. In this case, the equipment's performance is not damaged during and post earthquake.

4-2-3-Allowable stress design method

4-2-3-1-Stress calculation

The calculated stress for the structure is obtained from the summation of stresses caused by inner forces, weight of structure, components and system's loading during normal operation and the stress caused by earthquake during the most severe conditions.

4-2-3-2-Allowable stresses

The allowable stresses for seismic design of facilities are defined in the relative chapters based on the divisions of compressive sector materials, support structural materials, and piping.

The allowable stresses of materials are presented in sector 4.3.

4-2-3-3-Evaluation of the calculated stress

If all the calculated stresses are smaller than the corresponding allowable stresses, the seismic performance evaluation would be acceptable.

4-2-4-Ductility design method

4.2.4.1 Earthquake hazard level for designing

The procedure of seismic design evaluation by the ductility method is used for earthquake hazard level-2.

4-2-4-2-Seismic response analysis

The elastoplasts deformations in each point of structure could be obtained by response analysis under effect of earthquake.

For seismic design of structures, plastic deformations, is obtained from response analysis of one of the following methods.

1-Energy method
2-Equivalent linear response method
3-Non-linear response analysis method

The general discussions for these methods are as follows:

a) Energy method

i) Ultimate plastic deformation design method

In structures under seismic design and with dominant first vibration mode, the ductility coefficient could be obtained from each destruction mode's energy conservation law.

- Design's modified earthquake factor

The modified earthquake coefficient used in structure's seismic design is obtained from normalized response magnification coefficient.

- Ductility factor

The ductility factor, μ_p, of the structure's damaged sectors is calculated from equation 4.1.

$$\mu_p = \frac{1}{4C}\left\{\left(\frac{K_{MH}}{K_y}\right)^2 - 1\right\} \tag{4.1}$$

μ_p: ductility coefficient of member related to destruction mode, if $K_y \leq K_{MH}$ then $\mu_p = 0$.
K_{MH}: modified horizontal earthquake coefficient
K_y: horizontal earthquake coefficient at the start of damaged member's yielding
C: constant, determined according to the damage mode

- Plastic deformation evaluation

$$\mu_p \leq \mu_{pa} \tag{4.2}$$

μ_{pa}: allowable ductility coefficient

ii) Yield strength designing method

For frame-shaped structures and foundations, the plastic seismic design is performed by yield strength method due to the following steps.

- Design modified earthquake coefficient calculation

It is calculated similar to the pseudo-static method (K_{MH}).

- Structural characteristic factor

Structural characteristic factor, D_s, is obtained from equation 4.3 or values presented in other regulations. D_s should be ranged from 0.25 to 0.5.

$$D_s = \frac{1}{\sqrt{1+4C\mu_{pa}}} \tag{3.4}$$

D_s: structural characteristic factor

- Final seismic demand calculation

The seismic demand is obtained from equation 4.4.

$$Q_u = K_y \times W_H \tag{4.4}$$

Q_u: Final seismic demand
W_H: Structure's operational weight

- Calculating current seismic demand

$$Q_{un} = D_s \times K_{MH} \times W_H \tag{4.5}$$

Q_{un}: current seismic demand

- The displacement response is calculated from the considered component under seismic loading
- Evaluation of seismic demand

The current seismic demand, Q_{un}, of the structure should not exceed the ultimate seismic demand, Q_u. ($Q_{un} < Q_u$)

b) Equivalent linear response method

In case of members with non-linear behavior, which it capacity exceeded the allowable capacity, the linear response analysis could be carried out by reducing stiffness from elastic stiffness, according to the amount of non-linearity and the equivalent damping coefficient.

i) Linear modal response analysis, using the acceleration response analysis, is performed based on steps (1) to (6).

(1) Design's horizontal and vertical acceleration spectrum are calculated according to equations 4.6 and 4.7.

$$A_H^{(i)} = 350\,\beta_1\beta_2\beta_5 \tag{4.6}$$

$$A_V^{(i)} = 175\beta_1\beta_2\beta_6 \tag{4.7}$$

$A_H^{(i)}$: Design's horizontal response acceleration of vibration's first mode (unit gal)

$A_V^{(i)}$: Design's vertical response acceleration of vibration's first mode (unit gal)

β_1: importance factor

β_2: design's basis acceleration ratio

β_3: building site's amplification factor

β_5: horizontal response magnification factor

β_6: vertical response magnification factor

(2) The member's stiffness should be reduced depending on the non-linearity degree.

(3) The equivalent damping coefficient to the plastic strain energy which is obtained from the non-linear response could be used.

(4) The value of response, R, such as shear force, moment, acceleration, and design displacement, is calculated from appropriate combined method for each vibration mode.

$$R = \sqrt{\sum_i R_i^{\,2}} \tag{4-8}$$

Where, R_i is i^{th} value for response mode.

(5) The response displacement is obtained from behavior analysis in different modes.

(6) The ductility factor obtained from (5) should not exceed the allowable ductility factor.

ii) Response analysis using the equivalent load method

Response analysis using the equivalent load method is carried out through the following steps.

(1) The equivalent load is determined via an appropriate method. If the structure could be modeled as a system with one degree of freedom, the equivalent load could be calculated by multiplying member's weight to modified earthquake factor.

(2) The member's stiffness should be reduced relative to its non-linearity degree.

(3) For damping factor, the damping factor equivalent to the plastic strain energy which is obtained from structure's non-linear analysis could be used.

(4) The response displacement is obtained from behavior analysis in different modes.

(5) The ductility factor obtained from (4) should not exceed the allowable ductility factor.

iii) Equivalent displacement method

The equivalent displacement method is carried out through the following steps.

(1) The enforced displacement at supports should be equal to the support structure's response displacement or the displacement induced in the foundation due to ground movement.

(2) The stiffness of the designed member is reduced relative to the non-linearity degree.

(3) The ductility factor is obtained from member's displacement.

(4) The ductility factor from (3) should not exceed the allowable ductility factor.

c) Non-linear response analysis method

i) Time-history response analysis

This analysis is as follows:

(1) Characteristics of load-deformation should be defined as nonlinear cyclic model and the results should be obtained directly from time-history analysis.

(2) Imposing the earthquake's wave with maximum acceleration at a desired point.

(3) The ductility factor is obtained from the member's displacement.

(4) The ductility factor from (3) should not exceed the allowable ductility factor.

ii) Static nonlinear response analysis in equivalent load method

The static nonlinear response analysis from equivalent load method is performed based on the following the steps.

(1) The equivalent load is determined from an appropriate method. If the structure could be modeled as a system with one degree of freedom, the equivalent load could be calculated by multiplying member's weight to modified earthquake factor.

(2) The modified earthquake factor could be obtained using the damping coefficient which is equivalent to the plastic strain energy caused by the structure's nonlinear response.

(3) The member's displacement could be obtained by model's static analysis using the nonlinear load-deformation relationship.

(4) The ductility factor is obtained from the member's displacement.

(5) The ductility factor from (4) should not exceed the allowable ductility factor.

4-2-4-3-Ductility factor

The ductility factor of the damaged area is determined from the plastic deformation which is obtained by response analysis corresponding to the considered equipment's damage mode, in designing due to design's earthquake movement (horizontal and vertical movements). The force in vertical direction is considered for the part of damage which is susceptible to intensification based on structural conditions and different facilities' damage mode.

4-2-4-4-Allowable ductility factor

The member's allowable ductility factor, considering the characteristics of plastic deformation, like fatigue and buckling according to the elastoplastic distortion with small-period loading, is determined in conformity with the equipment's damage mode for earthquake, at its worst conditions.

4-2-4-5-Ductility factor evaluation

If the allowable ductility factor for all main members is equal or larger than the desired ductility factor, the seismic performance assessment would be acceptable.

4-3-Materials

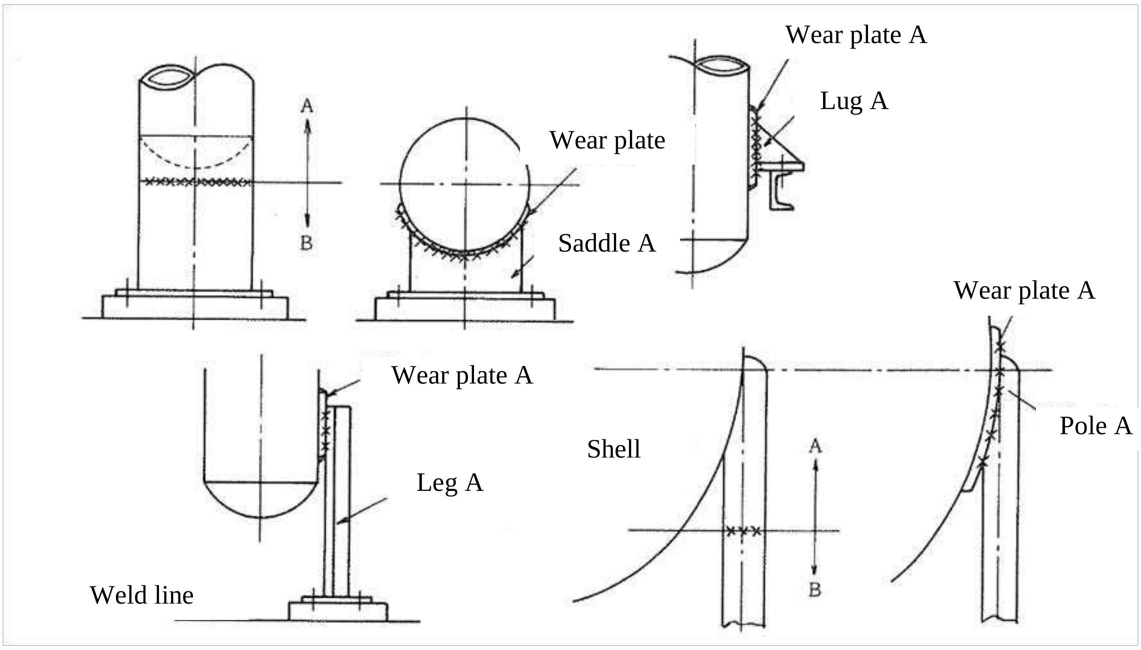
4-3-1-Allowable values for allowable stress design method

In the allowable stress design method, according to the materials of the compressive part, support structure's materials, foundation, and ground, different allowable stresses are used for seismic design.

If the support structure is directly welded to the materials of the compressive part, between the allowable stress values of the compressive part materials and support structure's material, the smaller value is used.

Conditions for applying compressive part materials and support structure materials:

The first weld line on which the allowable stress conditions of compressive part materials and support structure materials is applied, is presented in figure 4.1.



A: the part, in spite of support structure materials, needs to study the allowable stresses for compressive part materials' seismic design.

B: the part on which only the support structure's materials seismic allowable stress is applied.

Figure 4.1: Conditions for applying allowable stress in seismic design of compressive part materials and support structure materials

The least requirements for support members are presented in table 4.1.

Table 4.1: Minimum requirements for support members' materials

Operational temperature	Compressive part materials	Support structure's materials which are directly welded to the compressive part materials
-10°C or more	SPV450 and SPV490 (For normal temperature)	Materials with characteristics equivalent to ASTM A573 Gr.70 or more.
	HW685 (For normal temperature)	Materials with characteristics equivalent to ASTM A678 GR. C or more.
Less than -10°C	SPV450 and SPV490 (For low temperature)	Omitted
	HW685 (For low temperature)	Omitted

Table 4.3: Allowable stress related to seismic design based on materials' type

Type of Materials	S
Aluminum and 9% nickel-steel alloy which is used at temperatures lower than room temperature	The minimum of the following values 0.6 Su 0.9 Sy
Other materials	The minimum of the following values 0.6 Su0 0.6 Su 0.9 Sy0 Sy

Table 4.4: The allowable stress related to the starting point of buckling

Type of Materials	S'
Towers and horizontal cylindrical tanks	$\dfrac{0.6Et}{\left(1+0.004\dfrac{E}{S'_y}\right)D_m}$
Cylindrical tanks	$\dfrac{Et}{3D}$

In the above tables:

Su: tensile strength at material's design temperature, (design temperature between 0 and 40°C), and is assumed lower than the materials standard's given value.

S_{u0}: tensile strength at normal material temperature and is assumed as a value lower than the standard's given value.

S_y: tensile yield strength or equivalent strength to 0.2% strain at materials' design temperature, the design temperature is between 0 and 40 and is lower than the value given in the standard.

S_{y0}: tensile yield strength or equivalent strength to 0.2% strain at material's normal temperature, and is lower than the given value in the materials' standard.

$S_{y'}$: the minimum value between S_y and S_{y0} (N/mm^2)

E: modulus of elasticity at materials' design temperature (N/mm^2)

D_m: shell mean diameter (mm)

t: thickness of shell or lateral plate (mm)

D: tank’s inner diameter (mm)

4-3-1-1-Allowable stress for compression materials

The allowable seismic design of compression materials is a value which is obtained by multiplying a coefficient to tensile stress or yield stress or the equivalent stress to 0.2% strain at material design temperature and based on the stress’s type.

4-3-1-2-Seismic design allowable stress of support structure materials

The seismic design allowable stress of support structure materials is obtained by multiplying a coefficient to the minimum value between yield stress and tensile stress equivalent to 0.2% strain.
Next, the allowable stress of support structural metallic materials are presented. The allowable stresses of structural steel should be in conformity with AISC regulations.

1-Support structure materials

The allowable stress (N/mm^2) of support structure materials, which are not directly welded to the compressive part materials, is presented in table 4.5 based on stress type. If necessary and combined stresses, table 4.7 should be used for support structure materials according to the combination type.

Table 4.5: Allowable stress of support structure materials for seismic design

Type of Stress	Allowable Stress of Seismic Design	
Tensile stress	F	
Compressive stress	F	
Compressive stress	(a) Skirt	Minimum value of S’
	(b) Saddle	F
	(c) support structure materials other than (a) and (b)	Minimum value of F or F’
Shear stress	$\sqrt{3}F$	

F: minimum value between 70% of yield strength and tensile strength equivalent to 0.2% strain (N/mm^2)

F’: allowable compressive stress of buckling considering the slenderness ratio of member (N/mm^2)

S’: value obtained from table 4.4

Note: evaluation of allowable compressive stress, F’, for buckling considering member’s slenderness ration is carried out as follows.

In case $\lambda_s \leq \Lambda$:

$$F' = \frac{1.5\left\{1 - 0.4\left(\frac{\lambda_s}{\Lambda}\right)^2\right\}F}{v} \qquad (4\text{-}9)$$

$$F' = \frac{1.5\left\{1 - 0.4\left(\frac{\lambda_s}{\Lambda}\right)^2\right\}F}{\nu}$$

In case $\lambda_s > \Lambda$:

$$F' = \frac{1.5 \times (0.277\,F)}{\left(\frac{\lambda_s}{\Lambda}\right)^2} \tag{4.10}$$

Where λ_s is the slenderness ratio for compressive member and is obtained from equation 4.11.

$$\lambda_s = \frac{l_k}{i} \tag{4.11}$$

L_k: buckling length (mm) presented in table 4.6 considering the type of support at both ends of member

i: surface radius of gyration related to buckling axis (mm)

Table 4.6: Buckling length

Movement conditions	Constraints		
Rotation conditions	Free ends	Fixed ends	One fixed and one free end
l_k	l	0.5l	0.7l

l: length of member (mm)

Λ: limit of slenderness ratio obtained from equation 4.12.

$$\Lambda = \sqrt{\frac{\pi^2 E}{0.6 F}} \tag{4.12}$$

v: value obtained from equation 4.13.

$$v = \frac{3}{2} + \frac{2}{3}\left(\frac{\lambda_s}{\Lambda}\right)^2 \tag{4.13}$$

Table 4.7: Stress combination

Type of Stress Combination	Control Equation
Combination of ompression and bending stresses	$\frac{\sigma_c}{f_c} + \frac{\sigma_b}{f_b} \le 1$
Combination of tensile and bending stresses	$\frac{\sigma_t}{f_t} + \frac{\sigma_b}{f_b} \le 1$
Combination of compression, bending, and shear	$\sqrt{(\sigma_c + \sigma_b)^2 + 3\tau^2} \le f_t$
Combination of tensile and shear stresses (limited to the anchor bolt)	$\frac{\sigma_t + 1.6\tau}{1.4} \le f_t$

f_c: allowable compressive stress of support structure materials for seismic design, obtained from table 4.5 (N/mm^2)

f_b: allowable bending stress of support structure materials for seismic design, obtained from table 4.5 (N/mm^2)

f_t: allowable tensile stress of support structure materials for seismic design, obtained from table 4.5 (N/mm^2)

σ_c: created compressive stress in support structure materials (N/mm^2)

σ_b: created bending stress in support structure materials (N/mm^2)
σ_t: created bending stress in support structure materials (N/mm^2)
τ: created shear stress in support structure materials (N/mm^2)

2-Support structure materials directly welded to compressive part materials

In support structure materials directly welded to compressive part materials, the allowable stress is similar to compressive part materials.

Since it is necessary for these materials to have support structure performance, the following conditions should be established.

- The allowable stress elated to seismic design is the minimum value between the values obtained from table 4.5.
- In case of stress combination, the stress combination control equation from table 4.7 should be satisfied.

4-3-1-3-Piping materials allowable stress

a) Piping allowable stress

Table 4.8: Piping allowable stress for seismic design

Type of Stress	Allowable Stress for Seismic Design
Longitudinal stress of piping	S
Cyclic stress range	$2S_y$

b) Flange connection allowable stress

Table 4.9: Flange connection allowable stress for seismic design

Type of Stress	Allowable Stress for Seismic Design
Flange radial stress	S
Flange circumferential stress	S
Axial stress of pipe's hub	$2S_y$

c) Valve allowable stress

Table 4.10: Valve allowable stress for seismic design

Type of Valve	Allowable Stress for Seismic Design
Earthquake shut-off valve	0.5S
Other valves	S

d) Expansion joint allowable stress

The joint is exposed udder 500 cycles loading and the final load is determined.

e) Allowable stress for seismic design of towers and tanks

Table 4.11: Towers and tanks allowable stress

Strength type of Stress	Allowable Stress of Seismic Design
(a) general primary membrane stress	S
(b) Sum of local membrane primary stress and bending primary stress	1.5S
(c) difference between maximum and minimum value of sum of local membrane primary stress, bending primary stress strength, and secondary stress strength in a cycle	$2S_y$

4-3-2-Allowable value for ductile design method

The ductile designing of refinery components is basically carried out using the plastic deformation method and the frame structure is also designed by the yield strength method.

The ductility coefficient used in the plastic deformation method and the structural characteristic factor used in the yield strength method are described in the designing steps of each equipment.

Furthermore, since the allowable limit depends on the evaluation of each piping's part, the tolerance for ductile designing of each part is presented in the corresponding sector of piping designing.

4-4-Acceptable performance level of component and seismic input

4-4-1-Required seismic performance

The seismic performance required for structures (towers and tanks, piping, foundation, etc.) is as follows.

(a) In operational earthquake at hazard level-1, no destructive deformation is inflicted. Gas support structures should not receive any physical damages at this hazard level and based on allowable stress method.

(b) At earthquake hazard level-2, the equipment must continue its operation while received tiny damages. Therefore, the design criteria are appropriately for this hazard level and based on their ductility.

The seismic performance of facilities to prevent earthquake hazards (earthquake shut-off valve…) are as follows.

(a) Preventing from high-pressure gas leakage in equipment and preventing the incidents caused from this leakage by cutting the gas flow

(b) Attempt to prevent from occurrence and spread of damages caused by earthquake, in cases where the seismic performance of the structure fails.

4-4-2-Seismic performance evaluation

1-Seismic performance evaluation of facilities should be performed using the following methods.

1-1-The following cases are applied for operational earthquake at hazard level-1.

a) Performing the response analysis for the plan's seismic movement at normal operation state, the created stress in the important member should not exceed materials' allowable stress.

b) In clause 1, plan's seismic movement evaluation for inertia force (liquid rigid movement mode) and also in order to calculate turbulence in cylindrical tank (liquid turbulence mode) could be performed separately.

1-2-The following cases are applied at plan's earthquake at hazard level-2.

a) The value obtained from dividing the materials response non-elastic displacement to yield displacement (from now on called "ductility factor"), which is occurred in an important member by ductility design method, should not exceed the value obtained from dividing maximum acceptable plastic deformation of materials to yield deformation (from now on called "allowable ductility factor"), which is obtained from design's seismic movement response evaluation at normal operation state.

b) In clause 1, design's seismic movement evaluation for inertia force (liquid rigid movement mode) and also in order to calculate turbulence in cylindrical tank (liquid turbulence mode) could be performed separately.

2-Seismic performance evaluation of piping system should be carried out as follows.

2-1-At operational earthquake the following clauses should be applied.

a) The created stress in an important member, obtained from response analysis for design's seismic movement at normal operational state should not exceed the allowable stress of materials.

b) If the piping system is of medium or low importance factor, designing should be performed through allowable span method and clause 1 could be ignored.

2-2-During earthquake the following cases are applied.

a) Ductility factor should not exceed the ratio of allowable plastic deformation of materials, obtained from appropriate response analysis for design's seismic movement at normal operation state.

b) In clause 1, the design's seismic evaluation for inertia force (liquid rigid movement mode) and also in order to calculate turbulence in cylindrical tank (liquid turbulence mode) could be performed separately.

Chapter 5

Seismic Design and Safety Control of Piping and Pipe Rack

5-1-Piping seismic evaluation steps

In cases with low-importance structure, there is no need for seismic design method and designing is performed using piping support simplified evaluation methods (pipe's allowable span method).

The seismic design should be performed for the following pipes:

(a) Pipes with outer diameter 45mm or more

(b)Pipe's contents $3m^3$ or more

(c)Pipes connected to towers and tanks

5–2-Simplified evaluation method (allowable span method)

If the importance degree of the piping system is medium or low, the length of pipe's span would be smaller than the allowable span length and also the deformation absorption capacity for pipe in different support structures should be higher than the structure's relative deformation, then the seismic performance evaluation is finished. The following cases should be considered in applying the allowable span method:

(a) Performance evaluation should be carried out in each profile of pipe between two fixed support points. If the piping profile is variable, evaluation is performed by unifying the section similar to the equation presented in appendix 1.

(b)The allowable span method should be used for the following cases:

- The longest piping system span from each section
- Spans with concentrated load

(c) If the pipe's support point has different support structures, the displacement capacity evaluation should be performed and if the pipe's span has junctions and the junction pipe's external diameter is half or smaller than the main pipe's diameter, the evaluation should be performed from the junction point to the first junction pipe support point.

(d) The evaluation of (b) and (c) clauses are carried out in all three directions (two horizontal and one vertical direction). If the pipes span has expansion junction, the deformation capacity evaluation is performed considering the junction's capacity.

5-2-1-Allowable span length

The pipe must be constrained against earthquake in three directions, two perpendiculars to and one parallel with the piping axis.

The pipe length between adjacent support structures (pipe span length), which has an efficient support performance in the earthquake direction, should not exceed the allowable span length corresponding to high-pressure gas operation state and external diameter. The acceptance criteria of the allowable span method are shown by equation 5.1.

$$L_{ps} \leq L \qquad (5.1)$$

L_{ps}: pipe's span length calculated in 1.1 of appendix (m)

L_a: allowable span length calculated in 1.2 of appendix (m)

5-2-2-Displacement capacity

The relative displacement of piping in the earthquake's direction between supports should not exceed the piping's displacement capacity.
When the pipe's span is on different supports, the evaluation of displacement capacity should be carried out and if the pipe's span has junctions and the external diameter if the junction pipe is half or smaller than the main pipe's diameter, the evaluation must be performed from the junction point to the first support point of the junction pipe.
Displacement capacity evaluation should be performed as follows:

$$\Delta \leq \delta_a \tag{5.2}$$

Δ: relative displacement between two supports or between junction and the first pipe's junction support in 4.1 of appendix (mm)
δ_a: pipe's span displacement capacity in the direction of earthquake calculated by 3.1 of appendix (mm)

5-3-Allowable stress design method

The seismic design of piping systems at risk level-1 is carried out by the allowable stress method using the allowable span method or detailed analysis.
The standard seismic design procedure of piping system is presented in section 2 of the appendix.
The piping system's behavior could be measured by the response analysis as an integrated system, but for simplification, the response analysis of pipe support structure, and piping, is performed separately.

5–3-1-Piping's support structure response analysis

The horizontal earthquake factor and the pipe's support point response displacement are calculated using the pipe's structure response analysis.
In analysis of support structure response, the pipe's rigidity is ignored and the piping's weight is imposed as a load to the support structure.
To analyze the support structure response with high and very high importance, the modified pseudo-static method and the modal analysis method are used.
The time-history response analysis method is also could be applied.
For support structures with medium and low importance, the pseudo-static method could be used.
The piping's support structure response analysis details are presented in section 3 of the appendix.

5-3-2-Piping's support structure seismic performance evaluation

In seismic performance evaluation of piping support structure (support and equipments of piping support), the created stress in materials should be smaller than the design allowable stresses during earthquake.
In case of performing seismic design of piping support equipments, their seismic performance evaluation could be done by equipments evaluation methods. Moreover, the support performance evaluation method is shown in section 11 of the appendix.

5-3-3-Piping system response analysis

The calculation of static stress of piping system is performed for modified earthquake force, piping's

support response displacement, pressure, and weight of pipe and its contents.
The analysis is carried out as follows by modeling the piping system.
This analysis is done using the modified pseudo-static method and then the design earthquake force is obtained from 5.3 and 5.4.

$$F_{MH} = \beta_8 \mu K_{MH} W_H \tag{5.3}$$

$$F_{MV} = \beta_9 K_{MV} W_V \tag{5.4}$$

F_{MH}: design's modified horizontal earthquake force (N)
F_{MV}: design's modified vertical earthquake force (N)
β_8: magnification factor of piping support structure horizontal response which is equal to 2. If the piping support structure is for towers and tanks, the structure's response magnification coefficient, β8, must be multiplied by 2. Additionally, at the valve's installation location, the response magnification coefficient must be multiplied by 1 to 3 depending on the structure and valve's support type (according to table 5.1).
β_9: vertical response magnification coefficient of piping's support structure which is equal to 2.
μK_{MH}: calculated value based on response analysis type of piping support structure, using design horizontal earthquake factor at the piping support point (appendix section 3).
K_{MV}: modified design vertical earthquake factor
W_H: driver's weight (N). In this case, the corrosion allowable weight is considered.
W_V: sum of contents' weight and piping's dead weight imposed on the location where the design's modified vertical earthquake force is calculated (N). In this case, the corrosion allowable weight is considered (appendix section 4).

5-3-4-Piping stress calculation

The piping stress at the longitudinal direction caused by the seismic force, fluid pressure, and the driver's weight is calculated by combining operation and earthquake loads.

1-Longitudinal direction stress

The longitudinal direction stress caused by the fluid pressure, driver's weight, design horizontal earthquake force, and design's vertical earthquake force for the curved part of the piping, junction area, and the support section of piping, is calculated by equation 5.5.

$$\sigma_\ell = \frac{\sqrt{(i_i M_i)^2 + (i_0 M_0)^2}}{Z} + \left| \frac{F_T}{A_P} \right| \tag{5.5}$$

σ_l: longitudinal stress caused by pressure, weight, and earthquake force (N/mm^2)
i_i: in-plane stress intensification factor calculated using the appropriate method depending on the pipe's connection type
i_o: out-plane stress intensification factor calculated using the appropriate method depending on the pipe's connection type
M_i: in-plane bending moment of piping caused by fluid pressure, driver weight, design's horizontal and vertical earthquake force imposed on piping (refer to the appendix section 5) (N.mm)

M_o: out-plane bending moment of piping caused by fluid pressure, driver weight, design's horizontal and vertical earthquake force imposed on piping (refer to the appendix section 5) (N.mm)

Z: section modulus of pipe which its calculations are performed ignoring the corrosion allowable value. For junction pipe with different diameters, equation 5.6 is applied (mm^3).

$$Z = \pi(r_p)^2 t_s \tag{5.6}$$

r_p: pipe's average radius at curvature location (mm)

t_s: pipe's effective thickness at the curvature (location) where the allowable corrosion thickness and the reinforcement plate are not considered (mm).

F_T: pipe's axial force caused by fluid's pressure, driver weight, and design's horizontal and vertical earthquake forces imposed on piping (N)

A_P: pipe's section area in which the corrosion allowable value is ignored (mm^2)

2-Cyclic stress based on earthquake force

The cyclic stress based on horizontal earthquake force, design's vertical seismic force, and displacement of piping's support point is calculated using equation 5.7:

$$\sigma_E = 2\frac{\sqrt{(i_i M_i)^2 + (i_0 M_0)^2 + M_t^2}}{Z} \tag{5.7}$$

σ_E: cyclic stress (N/mm^2)

M_i: in-plane bending moment of piping caused by design's horizontal earthquake force, design's vertical earthquake force imposed on piping, and displacement of piping's support (table in the appendix section 5) (N.mm)

M_o: out-plane bending moment of piping according to horizontal earthquake force, design's vertical earthquake force imposed on piping, and displacement of piping support (table of the appendix section 5) (N.mm)

M_t: torsion moment of piping according to horizontal earthquake force, design's vertical earthquake force imposed on piping, and displacement of piping support (refer to the appendix section 5) (N.mm)

5-3-5-Piping tension evaluation

If the calculated stress do not exceed the allowable stress, the evaluation would be acceptable.

In cases where the calculated stress exceeds the allowable stress, the structure's characteristics and support conditions are changed and the seismic performance evaluation is repeated. The allowable stresses are presented in section 6 of the appendix.

5-3-6-Flange connection seismic performance evaluation

The leakage evaluation due to the axial force or the banding moment calculated by the acceleration response analysis and the displacement response analysis of the piping should be performed. The following cases are considered in this evaluation:

– This evaluation should be carried out on the flange connection and its surroundings.

– Lack of leakage at the flange connection is only accepted when the created stresses at the connection (flange radial stress, flange circumferential stress, and pipe's hub axial stress) are smaller than the seismic design stress.

- The seismic performance evaluation of flange connecting with medium and low importance could be ignored.
- Calculations related to flange connections are as follows:

1-Calculation of total equivalent pressures

The internal equivalent pressure, P_e, caused by the axial tensile force, F_T (N), and bending moment, M (N.mm) caused by the seismic load, are obtained using equation 5.8:

$$P_e = \frac{4F_T}{\pi D_e^2} + \frac{16M}{\pi D_e^3} \qquad (5.8)$$

P_e: equivalent pressure during earthquake (MPa)
F_T: axial tensile force caused by earthquake (N)
M: bending moment (N.mm)
D_e: average diameter on gasket's contact surface (mm)

$$D_e = D_{gi} + 2(N_g - b_g) \qquad (5.9)$$

D_{gi}: gasket internal diameter (mm)
N_g: gasket width (mm)
B_g: gasket effective width (mm)

The total equivalent pressure, P_{eq}, is obtained from equation 5.10 and using the fluid's pressure, P_P, in piping and the equivalent pressure during earthquake. The total equivalent pressure is used for calculating the load which is imposed on the inner surface (diameter) of the flange.

$$P_{eq} = P_P + P_e \qquad (5.10)$$

P_{eq}: total equivalent pressure (MPa)
P_P = fluid's pressure inside pipe (MPa)

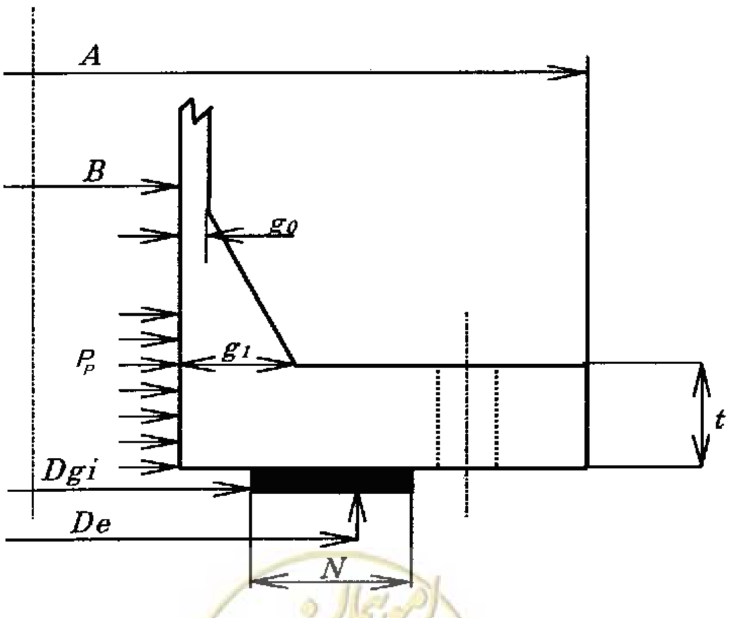


Figure 5.1: Flange with flat gasket

2-Stress measurement

The stress in flange connections is calculated at operation state. The total equivalent pressure is used for calculating the load which is imposed on the inner surface (diameter) of flange due to the total load, H, and the flange's internal pressure.

Stresses in different types of flanges without hub are measure using equations 5.11 to 5.13 and in flanges with hub, equations 5.14 to 5.16 are used.

Axial stress of pipe's hub

$$\sigma_H = 0 \quad (5.11)$$

Radial stress of flange

$$\sigma_R = 0 \quad (5.12)$$

Ring stress of flange

$$\sigma_T = \frac{YM}{t^2 B_f} \quad (5.13)$$

The single-piece flange and the loose flange stress considering the pipe's hub is obtained using equation 5.14:

Axial stress of pipe's hub

$$\sigma_H = \frac{fM}{Lg_1^2 B_f} \quad (5.14)$$

Radial stress of flange (N/mm^2)

$$\sigma_R = \frac{(1.33t_e + 1)M}{Lg_1{}^2 B_f} \quad (5.15)$$

Ring stress of flange (N/mm^2)

$$\sigma_T = \frac{YM}{t^2 B_f} - Z\sigma_R \quad (5.16)$$

σ_H: axial stress of collar (N/mm^2)

B_f: flange's internal diameter. In calculating the axial stress of pipe's hub, when B_f is smaller than $20g_1$, B_1 could be used instead of B_f.

B_1: $B_f + g_0$ in multi-purpose flange, and $B_f + g_1$ in loose flange

f: modified factor of collar stress determined based on the value of g_1/g_0 and h_h/h_0. See diagram in section 7 of the appendix.

F: determined factor based on the value of g_1/g_0 and h_h/h_0. See section 7 of the appendix.

F_L: determined factor based on the value of g_1/g_0 and h_h/h_0. See section 7 of the appendix.

h_h: collar's length (mm)

h_0: equal to $\sqrt{Bg_0}$

g_0: pipe's hub thickness (mm)

g_1: collar thickness of flange's back surface (mm)

L: a factor and is equal to $(t_e + 1)/T + t_f{}^3/d$

d: a factor, for integrated flange is $\frac{U}{V} h_o g_o{}^2$ and for loose flange is $\frac{U}{V_L} h_o g_o{}^2$

M: moment imposed on flange considering the equivalent pressure due to seismic force (N.mm)

T: determined factor based on the value of K = (A/B_f). See section 7 of the appendix.

A: flange's external diameter (mm)

T_f: flange's thickness (mm)

U: determined factor based on the value of $K = (A/B_f)$. See section 7 of the appendix.
V: determined factor based on the value of g_1/g_0 and h_h/h_0. See section 7 of the appendix.
V_L: determined factor based on the value of g_1/g_0 and h_h/h_0. See section 7 of the appendix.
Y: determined factor based on the value of $K = (A/B_f)$. See section 7 of the appendix.

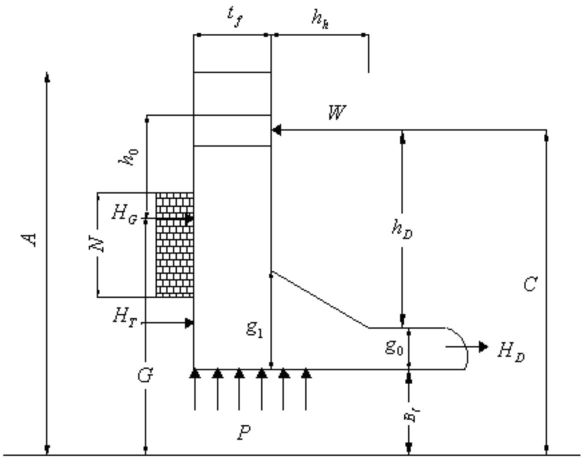


Figure 5.2: flange parameters with flat gasket (pipe's internal side-pipe's axis)

5-3-7-Valve's seismic performance evaluation

The stress from the inertia force caused by earthquake is measured in the weak part of the valve's main body and parts with high weight eccentric from pipe's axis. If the valve's strength is sufficient, the valve's performance would be safe.

1-In evaluation of valve's seismic performance, the stress caused by earthquake in the profile, between parts with weight eccentric, such as main body and valve's driver, should be smaller than the seismic design allowable stress.

2-For the following valves, the evaluation of the calculated stress could be ignored.

1-1-If the piping system with valves is designed using the allowable span method.

1-2-If parts with weight eccentric, such as drivers, are constrained.

1-3-If the natural frequency obtained from (3) is 20Hz or more.

3-Valve's natural frequency criterion

If the condition $\frac{H_{VD}}{\sqrt{D_V}} \leq 40$ is satisfied in the valve, the natural frequency is assumed 20Hz or more.

H_{VD}: the minimum distance between flanges center of mass and valve's driving part center of mass (mm)

D_V: the minimum width in parts with weight eccentricity, such as flat flange sides and valve driving parts (mm)

For manual valves disregarding the above cases is assumed 20Hz.

4-Modified horizontal earthquake force

The modified horizontal earthquake force imposed on the valve is obtained from equation 5.17.

$$H = \beta_8 \mu K_{MH} W_H \quad (5.17)$$

If the valve's rod direction is perpendicular to the earthquake vertical direction, the design force is obtained from 5.18 instead of 5.17.

$F_{MH} = \beta_9 K_{MV} W_H$ (5.18)

F_{MH}: modified horizontal seismic force of valve's design (N)

β_8: Magnification coefficient of valve piping horizontal response determined using table 5.1.

Table 5.1: The magnification coefficient of valve piping response

Coefficient	$\frac{H_{VD}}{\sqrt{D_V}}$
1 (though the evaluation could be ignored)	40 or smaller
$0.1\frac{H_{VD}}{\sqrt{D_V}}-3.0$	40-60
3.0	More than 60

B_9: valve's vertical response magnification coefficient. Its value ranges from 1 to 3, which is multiplied by 2 which in turn is the determined response magnification coefficient of piping based on the structure and valve support method.

μK_{MH} and K_{MV}: design's modified horizontal earthquake factor and design's modified vertical earthquake factor of piping support point

W_H: weight of parts with weight eccentricity, such as valve's driving parts (N)

5-Stress measurement

The maximum stress in the valve's driver main body is calculated using equation 5.19.

$$\sigma_n = \frac{F_{MH} . L_b}{Z} + \sigma_L \quad (5.19)$$

F_{MH}: valve's design modified horizontal earthquake force (N)

L_b: members' center of mass distance between weight parts, such as valves main bodies and drivers, and parts with weight eccentricity such as drivers (mm)

Z: elastic modulus of section (mm^3)

σ_L: created stress in section due to internal pressure and driving force (N.mm)

Since the stress creation mechanism varies with structures, thus valve's σ_L should be obtained using an appropriate procedure. Here is an example.

If the member's profile is cylindrical and the internal pressure is imposed to the valve's main body and the valve's leg is in the axial direction and the driver's outlet is along with the valve's leg direction, then the stress is obtained from equation 5.20.

$$\sigma_L = (F_p + F_m) . \frac{4}{\pi(D_o^2 - D_i^2)} \quad (5.20)$$

F_p: force due to internal pressure (N)

$$F_p = \frac{\pi D_i^2}{4} . P_p \quad (5.21)$$

F_m: exit force from driver part (based on valve's technical characteristics) (N)

D_o: external diameter of section (mm)

D_i: internal diameter of section (mm)
P_p: fluid's pressure on valve's main body (MPa)

In regular valves, such as manual valves, while the natural frequency is sufficiently high, the acceleration caused by earthquake would not highly increase. However, in a valve which its driving weight is high and its center of mass is far from the piping axis, a large driving force would be created in the driver part due to relative decreased natural frequency. For valves with natural frequencies lower than 20Hz, the stress is calculated in weak parts between the valve's main body and parts with weight eccentricity from the piping axis (from now on called "the weight eccentric parts of the driver part") for the inertia force caused by earthquake, and the seismic performance is evaluated. If the strength is sufficiently high, the flow shut-off performance is also assumed to be safe. The evaluation method procedure is presented in section 8 of the appendix.

5-3-8-Seismic performance evaluation of expansion connection

In evaluation of expansion connection's seismic performance, the maximum created axial stress range in accordions should be smaller than the acceptable stress range corresponding to 500 cycles of design.

1-Stress measurement

1-1-Axial displacement of accordion's corrugation

The accordions' displacement in piping support due to earthquake converts into corrugation axial displacement as follows:

$$e_{be} = e_x + e_y + e_\theta \tag{5.22}$$

a) For simple accordions

$$e_x = \frac{x}{N_b} \tag{5.23}$$

$$e_y = \frac{3 d_p y}{L_{lb} + x_c} \tag{5.24}$$

$$e_\theta = \frac{d_p \theta_A}{2N_b} \tag{5.25}$$

b) For double accordions

$$e_x = \frac{x}{2N_b} \tag{5.26}$$

$$e_y = \frac{K_{bl} d_p y}{2N_b \left(L_{1b} - C_{bl} + 0.5 x_c\right)} \tag{5.27}$$

e_{be}: displacement amount of accordion's corrugations (mm)
x: total axial displacements (mm)
y: amount of displacement perpendicular to axis (mm)
θ_A: total bending rotations around all axes (rad)
x_c: shrinkage side displacement at axial direction (mm)
N_b: number of accordion's corrugations of a part
d_p: average diameter of accordion (mm)

L_{Ib}: accordion part's effective length (mm)
C_{bI}: effective length of an accordion (mm)
K_{bI}: modification factor of double accordion equivalent displacement

1-2-Stress measurement

The stress measurement in U-shaped accordions is carried out as follows:

In case of non-U-shaped accordions, such as Gesc and Omega types, the stress should be calculated using an appropriate equation.

a) For accordions without reinforcement ring

i) Membrane stress in the axial direction caused by pressure

$$\sigma_{mmp} = \frac{P_p W_b}{2n_b t_p} \quad (5.28)$$

ii) Bending stress at axial direction caused by pressure

$$\sigma_{mbp} = \frac{P_p}{2n_b}\left(\frac{W_b}{t_p}\right)^2 C_p \quad (5.29)$$

iii) Membrane stress at axial direction caused by total displacement at each corrugation

$$\sigma_{mmd} = \frac{E'_b {t_2}^2}{2W^3 C_f} e_{ba} \quad (5.30)$$

iv) Bending stress at axial direction caused by total displacement at each corrugation

$$\sigma_{mbd} = \frac{5E'_b t_p}{3{W_b}^2 C_d} e_{ba} \quad (5.31)$$

b) For accordions with reinforcement ring

i) Axial membrane stress caused by pressure

$$\sigma_{mmp} = \frac{P_p(W_b - k_r q)}{2n_b t_p} \quad (5.32)$$

ii) Axial bending stress caused by pressure

$$\sigma_{mbp} = \frac{P_p}{2n_b}\left(\frac{W_b - k_r q}{t_p}\right)^2 C_p \quad (5.33)$$

iii) Axial membrane stress caused by total displacement in each corrugation

$$\sigma_{mmd} = \frac{E'_b t_p^2}{3(W_b - k_r q)^3 C_f} e_{ba} \quad (5.34)$$

iv) Axial bending stress caused by total displacement in each corrugation

$$\sigma_{mbd} = \frac{5E'_b t_p}{(W_b - k_r q)^2 C_d} e_{ba} \quad (5.35)$$

σ_{mmp}: axial membrane stress caused by pressure (N/mm^2)
σ_{mbp}: axial bending stress caused by pressure (N/mm^2)
σ_{mmd}: axial membrane stress caused by displacement in each corrugation (N/mm^2)
σ_{mbd}: axial bending stress caused by total displacement in each corrugation (N/mm^2)
P_p: driving pressure (MPa)

E'_p: modulus of elasticity at normal temperature of accordions' materials (N/mm^2)
W_b: height of accordion corrugation (mm)
q: pitch of accordion corrugation (mm)
n_b: number of accordion layers
t_p: calculation thickness of an accordion layer (mm)
k_r: modification factor of accordion with reinforcement ring
C_p: calculation bending stress modification factor caused by pressure
C_f: calculation membrane stress modification factor caused by accordion displacement
C_d: calculation bending stress modification factor caused by accordion displacement
e_{ba}: total displacement of each accordion membrane

2-Measurement of total stress domain

The maximum axial stress domain is calculated as follows.

$$S_{am} = 0.7(\sigma_{mmp} + \sigma_{mbp}) + (\sigma_{mmd} + \sigma_{mbd}) \tag{5.36}$$

S_{am}: maximum stress axial domains (N/mm^2)

It is necessary to install an appropriate type of expansion connection at an appropriate location in order to improve the seismic performance of piping system. The evaluation method procedure is shown in section 9 of the appendix.

5-3-9-Seismic performance evaluation of towers and tanks nozzle

The method for calculating the stress in nozzle of towers and tanks using a simple technique based on the thin shell theory is presented in the following.

1-Shell stress of thin wall

The stress at i direction is obtained using equation 5.37:

$$\sigma_i = K_N \frac{N_i}{t_W} \pm K_b \frac{6M_{ii}}{t_W^{\ 2}} \tag{5.37}$$

t_w: thickness of thin wall shell (mm)
N_i: membrane load at i direction per length unit (N/mm)
M_{ii}: bending moment at i direction per length unit (N.mm/mm)
K_N: stress concentration factor equal to 1 for membrane force
K_b: stress concentration factor equal to 1 for bending moment

2-Stress coefficient calculation

The stress coefficient is obtained using equation 5.38.

$$S_I = M_{ax}\left(\frac{\sigma_x + \sigma_\phi \pm \sqrt{(\sigma_x - \sigma_\phi)^2 + 4\tau^2}}{2} \times \sqrt{(\sigma_x - \sigma_\phi)^2 + 4\tau^2}\right) \tag{5.38}$$

S_I: stress coefficient
σ_x: radial stress
σ_φ: axial stress
τ: perimeter stress

The stress related to the design's horizontal seismic force, design's vertical force, and piping support displacement should be calculated in nozzle of towers and tanks. The details of the evaluation method are presented in section 10 of the appendix.

5-4-Ductile design method

When dealing with risk level-2, the seismic design should be performed using the ductile design method. The evaluation of ground displacements should also be carried out in addition to evaluation of ground's vibration effect with inertia force and response displacement.

The seismic design framework of ductile method is shown in section 12 of the appendix.

The piping system's seismic performance evaluation is done by confirming the safety against inertia force and approving the evaluation of response displacement and evaluation of ground movement. The piping system consists of pipe, support equipments, and pipe's support connected to both of them.

1-In expected seismic performance of the piping system, when the system is under ground movements, the high-pressure tank must remain impenetrable.

2-This design method is performed in case that the ground displacement effects, such as liquefaction are considered as well as effects related to inertia force and the relative displacement of support response,

3-In this design method, the ductility factor of materials should be smaller than the allowable ductility factor (in regard to being earthquake-resistant). Thus, in this design method, the behavior must be assumed nonlinear.

4-If the piping system is unaffected by the ground movement due to being located on a proper foundation, there would be no need for seismic performance evaluation of ground movement.

5-The flexibility in large deformations of the piping system depends on the pipe's curve. In this design, the performance in large deformations of curved pipes is very important. The evaluation of pipe's curvature is presented in section 13 of the appendix.

5-4-1-Evaluation of seismic design of inertia force and response displacement

The evaluation steps of inertia force and response displacement are shown in section 14 of the appendix.

5-4-1-1-Pipe's support structure response analysis

The design's lateral earthquake factor and pipe's support response displacement are calculated using the piping's support structure response analysis.

In response analysis, the support structure is replaced by an appropriate seismic system model, and the response acceleration and response displacement in support points are calculated using the modified pseudo-static method (appendix section 14.1), whether modal analysis or time-history response analysis.

5-4-1-2-Piping system's response analysis

The pipe's support acceleration and displacement are obtained through support structure's response analysis.

The analysis is then performed using an analytic model which considers the plastic deformation nonlinear behavior.

In order to analyze the acceleration and displacement responses, the piping system's analytic model is prepared based on the following rules.

1-The piping support structure's analytic model is prepared based on the performance evaluation steps of towers, tanks, and framed structures.

2-In the piping system's analytic model, straight pipes are considered as beam elements and curved pipes as curved beam elements.

3-In general, the analytic modeling of piping is performed between fixed points.

4-The calculation of piping's beam element's rigidity is carried out using dimensions in which the allowable corrosion amount is smaller than the nominal dimensions.

5-In order to calculate weight, the allowable corrosion amount is not considered and the nominal dimensions are used.

6-In order to calculate the piping's stress, the allowable corrosion amount is ignored.

The seismic performance against inertia force and displacement response is evaluated using the energy method, the equivalent linear analysis method, or the nonlinear response analysis method or combined methods. In such cases, the response analysis should be performed considering the following cases.

1-Piping element

In piping elements, such as straight pipes and T-shaped pipes, the beam elements are considered as linear. In curved pipes, the load-strain relation is considered as nonlinear. The linear behavior could be calculated using a proper flexibility factor.

2-Damping (attenuation) factor

The effect of energy absorption by plastic deformation of pipe's curvature, pipe's support, and piping supports is replaced by the damping factor which properly decreases.

Considering that the nonlinear analysis could be easily carried out, the response analytic model is consistent with the equivalent linear beam model considering the elastoplastic effect of pipe's curvature.

The general procedure of the equivalent linear analysis and the detailed analysis as well as the response magnification coefficient is described in section 14.2 of the appendix.

The seismic performance evaluation of piping system is obtained from combining the results of response analysis which acceleration is its input, and the response analysis which displacement of support is its input.

5-4-1-3-Damage mode

In piping system, the seismic performance of damage mode of items 1 to 8 is evaluated for inertia force and displacement response. The evaluation of inertia force and displacement response of piping system could be performed separately by evaluating the displacement caused by ground movement.

1-The plastic deformation of pipe's curvature

2-Cracks at junction area

3-Pipe cracks at piping support

4-Pipe's wave-like deformation

5-Damage in nozzle of seismic-designed equipments

6-Expansion connection damage

7-High-pressure gas leakage from flange connection

8-Damages in pipe support

5-4-1-4- Piping seismic evaluation

According to the earthquake direction and piping load combination, the cyclic stress range and the longitudinal stress caused by the earthquake force, pressure, and fluid's operation weight is calculated and designed so that the mentioned items would be smaller than the allowable final plastic deformation. In such cases, the damage modes of junction pipes and straight pipes are evaluated.

The allowable ductility factor caused by inertia force and displacement seismic response of design is obtained using table 5.2.

Table 5.2: The allowable final deformation against inertia force and ground displacement

Load	Allowable Final Deformation
Internal pressure, dead weight, and longitudinal stress caused by seismic inertia force	The equivalent deformation of area's internal pressure with the resistance of pipes' connection screw thread
In repeating stress (earthquake)	2% plastic strain of half domain in the hysteresis curve

In these cases the evaluation could be replaced by the following cases.

1-Pipe's curvature damage mode evaluation

The distortion angle of pipe's curvature should not exceed the allowable angle. Here, the allowable angle of pipe's curvature, θ, is equal to the distortion angle of pipe's curvature corresponding to the maximum plastic strain equivalent to 2% of half-domain.

2-Evaluation of junction pipes and straight pipes damage mode

First, the apparent stress is calculated which should be smaller than the seismic design allowable stress according to table 5.3.

Table 5.3: The simple seismic performance evaluation using the linear model for inertia force and displacement response

Load	Seismic Design Allowable Stress	Considerations
Longitudinal stress	2S	S: addressed in 4.1.3
Repeating stress range in earthquake	$4S_y$	S_y: yield strength at materials' design temperature or strength equivalent to 0.2% strain

The damage mode evaluation of curved pipe, junction pipes, and straight pipes as well as screw thread details are presented in section 14.3 of the appendix.

5-4-1-5-Seismic evaluation of flange connection

Around the flange connection, the leakage evaluation is performed at the direction of the axial force, and the bending moment is obtained by evaluation of acceleration response and displacement response of piping.

When the tensile force is imposed at the axial direction, F, and the bending moment, M, on the flange connection, then the leakage is evaluated using equation 5.39.

$$mP_p + \alpha_1 P_e \leq \sigma_a \tag{5.39}$$

m: gasket factor

P_P: internal pressure (MPa)

α_1: the modified leakage factor related to equivalent internal pressure which is assumed 0.75 of gasket factor

P_e: equivalent internal pressure caused by axial tensile force, F, and bending moment, M, under seismic effects (N/mm^2). For the overall gasket, the virtual internal pressure is considered at a location where the stress caused by this is equivalent to the created stress by F and M at pipe's edge, and for flat seat gasket, is considered at the gasket's edge.

σ_a: gasket load bearing (N/mm^2) caused by primary fastening of bolt. In cases where the bolts are not in an orderly fashion, the primary fastening stress of each bolt is the minimum amount between the bolt's yield stress and $1500/\sqrt{D}$. Where, D (mm) is the bolt's nominal diameter. The detailed study and the required load bearing of gasket in flange connections are presented in section 14.4 of the appendix.

5-4-1-6-Expansion connection seismic evaluation

The relative displacement of expansion connection both ends should be smaller than the relative displacement related to 50 times repetition in the connection.
In such cases, a separate evaluation similar to the ground displacement evaluation is carried out. In the direction which no relative displacement should occur, sufficient strength must exist for the reaction calculated from response calculations. The details for expansion connection stages are presented in section 5.14 of the appendix

5-4-1-7- Seismic evaluation of towers and tanks nozzle

The bending moment, torsion moment, and axial tension imposed on the nozzle should be smaller than the allowable values. The details for evaluation steps of towers and tanks nozzle are presented in section 14.6 of the appendix.

5-4-1-8- Seismic evaluation of pipe's support

The evaluation related to the inertia force and displacement response of pipe's support is carried out using the reaction obtained from response calculations related to the following damage modes at pipe's support.

1-Plastic deformation of pipe's support
2-Pipe's support crack
3-Displacement limit of pipe's support
4-Buckling limit of pipe's support

The seismic performance evaluation of pipe's support is shown in section 14.7 of the appendix.

5-4-2- Seismic design evaluation to ground movement

5-4-2-1- Piping system design caused by foundation displacement due to ground movement

For a piping system installed on a ground with liquefaction possibility, the effects of relative displacement of single foundations due to ground movement should be avoided using common (joint) foundation. If the impenetrability of high-pressure gas is confirmed in spite of sufficient flexibility of piping, there would be no need for this.

In cases in which displacement occurs in foundation (settlement and horizontal displacement due to liquefaction), the relative occurred displacement and the imposed load on pipe's support points become larger than different foundations' capacity. In fact, in spite of this excess load, the pipe must maintain its performance for impenetrability of high-pressure gas. Studies on the foundation displacement caused by ground movement continue dynamically, and currently it is hardly could be said that a single evaluation method has been proved. In order to maintain the design's reliability, it is a principle to design a structure in such a way that no additional displacements would be imposed on the piping by using common foundation instead of different foundations.

In cases where it is hard to carry out such measures, it is necessary for the piping to be flexible so that it could maintain the high-pressure gas impenetrability in relative displacement. The proper measurement of relative displacement of pipe's support points caused by ground movement could be done by referring to suggestions and results of the studies related to foundation displacement due to ground movement. The designing steps are presented after estimating the foundation displacement, in section 15 of the appendix, the method to provide flexibility of the piping system and seismic design steps are described.

5-4-2-2-Foundation displacement due to ground movement

The seismic performance evaluation of foundation against ground movement for maximum relative displacement between foundations dependant on the foundation's displacement is applied as follows.

- Foundation's uniform settlement caused by ground liquefaction and liquefied soil's flow
- Foundation's non-uniform settlement caused by ground liquefaction and liquefied soil's flow
- Lateral displacement of foundation caused by flow due to liquefaction

The displacement and relative displacement values between foundations caused by ground movement are described in section 16 of the appendix. However, the foundation's displacement caused by ground movement is obtained using the results of past earthquakes or numerical analysis technique.

5-4-2-3-Response analysis method

The seismic performance evaluation under ground movement is performed using the energy method or the equivalent linear analysis method or the nonlinear response analysis method, or a combination of them. In such cases, for pipe's curvature, the stress-strain relation is assumed as nonlinear and for other types of pipe could be linear. However, in pipe curvature, the linear response could be calculated using the modified proper flexibility factor in plastic deformation.

The flexibility factor of pipe's curvature in piping analysis for foundation displacement and analysis steps using the flexibility factor are presented in section 15 of the appendix.

5-4-2-4- Failure mode

As for the piping system, the earthquake performance evaluation of the failure mode from the following (i) to (viii) is performed for the inertial force and the response displacement. The evaluation of the inertial force and the response displacement of the piping system can be separately performed to the evaluation concerning the ground displacement by the movement of the ground.

1- Plastic deformation of curved pipe
2- Crack in bifurcation area
3- Crack of piping in piping support
4- Ratcheting
5- Failure of nozzle of seismic design facilities etc.
6- Failure of expansion joint
7- Leakage of high pressure gas from flange joint
8- Failure of pipe support

5-4-2-5- Seismic evaluation of pipelines

Calculating the fluid's pressure, operational weight, longitudinal stress caused by earthquake force, and repeating stress range caused by earthquake force and also considering the piping's load combination and earthquake's direction, the ductility factor should be inside the allowable ductility factor range.

In these cases, it is likely to use the evaluation related to the damage mode of pipe's curvature or junction pipes and straight pipes.

The allowable ductility factor of piping is addressed in equivalent plastic strain and it is assumed equal to 5% for ground displacement in design's seismic movement. In these cases, it is possible to use the evaluation of cases 1 and 2.

1-Evaluation of pipe's curvature damage mode

The method for seismic performance evaluation of pipe's curvature is that the pipe's curvature angle, θ_a, should not exceed the allowable angle at the location where the evaluation is being performed using the ductility factor. Here, the allowable angle, θ_a, of pipe's curvature is assumed equal to the pipe curve's curvature angle according to the maximum plastic strain equal to 5%.

2-Evaluation of damage mode of junction pipes, straight pipes, and other pipes

The method to evaluate the seismic performance of junction, straight, and other pipes is that the measured apparent stress should be equal or smaller than the allowable stress, $4S_y$, for seismic design at the location where the evaluation is carried out using the ductility factor. Details for pipe's curvature allowable angle are presented in section 18 of the appendix.

5-4-2-6-Seismic evaluation of flange connection

When the tensile force at the axial direction of F and bending moment of M is imposed on a flange connection, the leakage evaluation is performed as shown in section 5.1.

The seismic performance evaluation steps of flange connection are shown in section 7 of the appendix.

5-4-2-7-Expansion connection seismic evaluation

The relative displacement of expansion connection both ends should be smaller than the allowable relative displacement related to 10 repetitions of expansion connection. In such cases, the ground displacement evaluation could be performed separately from the evaluation of inertia force and displacement response. Additionally, in places where the relative displacement could not be expected for expansion connections, the expansion connection should have sufficient resistance for the reaction obtained from the response calculations.

Details of estimation steps are presented in section 20 of the appendix.

5-4-2-8-Seismic evaluation of towers and tanks nozzle

The bending moment, torsion moment, and axial tension imposed on the nozzle must be smaller than the allowable values.

Details for evaluation steps related ground displacement of towers and tanks nozzle are presented in section 21 of the appendix.

5-4-2-9-Pipe's support seismic evaluation

The ground displacement (ground movement) of pipe's support is evaluated for pipe's support reaction obtained from calculation of response related to (a) to (d) damage modes.

In such cases, the evaluation could be carried out separately for the evaluation related to the inertia force and response displacement.

(a) Plastic deformation of pipe's support
(b) Displacement limit of pipe's support
(c) Release load of free support
(d) Buckling limit of pipe's support

The seismic performance evaluation of support is presented in section 22 of the appendix.

Chapter 6

Seismic design and Safety Control of Horizontal Vessel

6-1-Steps of seismic design of horizontal vessel

When risk level-1 earthquake is in use, the allowable stress method could be applied, and when risk level-2 earthquake is in use, the ductility method is applied.
The target facilities of seismic design are horizontal vessels which their capacity is more than 3 tons in regard to mass, or 300m^3 in regard to volume.

The weight criterion is used for liquid gas tanks, and the volume criterion is applied in case of gas vapor such as under pressure.

6-2-Allowable stress design method

6-2-1-Response analysis method

Based on size and natural period of horizontal vessels, the seismic performance is evaluated using pseudo-static method, modified pseudo-static method, or modal analysis methods.

6-2-1-1-Pseudo-static method

The pseudo-static method could be applied to tanks with medium and low importance factor and storage weight smaller than 100 tons.

$$K_{SH}=\beta_4 K_H \qquad (6.1)$$

K_{SH}: Design's horizontal earthquake factor
K_H: Horizontal seismic intensity at surface
The horizontal earthquake force is calculated using equation 6.2:

$$F_{SH}=K_{SH}W_H \qquad (6.2)$$

F_{SH}: Design's static horizontal earthquake force (N)
W_H: Sum of tank's weight and fluid's weight

End

Figure 6.1: Designing procedure of allowable stress method

6-2-1-2-Modified pseudo-static method

1-Natural period of horizontal vessel

$$T = 2\pi\sqrt{\frac{W_0}{Kg}} \tag{6.3}$$

T: natural period (s)

g: gravity acceleration (mm/s^2)

W_0: operational weight (N), sum of tank's weight and fluid's weight

K: horizontal rigidity of horizontal vessel obtained from equation 6.4 (N/mm)

$$K = \frac{1}{\frac{\lambda}{K_1} + \frac{1}{K_2}} \tag{6.4}$$

K_1, K_2: values obtained from equations 6.5, 6.6, and 6.7 (N/mm)

$$K_1 = \frac{3n_l E A_l D_l^2}{2H_l^3} \tag{6.5}$$

$$K_2 = \frac{n_l K_C}{1 + \frac{H_l K_C}{E_G A_l}} \tag{6.6}$$

$$K_c = \frac{4E(I_1 + I_2)}{H_l^3} \tag{6.7}$$

H_l and n_l: length (mm) and number of legs
E: longitudinal modulus of elasticity of leg materials (N/mm^2)
E_G: transverse modulus of elasticity of leg materials (N/mm^2)
A_l: leg's cross section area (mm^2)
D_l: created circle's diameter by legs' center (mm)
I_1: inertia moment of leg related to the circumferential direction (longitudinal section inertia moment (rectangular)) (mm^4)
I_2: inertia moment of leg in radial direction (transverse section inertia moment (circle)) (mm^4)
λ: value calculated from equation 6.8

$$\lambda = \left(\frac{H_2}{H_1}\right)^2 - \frac{H_2}{H_1} + 4 \tag{6.8}$$

H_2: height of center of mass from leg's plate (mm)

2-Design's modified horizontal force

$$F_{MH} = K_{MH} W_H \tag{6.9}$$

$$F_{MV} = K_{MV} W_v \tag{6.10}$$

F_{MH}: Design's modified horizontal earthquake force (N)
F_{MV}: Design's modified vertical earthquake force (N)
W_V: sum of weight of structure and weight of fluid imposed on the point on which the designing is performed

6-2-2-Stress measurement

The stress of horizontal vessels (limited to two support points) is obtained from the following equations. When shear plates (hardeners) are installed, there would be no need for calculating the shear stress of anchor bolts.

1-Shell stress at connection with seat

1-1-Tensile stress

$$\sigma_t = \frac{P_o D_m}{4t} + \frac{M_{LS}}{Z_s} \tag{6.11}$$

σ_t: shell's tensile stress at connection with seat (N/mm^2)

t: shell thickness (except for the allowable corrosion amount (mm))

P_0: design pressure (N/cm^2)

D_m: shell's mean diameter (mm)

M_{LS}: shell's bending moment at connection with seat calculated from equation 6.12 (N.mm)

$$M_{LS} = Q\left\{a - \frac{6a(L-a) + 3\left(R_m{}^2 - H_{00}{}^2\right)}{2(3L + 4H_{00})}\right\} \tag{6.12}$$

Q: reaction force of shell from seat, calculated from equation 6.13 (N)

$$Q = \frac{W_V + F_V}{2} + F_{ev} \tag{6.13}$$

W_V: operational weight (N)

F_V: vertical earthquake force of design (N)

F_{ev}: maximum calculated value from equations 6.14 and 6.15 (N)

$$F_{VX} = \frac{F_H H_V}{L_S} \tag{6.14}$$

$$F_{VY} = \frac{3F_H H_V}{4b} \tag{6.15}$$

F_{VX}: equivalent vertical load to design's horizontal earthquake force imposed at axial direction (N)

F_{VY}: equivalent vertical load to design's horizontal earthquake force imposed at direction perpendicular to axis

L_S: thread shown in figure 6.2 (mm)

F_H: design's horizontal earthquake force (N)

H_V: tank's height between shell's center and leg's plate shown in figure 6.2 (mm)

b: seat width shown in figure 6.2 (mm)

a: distance between seat center and shell's tangential line shown in figure 6.2 (mm)

L: shell's length at axial direction shown in figure 6.2 (mm)

H_{00}: head height shown in figure6.2 (mm)

R_m: mean radius shown in figure 6.2 (mm)

Z_S: shell's section modulus at connection with the seat obtained from equation 16.6 (mm^3)

a) When head is reinforced using head (limited to cases in which $a/R_m \leq 0.5$) or hardening ring:

$$Z_S = \pi R_m{}^2 t \tag{6.16}$$

b) Other cases:

$$Z_S = G R_m{}^2 t \tag{6.17}$$

G: value obtained from support angle, θ, shown in figures 6.2 and 6.3.

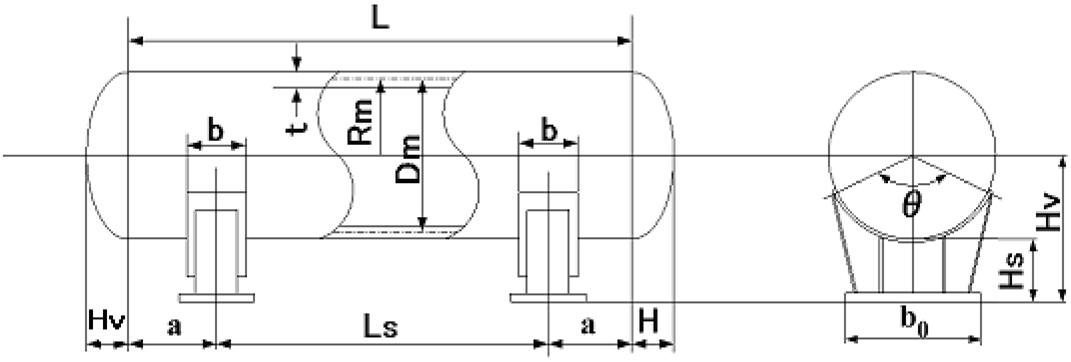


Figure 6.2: Assumed configuration of horizontal vessel

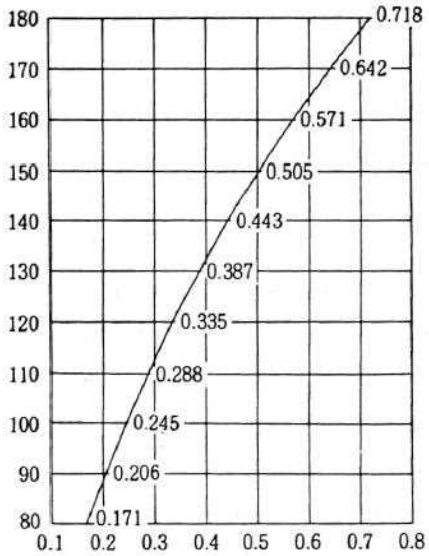


Figure 6.3: Support angle and G value

1-2-Compressive stress

$$\sigma_C = \frac{K_1 Q}{(b + 1.56\sqrt{R_m t})t} \tag{6.18}$$

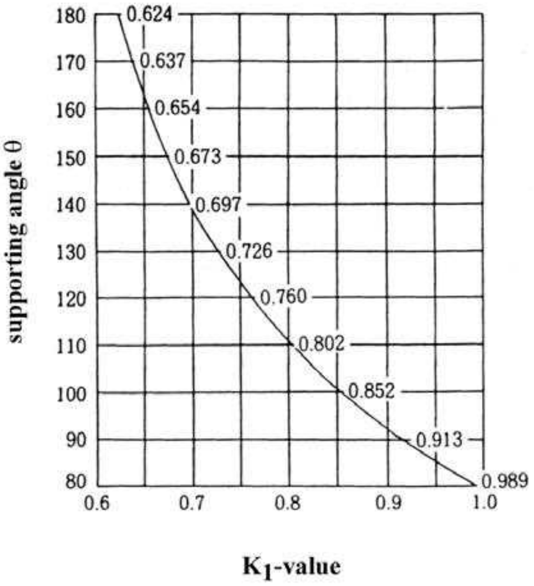
σ_c: shell compressive stress at connection with seat (N/mm^2)

b: seat's width shown in figure 6.4 (mm)

t: as described in the previous equations, but if the reinforcement plate width is larger than the calculated amount from 6.19, then the total thickness of shell and reinforcement plate could be used.

$$b + 1.56\sqrt{R_m t} \tag{6.19}$$

K_1: a value which is obtained using figure 6.4 and based on seat support angle, θ.



value

2-Shell center axial stress

2-1-Tensile stress

$$\sigma_t = \frac{P_0 D_m}{4t} + \frac{M_{LC}}{Z_c} \tag{6.20}$$

σ_t: shell center tensile stress (N/mm^2)

M_{LC}: bending moment of shell center calculated from equation 6.21 (N.mm)

$$M_{LC} = Q\left\{\frac{3L^2 + 6\left(R_m{}^2 - H^2\right)}{4(3L + 4H)} - a\right\} \tag{6.21}$$

Z_C: section modulus at shell center calculated from equation 6.22 (mm^3)

$$Z_c = \pi R_m{}^2 t \tag{6.22}$$

2-2-Compressive stress

$$\sigma_{bs} = \frac{M_{LC}}{Z_c} \tag{6.23}$$

σ_{bs}: compressive stress at shell's center (N/mm^2)

Head stress (cases in which the shell is reinforced by head)

$$\sigma_t = \frac{K_2 Q}{R_m t_h} + \sigma' \tag{6.24}$$

σ_t: tensile stress of head (N/mm^2)

t_h: head thickness (except for the allowable corrosion amount (mm))

K_2: value obtained from support angle, θ, in figure 6.5

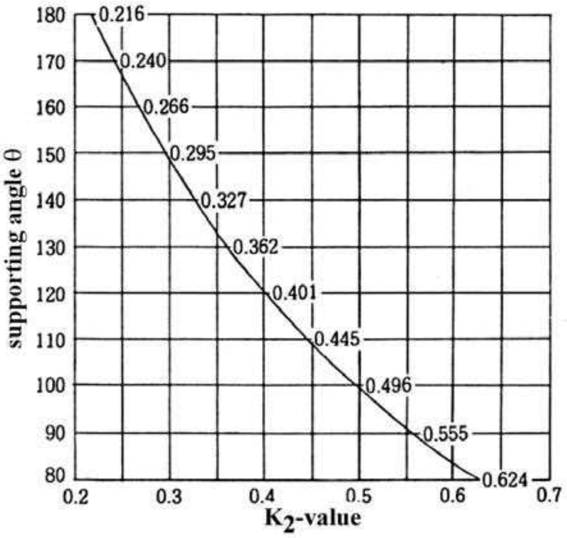


Figure 6.5: Support angle and K_2 value

σ': head tensile stress due to internal pressure, calculated from equations shown in table 6.1 in regard to geometric shape (N/mm^2)

Table 6.1: Equations for calculating σ'

Head Geometric Shape	σ'
Semi-ellipsoid head	$\frac{P_0 D_h}{2t_h} \cdot \frac{1}{6}\left\{2+\left(\frac{D_h}{2h_{ma}}\right)^2\right\}$
Semi-spherical head	$\frac{P_0 D_h}{4t_h}$
Concave head	$\frac{P_0 D_m}{4t_h} \cdot \frac{1}{4}\left\{3+\sqrt{\frac{R_m}{r_k}}\right\}$

D_h: larger internal diameter of semi-ellipsoid head, internal diameter of semi-spherical head, and crown radius of concave head (except for the allowable corrosion amount (mm))

h_{ma}: half of the smaller internal axis (except for the allowable corrosion amount (mm))

r_k: internal radius of bracing belt (mm)

3-Seat stress

3-1-In cases where one side of the seat is fixed

$$\sigma_c = \frac{W_V + F_V}{2A_{SD}} + \frac{\{2F_H - 0.1(W_v + F_v)\}H_S}{2Z_{SD}} + \frac{F_H H_V}{A_{SD} L_S} \quad (6.25)$$

3-2-In cases where both sides of the seat are fixed

$$\sigma_c = \frac{W_v + F_v}{2A_{SD}} + \frac{F_H H_S}{2Z_{SD}} + \frac{F_H H_V}{A_{SD} L_S} \quad (6.26)$$

σ_c: seat compressive stress (N/mm^2)

A_{SD}: seat's effective cross section (mm^2)

Z_{SD}: seat's effective section modulus (mm^3)

H_S: height from base plate to lower surface of seat (mm)

4-Stress of anchor-bolt

4-1-Tensile stress

$$\sigma_t = \frac{F_H H_V}{n_{abs} A_b C_b} - \frac{W_v - F_V}{2n_{abs} A_b} \tag{6.27}$$

σ_t: tensile stress of anchor-bolt (N/mm^2)

A_b: effective cross section of anchor-bolt (mm^2)

C_b: distance between anchor-bolts tangent to the tank's axis (mm)

n_{abs}: number of anchor-bolts of each seat

4-2-Shear stress

a)In cases where one side of the seat is fixed:

$$\tau = \frac{F_H - 0.2(W_V - F_V)}{n_{abf} A_b} \tag{6.28}$$

b)In cases where both sides of the seat are fixed:

$$\tau = \frac{F_H - 0.3(W_V - F_V)}{2n_{abf} A_b} \tag{6.29}$$

τ: shear stress of anchor-bolt for fixed seat (N/mm^2)

n_{abf}: number of anchor-bolts at each fixed side

5-Shear plate stress

5-1-Bending stress

$$\sigma_b = \frac{3h_{sa}^{\ 2}\sigma_{CP}}{t_{sa}^{\ 2}} \tag{6.30}$$

σ_b: bending stress of shear plate (N/mm^2)

t_{sa}: shear plate thickness (mm)

h_{sa}: shear plate height (mm)

σ_{CP}: compressive stress of shear plate due to concrete's compressive force shown in table 6.2 in the direction of earthquake force (N/mm^2)

Table 6.1: σ_{CP} equations

Earthquake Force Direction	σ_{CP}
Shell axis	$\frac{F_H}{b_y h_{sa}}$
Perpendicular to shell axis	$\frac{F_H}{b_x h_{sa}}$

b_y: length of shear plate in the direction of shell axis (mm)

b_x: length of shear plate in the direction perpendicular to shell axis (mm)

5-2-shear stress

its value is obtained using equations presented in table 6.3 and considering the earthquake force direction.

Table 6.3: Earthquake force direction and shear stress

Earthquake Force Direction	τ
In the direction of shell axis	$\frac{F_H}{b_y t_{sa}}$
P□rpendicular to shell axis	$\frac{F_H}{b_x t_{sa}}$

τ: shear stress of shear plate (N/mm^2)

Locations in which the horizontal vessel stress is calculated using two support points are presented in table 6.4.

Table 6.4: Determined points

Part which its stress is being determined	Type of Stress				
	Tension	Shear	Bending	Compressive	Buckling
Stress of shell connected to seat	○			○	○
Shell center axial stress	○			○	○
Head stress A/R_m<0.5	○				
Seat				○	○
Anchor-bol	○	○ One of these two			
Shear plate		○	○		

6-2-3-Allowable stress

The allowable stress is determined for resistant members against pressure and support members.

6.2.4 Acceptance criteria

All of the calculated stresses should be smaller than the allowable stresses.

6-3-Ductile Design

6-3-1-Damage mode

The seismic evaluation by the ductility design should be performed for the following modes.

1-Shell failure

-Yielding or buckling caused by compressive stress, tensile yield at the location of connection with seat

-Yielding or buckling caused by compressive stress, tensile yield at shell center

-Head tensile yielding

2-Lower support failure

-Compressive yield

3-Anchor-bolt failure

-Tensile yielding

-Shear yielding
-Tensile and shear stress combination
4-Shear plate failure
-Bending yielding
-Shear yielding
-Bending and shear stress combination

6-3-2-Yield earthquake coefficient

The yield earthquake coefficient should be measured for each damage mode.

1-Yield earthquake coefficient related to shell's damage mode

1-1- Yield earthquake coefficient related to tensile yield at connection with seat

$$K_{ytS} = K_{MH} \frac{S_y - (\sigma_{ps} + \sigma_{tSO})}{\sigma_{tSE}} \tag{6.31}$$

K_{ytS}: yield earthquake coefficient related to tensile yield at connection with seat

K_{MH}: modified horizontal earthquake coefficient of design for location of connection with seat

S_y: yield stress or equivalent stress to 0.2% strain from steel test of shell plate at designing temperature (N/mm^2)

σ_{ps}: tensile stress of shell plate caused by internal pressure (N/mm^2)

$$\sigma_{ps} = \frac{P_o D_m}{4t} \tag{6.32}$$

t: shell plate thickness (except for the allowable corrosion amount, (mm))

P_0: design pressure (MPa)

σ_{tSO}: tensile stress of shell plate caused by operation's normal load (N/mm^2)

$$\sigma_{tSO} = \frac{C_{LS} W_V}{2Z_S} \tag{6.33}$$

$$C_{LS} = a - \frac{6a(L-a) + 3(R_m{}^2 - H_{00}{}^2)}{2(3L + 4H_{00})} \tag{6.34}$$

a: distance between seat center and tangential line of shell shown in the following figure (mm)

Z_S: shell section modulus at the location of connection with seat obtained from the following equations (mm^3)

a)When shell is reinforced by head (limited to cases in which $a/R_m \leq 0.5$) or hardening ring:

$$Z_S = \pi R_m{}^2 t \tag{6.35}$$

b) Other cases:

$$Z_S = G R_m{}^2 t \tag{6.36}$$

G: value of the factor obtained from support angle, θ, shown in figure 6.3

σ_{tSE}: tensile stress of shell plate caused by seismic load

$$\sigma_{tSE} = \frac{C_{LS}}{Z_s}\left(\frac{F_v}{2} + F_{ev}\right) \tag{6.37}$$

$$F_V = K_{MV} W_V \tag{6.38}$$

K_{MV}: modified vertical seismic factor of design

W_V: operational weight (N)

$$F_{ev} = \max\left(F_{HX}\frac{H_V}{L_S}, F_{HY}\frac{3H_V}{4b}\right) \quad (6.39)$$

F_{HX}: horizontal earthquake force of design at axial direction (N)

F_{HY}: horizontal earthquake force of design perpendicular to axis (N)

H_V: tank height between shell center and base plate (mm)

b: seat width shown in figure 6.2 (mm)

L_S: seat thread shown in figure 6.2 (mm)

1-2-Yield earthquake coefficients related to buckling or yield at connection with seat

$$K_{ycS} = K_{MH}\frac{S_c - \sigma_{cSO}}{\sigma_{cSE}} \quad (6.40)$$

K_{ycS}: yield earthquake coefficient related to compressive stress

$$S_c = \min(S_y, S_f S') \quad (6.41)$$

S_y: yield strength or equivalent stress to 0.2% strain from steel test of shell plate at design temperature (N/mm^2)

S_f: 1.5

$$S' = \frac{0.6Et}{\left(1 + 0.004\frac{E}{S_y}\right)D_m} \quad (6.42)$$

E: longitudinal modulus of elasticity (N/mm^2)

σ_{cSO}: compressive stress of shell plate caused by operational normal load (N/mm^2):

$$\sigma_{cSO} = \frac{W_v}{2C_{K1}} \quad (6.43)$$

$$C_{K1} = \frac{\left(b + 1.56\sqrt{R_m t}\right)t}{K_1} \quad (6.44)$$

t: described in previous equations. But if the reinforcement plate's width is larger than the value obtained from equation 6.45, then the total sum of shell and reinforcement plate could be used.

$$b + 1.56\sqrt{R_m t} \quad (6.45)$$

b: seat width (mm)

K_1: value obtained using figure 6.4 based on seat support angle, θ.

σ_{cSE}: compressive stress of shell plate caused by earthquake load (N/mm^2)

$$\sigma_{cSE} = \frac{1}{C_{K1}}\left(\frac{F_V}{2} + F_{ev}\right) \quad (6.46)$$

For the above equations, F_{ev}, R_m, D_m, W_V, F_V, and K_{MH}: similar to definition in previous equations

1-3-Equivalent yield earthquake coefficient to tensile yield at shell center

$$K_{ytc} = K_{MH}\frac{S_y - (\sigma_{ps} + \sigma_{tCO})}{\sigma_{tCE}} \quad (6.47)$$

σ_{tCE}: tensile stress of shell plate caused by earthquake load (N/mm^2)

$$\sigma_{tCE} = \frac{C_{LC}}{Z_C}\left(\frac{F_V}{2} + F_{ev}\right) \tag{6.48}$$

$$C_{LC} = \frac{3L^2 + 6\left(R_m{}^2 - H^2\right)}{4(3L + 4H)} - a \tag{6.49}$$

1-4-Yield earthquake coefficient related to buckling or yield at shell center

$$K_{ycC} = K_{MH}\frac{S_c - \sigma_{tCO}}{\sigma_{tCE}} \tag{6.50}$$

1-5-Yield earthquake coefficient related to head tensile yield (limited to cases in which the head is reinforced by head)

$$K_{ytK} = K_{MH}\frac{S_y - \left(\sigma_{tKO} + \sigma_{ph}\right)}{\sigma tKE} \tag{6.51}$$

S_y: yield strength or equivalent stress to 0.2% strain from steel test of shell plate at design temperature (N/mm^2)

σ_{tKO}: tensile stress of head caused by operational normal load (N/mm^2)

$$\sigma_{tKO} = \frac{W_v}{2C_{K2}} \tag{6.52}$$

t_n: plate thickness (except for the allowable corrosion amount, mm)

K_2: value obtained from support angle, θ, and figure 6.5

$$\sigma_{tKE} = \frac{1}{C_{K2}}\left(\frac{F_V}{2} + F_{ev}\right) \tag{6.53}$$

C_{K2}: determined similar to C_{K1} and using K2

σ_{tKE}: tensile stress of head caused by internal pressure (N/mm^2)

2-Yield earthquake coefficient related to seat damage

2-1- Cases with only one fixed side

$$K_{ycD} = K_{MH}\frac{F - \left(\sigma_{cDO} - \sigma_{bDO}\right)}{\sigma_{cDE} + \sigma_{bDE}} \tag{6.54}$$

K_{ycD}: yield stress coefficient related to seat compressive yield

F: yield strength or equivalent strength to 0.2% strain from steel test of seat materials (N/mm^2)

σ_{cDO}: compressive stress of seat caused by operational normal load (N/mm^2)

$$\sigma_{cDO} = \frac{W_V}{2A_{SD}} \tag{6.55}$$

A_{SD}: effective section area of seat (mm^2)

σ_{bDO}: bending stress of seat caused by operational normal load (N/mm^2)

$$\sigma_{bDO} = \frac{0.1W_V H_S}{2Z_{SD}} \tag{6.56}$$

Z_{SD}: effective section modulus of seat (mm^3)

H_S: height from base plate to lower surface of seat (mm)

σ_{cDE}: compressive stress of seat caused by earthquake load (N/mm^2)

$$\sigma_{cDE}=\frac{F_V+2F_H\left(\frac{H_V}{L_S}\right)}{2A_{SD}} \tag{6.57}$$

σ_{bDE}: bending stress of seat caused by earthquake load (N/mm^2)

$$\sigma_{bDE}=\frac{(2F_H-0.1F_V)H_S}{2Z_{SD}} \tag{6.58}$$

2-2-Cases where both sides are fixed

$$K_{YCD}=K_{MH}\,\frac{F-\sigma_{cDO}}{\sigma_{cDE}+\sigma_{bDE}} \tag{6.59}$$

K_{YCD}: yield earthquake coefficient related to compressive yield of seat

σ_{bDE}: bending stress of seat caused by earthquake load (N/mm^2)

$$\sigma_{bDE}=\frac{F_H H_S}{2Z_{SD}} \tag{6.60}$$

3-Yield earthquake coefficient related to anchor-bolt

3-1-Yield earthquake coefficient related to tensile yield of anchor-bolt

$$K_{ytB}=K_{MH}\,\frac{F+\sigma_{tBO}}{\sigma_{tBE}} \tag{6.61}$$

K_{ytB}: yield earthquake coefficient related to tensile yield of anchor-bolt

F: yield strength or equivalent strength to 0.2% strain from steel test of anchor-bolt materials (N/mm^2)

σ_{tBO}: tensile stress of anchor-bolt caused by earthquake load operational normal load (N/mm^2)

nabs: number of bolts in each seat

A_b: effective cross-section area of anchor-bolt (mm^2)

σ_{tBE}: tensile stress of anchor-bolt caused by earthquake load (N/mm^2)

$$\sigma_{tBE}=\frac{1}{nA_b}\left(\frac{F_H H_V}{C_b}+\frac{F_V}{2}\right) \tag{6.63}$$

C_b: distance between anchor-bolts in the direction perpendicular to tank's axis (mm)

3-2-Yield earthquake coefficient equivalent to shear yield of anchor-bolt

a) Cases where one side is fixed

$$K_{ysB}=K_{MH}\,\frac{\frac{F}{\sqrt{3}}+0.4\sigma_{tBO}}{\tau_{BE}} \tag{6.64}$$

K_{ysB}: yield earthquake coefficient equivalent to shear yield of anchor-bolt

τ_{BE}: shear stress of anchor-bolt caused by earthquake load (N/mm^2)

$$\tau_{BE}=\frac{F+0.2F_V}{nA_b} \tag{6.65}$$

b) Case where both sides are fixed

$$K_{ysB}=K_{MH}\,\frac{\frac{F}{\sqrt{3}}+0.3\sigma_{tBO}}{\tau_{BE}} \tag{6.66}$$

K_{ysB}: yield earthquake coefficient equivalent to shear yield of anchor-bolt

$$\tau_{BE} = \frac{F_H + 0.3F_V}{2nA_b} \quad (6.67)$$

3-3-Yield earthquake coefficient equivalent to stress combination of anchor-bolt

a) Cases where one side is fixed

$$K_{ymB} = K_{MH} \frac{1.4F + 1.64\sigma_{tBO}}{\sigma_{tBE} + 1.6\tau_{BE}} \quad (6.68)$$

K_{ymB}: yield earthquake coefficient equivalent to stress combination of anchor-bolt

b) Cases where both sides are fixed

$$K_{ymB} = K_{MH} \frac{1.4F + 1.64\sigma_{tBO}}{\sigma_{tBE} + 1.6\tau_{BE}} \quad (6.69)$$

K_{ymB}: yield earthquake coefficient equivalent to stress combination of anchor-bolt

4-Yield earthquake coefficient related to damage of shear plate

4-1-Yield earthquake coefficient equivalent to bending stress of shear plate

$$K_{yb} = K_{MH} \frac{F}{\sigma_{bE}} \quad (6.70)$$

K_{yb}: yield earthquake coefficient related to bending stress of shear plate

F: yield strength of equivalent strength to 0.2% strain from steel test of shear plate materials (N/mm^2)

σ_{bE}: bending stress of shear plate caused y earthquake load (N/mm^2)

$$\sigma_{bE} = \frac{3h_{sa}{}^2 C_{cp1}}{t_{SP}{}^2} F_H \quad (6.71)$$

t_{SP}: thickness of shear plate (mm)

h_{sa}: height of shear plate

$$C_{cp1} = \max\left(\frac{1}{b_Y h}, \frac{1}{b_X h}\right) \quad (6.72)$$

b_Y: length of shear plate perpendicular to shell axis (mm)

b_X: length of shear plate in the direction of shell axis (mm)

4-2-Yield earthquake coefficient related to shear yield of shear plate

$$K_{ys} = K_{MH} \frac{F}{\sqrt{3}\tau_E} \quad (6.73)$$

K_{ys}: yield earthquake coefficient related to shear yield of shear plate

τ_E: shear stress of shear plate caused by earthquake load (N/mm2)

$$\tau_E = C_{cp2} F_H \quad (6.74)$$

$$C_{cp2} = \max\left(\frac{1}{b_Y t_{SP}}, \frac{1}{b_X t_{SP}}\right) \quad (6.75)$$

4-3- Yield earthquake strength related to bending and shear stresses combination of shear plate

$$K_{yms} = K_{MH} \frac{F}{\sqrt{\sigma_{bE}{}^2 + 3\tau_E{}^2}} \quad (6.76)$$

K_{yms}: yield earthquake strength related to bending and shear stresses combination of shear plate

6–3–3–Ductility coefficient

The ductility coefficient is obtained from equation 6.77 using the modified horizontal earthquake factor of design and yield earthquake coefficient for each damage mode of horizontal vessels.

$$\mu_p = \frac{1}{4C}\left\{\left(\frac{K_{MH}}{K_y}\right)^2 - 1\right\} \qquad (6.77)$$

μ_p: ductility coefficient for each damage mode, if $K_{MH} \leq K_y$ thus $\mu_p = 0$

K_{MH}: modified horizontal earthquake coefficient of design

K_y: yield earthquake coefficient for each damage mode

C: determined values as follows based on damage mode characteristics

1-Shell failure:

-Yielding or buckling caused by compressive stress, tensile yield, at connection location with seat, yielding or buckling caused by compressive stress, tensile yield at shell center and tensile yield of head C = 2.0

2-Saddle support failure:

-Compressive yield C = 2.0

3-Anchor-bolt failure:

-Tensile yield and combination of tensile and shear stresses C = 1.0

-Shear stress C = 2.0

4-Shear plate failure:

-Bending yield, shear yield, and combination of bending and shear stresses C = 2.0

6-3-4- Allowable ductility coefficient

The allowable ductility coefficient is calculated for each damage mode of horizontal vessel.

1-Shell damage:

-Yielding or buckling caused by compressive stress, tensile yield at connection location with seat, and yielding and buckling caused by compressive stress, tensile yield at shell center $\mu_{pa} = 0.35$

-Head tensile yield $\mu_{pa} = 1.0$

2-Saddle support failure:

-Compressive yield $\mu_{pa} = 1.0$

3-Anchor-bolt yield:

-Tensile yield and combination of tensile and shear stresses $\mu_{pa} = 1.8$

-Shear yield $\mu_{pa} = 0.35$

4-Shear plate failure:

-Bending yield, shear yield, and combination of bending and shear stresses $\mu_{pa} = 0.35$

6-3-5- Acceptance criteria

Equation 6.78 must be satisfied for each damage mode of horizontal vessels.

$$\mu_p \leq \mu_{pa} \qquad (6.78)$$

μ_p: ductility coefficient of each damage mode

μ_{pa}: allowable ductility coefficient of each damage mode

Chapter 7

Seismic Design and Safety Control of Spherical Tanks

7-1-Steps for seismic design of spherical tank

When earthquake risk level-1 is in use, the allowable stress, and whenever earthquake risk level-2 is in use, the ductility design method could be applied.
The target facilities of seismic design are spherical tanks with capacity more than 3 tons in regard to mass or $300m^3$ in regard to volume.

For liquid gas tanks, the weight criterion, and for vapor gases such as pressurized gas, the volume criterion could be used.

7-2-Allowable stress design method

7-2-1-Analysis methods

Based on size and natural period of spherical tanks, the seismic performance of these tanks is evaluated using pseudo-static method, modified pseudo-static method, or modal analysis methods.

7-2-1-1-Pseudo-static method

The pseudo-static method could be applied to structures with medium and low importance factor and storage weight smaller than 100 tons.

$$K_{SH} = \beta_4 K_H \tag{7.1}$$

K_{SH}: static horizontal seismic factor of design
K_H: horizontal seismic factor at surface
β_4: magnification factor of horizontal response

$$F_{SH} = K_{SH} W_H \tag{7.2}$$

F_{SH}: static horizontal seismic force of design (N)
W_H: sum of tank weight and fluid's effective weight
The effective weight could be obtained by multiplying fluid weight to the effective weight ratio shown in figure 7.1.

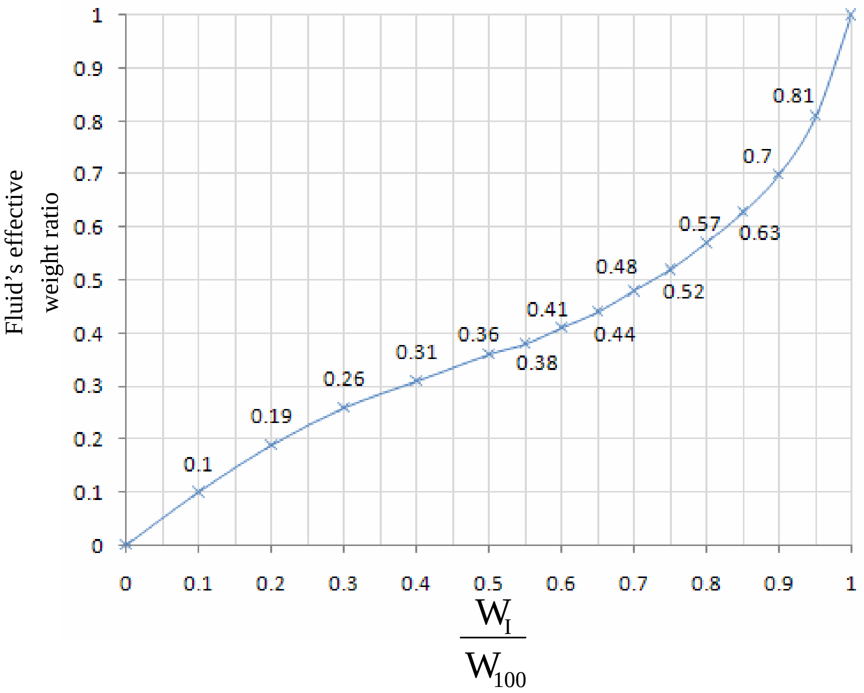


Figure 7.1: Fluid's effective weight ratio

W_1: fluid weight

W_{100}: fluid weight when tank is full

7-2-1-2-Modified pseudo-static method

1-Natural period of spherical tank

$$T = 2\pi\sqrt{\frac{W_0}{Kg}} \qquad (7.3)$$

T: natural period (sec)

K: horizontal rigidity of spherical tank obtained from equation 7.4 (N/mm)

g: gravity acceleration (mm/s^2)

W0: operational weight (N), sum of tank weight and fluid's effective weight

2-Horizontal rigidity of spherical tank

$$K = \frac{1}{\frac{1}{K_1} + \frac{1}{K_2}} \qquad (7.4)$$

K: horizontal rigidity (N/mm)

K_1: cyclic rigidity of entire body

$$K_1 = \frac{3n_s E A_{CL} D_B{}^2}{8H_C{}^3} \qquad (7.5)$$

K_2: shear rigidity of entire body

$$K_2 = n_s K_C \left(\frac{2C}{C_2 + \frac{4LK_C}{EA}} + 1 \right) \quad (7.6)$$

$$K_C = \frac{3EI_C}{H_1^3} \quad (7.7)$$

$$A = \frac{1}{\frac{C_3}{A_B \cos^3 \theta_e} + \frac{C_4 \tan^3 \theta_e}{A_{CL}}} \quad (7.8)$$

$$C_1 = \frac{1}{4} \lambda_C{}^2 (3 - \lambda_C{}^2)^2 \quad (7.9)$$

$$C_2 = \lambda_C{}^2 (1 - \lambda_C)^3 (3 + \lambda_C) \quad (7.10)$$

H_C: height from underneath surface of base plate to center of spherical body (mm)
n_s: number of supports
E: longitudinal modulus of elasticity of lower support materials
A_{CL}: section area of base on ground which is equal cross-section area of a single column on foundation (mm^2)
D_B: diameter of circle created by support's center (mm)
L: distance between adjacent supports (mm)
H_l: effective height obtained from equation 7.11

$$H_1 = H_C - L_W \quad (7.11)$$

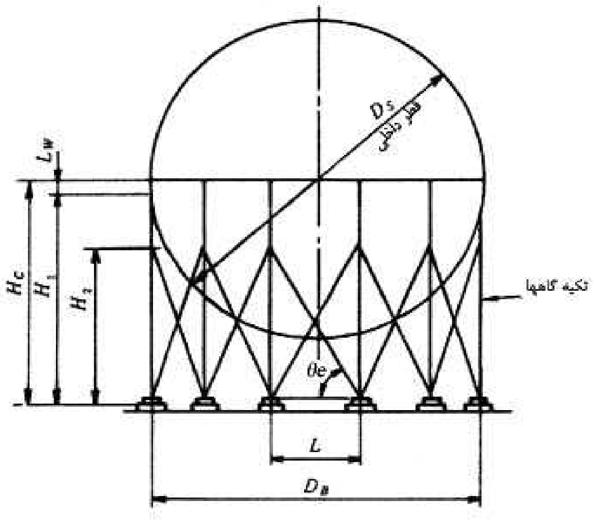


Figure 7.2: Spherical tank

$$L_W = \frac{1}{2} \sqrt{\frac{D_C D_S}{2}} \quad (7.12)$$

D_C: external diameter of upper support (mm)
D_S: internal diameter of spherical tank (mm)
I_C: inertia moment of lower support area (mm^4)

A_B: bracing section (mm^2)

θ_e: diagonal bracing angle with the horizon (deg)

C_3 and C_4: value given in table 7.1

Table 7.1: Bracing coefficient

	Tie-rod brace	Pipe brace
C_3	1.0	0.5
C_4	1.0	0.0

$$\lambda_C = \frac{H_2}{H_1} \quad (7.13)$$

H_2: height from lower surface of base plate to bracing connection (mm)

3-Modified horizontal seismic force of design

$$F_{MH} = K_{MH} W_H \quad (7.14)$$

$$F_{MV} = K_{MV} W_V \quad (7.15)$$

F_{MH}: modified horizontal seismic force of design (N)

F_{MV}: modified vertical seismic force of design (N)

W_V: sum of structure weight and fluid weight imposed on the designing point

7-2-2-Stress measurement

The stress of spherical tank should be measured as follows. When shear plated are installed, measurement of shear tension could be ignored.

1-Upper support tension

1-1-Compressive tension

$$\sigma_c = \frac{P_V}{A_{CU}} \quad (7.16)$$

σ_c: compressive stress of upper support (N/mm^2)

A_{CU}: upper base area (mm)

P_V: imposed compressive force on upper support obtained from equation 7.17 (N)

$$P_V = \frac{1}{n_s}\left\{W_V + F_V + \frac{4L(H_c - H_{21})F_H}{D_B^{\,2}}\right\} \quad (7.17)$$

n_s: number of supports

W_V: operation weight (N)

F_V: vertical seismic force of design (N)

D_B: diameter of created circle by supports' center (mm)

L: distance between adjacent supports (mm)

H_c: height from lower surface of shear plate to center of spherical body (mm)

H_{21}: height from lower surface of shear plate from bracing connection

F_H: horizontal seismic force of design (N)

1-2-Shear stress

$$\tau = \frac{2(\sqrt{C_1}KS_H + K_c F_H)}{A_{CU}K} \tag{7.18}$$

τ: shear stress of upper support (N/mm^2)

$$S_H = \frac{4\sqrt{C_1}EAK_c F_H}{(C_2EA + 4LK_c)K} \tag{7.19}$$

E: modulus of elasticity of lower support (N/mm^2)

L: distance between adjacent supports (mm)

A: obtained from equation 7.8

1-3-Bending stress

The maximum value obtained from the following equations:

$$\sigma_G = \frac{M_{G1} + M_{G2}}{Z_{CU}} \tag{7.20}$$

$$\sigma_O = \frac{|M_{O1} + M_{O2}|}{Z_{CU}} \tag{7.21}$$

σ_G: bending stress at point (G) shown in figure 7.3 (N/mm^2)

σ_O: bending stress at point (O) shown in figure 7.3 (N/mm^2)

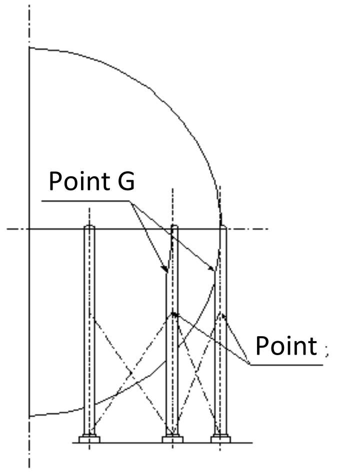


Figure 7.3: Points which should be evaluated

Z_{CU}: section modulus of upper support (mm^3)

M_{O2}, M_{O1}, M_{G2}, M_{G1}: values obtained from equations 7.22 to 7.25 (N.mm)

$$M_{G1} = \frac{\lambda_C\left\{\left(1-\lambda_C{}^2\right)H_1 - \left(3-\lambda_C{}^2\right)L_w\right\}\cdot S_H}{2} \tag{7.22}$$

$$M_{G2} = \frac{K_C\left(H_1 - L_w\right)F_H}{K} \tag{7.23}$$

$$M_{O1} = \frac{\lambda_C\left(1-\lambda_C\right)^2\left(2+\lambda_C\right)H_1 S_H}{2} \tag{7.24}$$

$$M_{O2} = \frac{K_C\lambda_C H_1 F_H}{K} \tag{7.25}$$

2-Lower support stress

2-1- Compressive stress

a) Column A:

The column location is shown at figure 7.4.

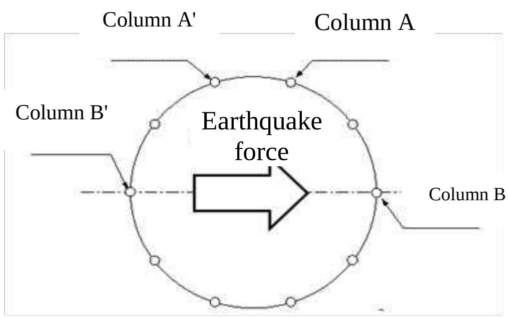


Figure 7.4: Location of column

$$\sigma_{CA} = \frac{P_A}{A_{CL}} \tag{7.26}$$

σ_{CA}: compressive stress of lower part of support (A) (N/mm^2)

A_{CL}: lower support section area (mm^2)

P_A: compressive force imposed on lower part of support (A) obtained from equation 7.27 (N)

$$P_A = \frac{1}{n}\left(W_V + F_V + \frac{4F_H H_C L}{D_B{}^2}\right)\left(C_4 + \frac{1 - C_4}{\dfrac{2A_B \sin^3\theta_e}{A_{CL}} + 1}\right) + 0.67 C_4 S_H \tan\theta_e \tag{7.27}$$

A_B: bracing section area (mm^2)

θe: convex bracing angle with horizon surface (deg)

b) Column B (see figure 7.4):

$$\sigma_{CB} = \frac{P_B}{A_{CL}} \tag{7.28}$$

σ_{CB}: compressive stress of lower part of support (B) (N/mm^2)

P_B: compressive force imposed on lower part of support (B) obtained from 7.29 (N)

$$P_B = \frac{1}{n}\left(W_V + F_V + \frac{4F_H H_C}{D_B}\right)\left(C_4 + \frac{1 - C_4}{\dfrac{2A_B \sin^3\theta_e}{A_{CL}} + 1}\right) + 0.67 C_4 S_H \tan\theta_e \tag{7.29}$$

2-2-Bending stress

a) Column (A):

$$\sigma_{bA} = \frac{\left|M_{O1} - M_{O2}\right|}{Z_{CL}} \tag{7.30}$$

σ_{bA}: bending stress of lower part of support (A) (N/mm^2)

Z_{CL}: section modulus of lower support (mm^3)

c) Column (B):

$$\sigma_{bB} = \frac{\left| 2C_4 M_{O1} \sin^2\left(\frac{180^\circ}{n}\right) - M_{O2} \right|}{Z_{CL}} \tag{7.31}$$

σ_{bB}: bending stress of lower part of support (B) (N/mm^2)

3-Bracing stress

3-1-Tensile stress

$$\sigma_t = \frac{C_3 S_H}{A_B \cos\theta_e} - \frac{(1 - C_3) P_V \sin^2\theta_e}{C_3 \left(2A_B \sin^3\theta_e + A_{CL}\right)} \tag{7.32}$$

σ_t: tensile stress of bracing (N/mm2)

3-2-Compressive stress (limited to pipe bracing)

$$\sigma_C = \frac{C_3 S_H}{A_B \cos\theta_e} - \frac{(1 - C_3) P_V \sin^2\theta_e}{C_3 \left(2A_B \sin^3\theta_e + A_{CL}\right)} \tag{7.33}$$

σ_C: compressive stress of pipe bracing (N/mm^2)

4-Anchor-bolt stress

4-1-Tensile stress

a) Column (A'):

$$\sigma_{tA'} = \frac{P_{A'}}{n_a A_b} \tag{7.34}$$

$\sigma_{tA'}$: tensile stress of anchor-bolt of support (A') (N/mm^2)

n_a: number of anchor-bolts of each support

A_b: effective section area of anchor-bolt (mm^2)

$P_{A'}$: bracing force of bolt of support (A') obtained from equation 7.35 (N), if the obtained value is negative, it would be assumed equal to zero.

$$P_{A'} = \frac{1}{n}\left(- W_V + F_V + \frac{4F_H H_C L}{D_B^{\,2}} \right) + 0.4 C_4 S_H \tan\theta_e \tag{7.35}$$

B) Column (B'):

$$\sigma_{tB'} = \frac{P_{B'}}{n_a A_b} \tag{7.36}$$

$\sigma_{tB'}$: tensile stress of anchor-bolt of support (B') (N/mm^2)

$P_{B'}$: bracing force of bolt of support (B') obtained from equation 7.37 (N), if the obtained value is negative, it would be assumed equal to zero.

$$P_{B'} = \frac{1}{n}\left(- W_V + F_V + \frac{4F_H H_C}{D_B} \right) \tag{7.37}$$

4-2-Shear stress

a) Column (A'):

$$\tau_{A'} = \frac{Q_{A'}}{n_a A_b} \tag{7.38}$$

$\tau_{A'}$: shear stress of anchor-bolt of support (A') (N/mm^2)

$Q_{A'}$: shear force of anchor-bolt of support (A') obtained from equation 7.39 (N)

$$Q_{A'} = S_H + \frac{K_C F_H}{K} \tag{7.39}$$

b) Column (B'):

$$\tau_{B'} = \frac{Q_{B'}}{n_a A_b} \tag{7.40}$$

$\tau_{B'}$: shear stress of anchor-bolt of support (B') (N/mm^2)

$Q_{B'}$: shear force of anchor-bolt of support (B') obtained from equation 7.41 (N)

$$Q_{B'} = 2C_3 S_H \sin^2\left(\frac{180^\circ}{n}\right) + \frac{K_C F_H}{K} \tag{7.41}$$

5-Shear plate stress

5-1-bending stress

$$\sigma_b = \frac{3R_{A'} b^2}{t^2} \tag{7.42}$$

σ_b: bending stress of shear plate (N/mm^2)

t: thickness of shear plate (mm)

b: height of shear stress shown in figure 7.5 (mm)

$R_{A'}$: load bearing force of concrete horizontal unit, imposed on shear plate of support (A'), obtained from equation 7.43 (N/mm^2)

$$R_{A'} = \frac{Q_{A'}}{2ab} \tag{7.43}$$

a: height of shear plate shown in figure 7.5 (mm)

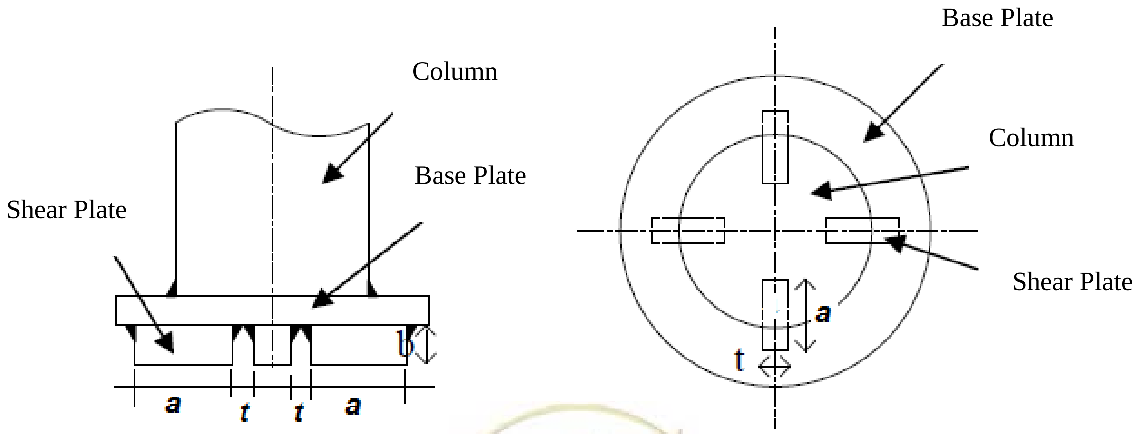


Figure 7.5: Shear plate

5-2-Shear stress

$$\tau = \frac{Q_{A'}}{2at} \tag{7.44}$$

τ: shear stress of shear plate (N/mm^2)

6-Base plate stress

6-1-Bending stress of base plate due to concrete load-bearing force

$$\sigma_{b1} = \frac{3p'_B\left(D_{bp} - D_C\right)^2}{\pi t^2 {D_{bp}}^2} \tag{7.45}$$

σ_{bl}: bending stress of base plate due to concrete load-bearing force (N/mm^2)

t: thickness of base plate (mm)

D_{bp}: diameter of base plate (mm)

D_c: external diameter of tower (mm)

P'_B: vertical reaction force imposed on base plate due to concrete of support (B), obtained from equation 7.46 (N)

$$P'_B = \frac{1}{n}\left(W_V + F_V + \frac{4F_H H_C}{D_B}\right) + 0.67C_4 S_H \tan\theta_e \tag{7.46}$$

6-2-Bending stress of base plate due to bracing force of bolt

$$\sigma_{b2} = \frac{6P_{B'}L'}{n_a\left(D_a + 2L'\right)t^2} \tag{7.47}$$

σ_{b2}: bending stress of base plate due to bracing force of bolt (N/mm^2)

D_a: diameter of anchor-bolt hole (mm)

L': value shown in figure 7.6 (mm)

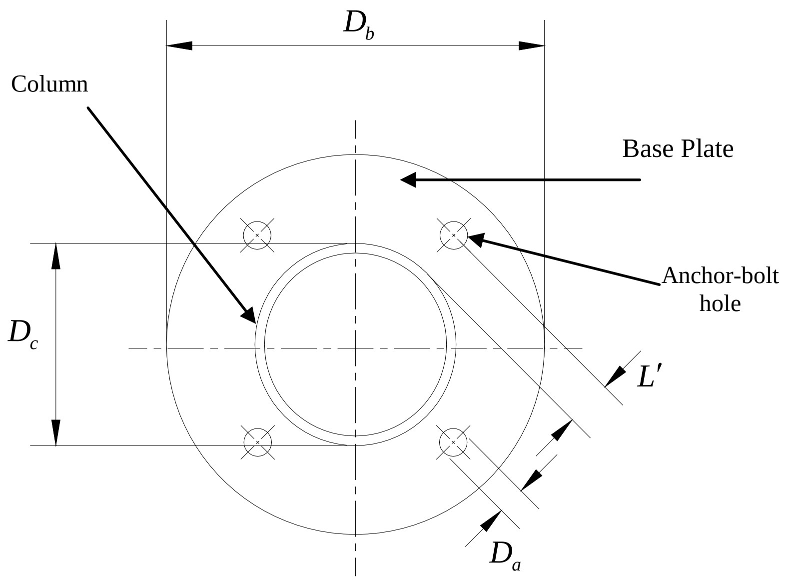


Figure 7.6: Base plate

Positions in which the stress should be measured are presented in table 7.2.

Table 7.2: Positions for stress measurement

Part which its stress is being determined	Type of Stress				
	Tension	Shear	Bending	Compressive	Buckling
Upper support		○	○	○	
Lower support			○	○	
Bracing	○			○	○
Anchor-bolt	○	○ one of these two			○
Shear plate		○	○		
Base plate			○		

7-2-3-Allowable stress

The allowable stress is determined for resistant members against compressive support members.

7-2-4-Acceptance criteria

All measured stresses must be smaller than the allowable stresses.

7-3-Ductility design

7-3-1-Damage mode

The seismic evaluation using the ductility method is performed for the following damage modes:

- 1-Upper support failure:
  - 1-1-Yielding or buckling caused by compressive stress of bending yield
  - 1-2-Shear yield
  - 1-3-Combination of compressive, bending, and shear stresses
- 2-Lower support failure:
  - 2-1-Yielding or buckling caused by compressive stress or bending yield
- 3-Brace rod failure:
  - 3-1-Tensile yield
- 4-Bracing pipe failure:
  - 4-1-Tensile yield
  - 4-2-Yielding or buckling caused by compressive stress
- 5-Anchor-bolt failure:
  - 5-1-Tensile yield
  - 5-2-Shear yield
  - 5-3-Combination of tensile and shear stresses
- 6-Shear plate failure:
  - 6-1-Bending yield
  - 6-2-Shear yield
  - 6-3-Combination of bending and shear stresses
- 7-Base plate failure:

7-1-Yielding caused by bending stress of base plate caused by concrete's crippling force
7-2-Yielding caused by bending stress of base plate caused by reaction force of anchor-bolt

7-3-2-Yield earthquake coefficient

The yield earthquake coefficient must be measured for each damage mode.

1-Yield earthquake coefficient related to damage mode of upper support

1-1-Yield earthquake coefficient equivalent to yielding or buckling caused by compressive stress or bending yielding

The yield earthquake coefficient is the minimum value of KyU obtained from equations 7.48 to 7.52.

$$K_{yc} = K_{MH} \frac{S_c - \sigma_{cN}}{\sigma_{cE}} \tag{7.48}$$

$$K_{ybG} = K_{MH} \frac{S_c}{\sigma_{GE}} \tag{7.49}$$

$$K_{ybo} = K_{MH} \frac{S_c}{\sigma_{oE}} \tag{7.50}$$

$$K_{yG} = K_{MH} \frac{1 - \frac{\sigma_{cN}}{S_c}}{\frac{\sigma_{cE}}{S_c} + \frac{\sigma_{GE}}{S_y}} \tag{7.51}$$

$$K_{yO} = K_{MH} \frac{1 - \frac{\sigma_{cN}}{S_c}}{\frac{\sigma_{cE}}{S_c} + \frac{\sigma_{OE}}{S_y}} \tag{7.52}$$

K_{yc}: yield earthquake coefficient equivalent to buckling and compressive yield

K_{ybG}: yield earthquake coefficient equivalent to bending yield at point G

K_{ybO}: yield earthquake coefficient equivalent to bending yield at point O

K_{yG}: yield earthquake coefficient equivalent to combination of compressive and bending stresses at point G

K_{yO}: yield earthquake coefficient equivalent to combination of compressive and bending stresses at point O

K_{MH}: modified horizontal seismic factor of design

S_c: yield stress in compressive side

a)Cases without reinforcing plate

$$S_c = \min\left(S_y , S_f S'\right) \tag{7.53}$$

S_y: minimum value between the minimum of yield strength at designing temperature or normal temperature of materials or the strength equivalent to 0.2% strain (N/mm^2)

S_f: 1.5

$$\frac{0.6Et}{\left(1+0.004\frac{E}{S_y}\right)D_m} \quad S'; \tag{7.54}$$

E: longitudinal modulus of elasticity at normal temperature (N/mm^2)

Dm: mean diameter of upper support (mm)

t: thickness of upper support plate (mm)

b)Cases with reinforcing plate

$$S_c = \min(F, F') \tag{7.55}$$

F: yield strength or strength equivalent to 0.2% strain (N/mm^2)

F': compressive stress for controlling buckling considering the effective slenderness coefficient (N/mm^2)

σ_{cN}: upper support compressive stress caused by operation normal load (N/mm^2)

$$\sigma_{cN} = \frac{W_V}{nA_{CU}} \tag{7.56}$$

σ_{cE}: upper support of compressive stress caused by seismic load (N/mm^2)

$$\sigma_{cE} = \frac{1}{n_s A_{CU}}\left\{F_V + \frac{4L(H_C - H_2)F_H}{{D_B}^2}\right\} \tag{7.57}$$

n_s: number of supports

A_{CU}: section area of upper support (mm^2)

W_V: operation weight (N)

F_V: obtained design’s vertical seismic force (N)

D_B: diameter of created circle by supports’ center (mm)

L: distance between adjacent supports (mm)

H_C: height from lower surface of base plate to spherical body center (mm)

H_2: height from lower surface of base plate to bracing connection (mm)

F_H: design’s horizontal seismic force (N)

W_H: operation weight equal to sum of weight and fluid’s weight

σ_{GE}: bending stress at point G caused by similar seismic load from equation 7.20

σ_{OE}: bending stress at point O caused by similar seismic load from equation 7.21

1-2-Yield earthquake coefficient related to shear yield of upper support

$$K_{ys} = K_{MH}\frac{S_y/\sqrt{3}}{\tau_{uE}} \tag{7.58}$$

K_{ys}: yield earthquake coefficient equivalent with shear yield of upper support

τ_{uE}: shear stress of upper support caused by seismic load

$$\tau_{uE} = \frac{2\left(\sqrt{C_1}KS_H + K_C F_H\right)}{A_{CU}K} \tag{7.59}$$

1-3-Yield earthquake coefficient equivalent to stress combination of upper support

The minimum value between the values obtained from equations 7.60 and 7.61 would be K_{yC} which is the yield seismic coefficient of upper support equivalent to combination of compressive, bending, and shear stresses.

$$K_{ymG} = K_{MH} \frac{-\sigma_{cN}(\sigma_{cE}+\sigma_{GE}) + \sqrt{\sigma_{cN}^2(\sigma_{cE}+\sigma_{GE})^2 - \left\{(\sigma_{cE}+\sigma_{GE})^2 + 3\tau_E^2\right\}\left(\sigma_{cN}^2 - S_y^2\right)}}{(\sigma_{cE}+\sigma_{GE})^2 + 3\tau_E^2} \quad (7.60)$$

$$K_{ymO} = K_{MH} \frac{-\sigma_{cN}(\sigma_{cE}+\sigma_{OE}) + \sqrt{\sigma_{cN}^2(\sigma_{cE}+\sigma_{OE})^2 - \left\{(\sigma_{cE}+\sigma_{OE})^2 + 3\tau_E^2\right\}\left(\sigma_{cN}^2 - S_y^2\right)}}{(\sigma_{cE}+\sigma_{OE})^2 + 3\tau_E^2} \quad (7.61)$$

K_{ymG}: yield earthquake coefficient equivalent to stress combination at point G

K_{ymO}: yield earthquake coefficient equivalent to stress combination at point O

2-Yield earthquake coefficient equivalent to damage mode of lower support

2-1-Yield earthquake coefficient equivalent to yielding and buckling caused by compressive stress and bending yield

The minimum value between values obtained from equations 7.62 to 7.67, would be K_{yL} which is the yield earthquake coefficient of lower support equivalent to yielding and buckling caused by compressive stress and bending yield.

$$K_{yCA} = K_{MH} \frac{F' - \sigma_{cLN}}{\sigma_{cAE}} \quad (7.62)$$

$$K_{ybA} = K_{MH} \frac{F}{\sigma_{bAE}} \quad (7.63)$$

$$K_{yA} = K_{MH} \frac{1 - \frac{\sigma_{cLN}}{F'}}{\frac{\sigma_{cAE}}{F'} + \frac{\sigma_{bAE}}{F}} \quad (7.64)$$

$$K_{ycB} = K_{MH} \frac{F' - \sigma_{cLN}}{\sigma_{cBE}} \quad (7.65)$$

$$K_{ybB} = K_{MH} \frac{F}{\sigma_{bBE}} \quad (7.66)$$

$$K_{yB} = K_{MH} \frac{1 - \frac{\sigma_{cLN}}{F'}}{\frac{\sigma_{cBE}}{F'} + \frac{\sigma_{bBE}}{F}} \quad (7.67)$$

K_{yCA}: yield earthquake coefficient of column (A) equivalent to buckling and yielding due to pressure

K_{ybA}: yield earthquake coefficient of column (A) equivalent to shear yield

K_{yA}: yield earthquake coefficient of column (A) equivalent to combination of compressive and bending stresses

K_{ycB}: yield earthquake coefficient of column (B) equivalent to buckling and yielding due to pressure

K_{ybB}: yield earthquake coefficient of column (B) equivalent to shear yield

K_{yB}: yield earthquake coefficient of column (B) equivalent to combination of compressive and bending stresses

F': compressive stress for controlling buckling considering the effective slenderness coefficient

σ_{cLN}: compressive stress of lower support caused by operation normal load (N/mm^2)

$$\sigma_{cLN} = \frac{W_V}{nA_{CL}} C_5 \tag{7.68}$$

σ_{cAE}: compressive stress of column (A) caused by seismic load (N/mm^2)

$$\sigma_{cAE} = \frac{1}{nA_{CL}} \left(F_V + \frac{4F_H H_C L}{{D_B}^2} \right) C_5 + \frac{0.67 C_4 S_H \tan\theta_e}{A_{CL}} \tag{7.69}$$

C_S: value obtained from equation 7.70

$$C_5 = C_4 + \frac{1 - C_4}{\dfrac{2A_B \sin^3\theta_e}{A_{CL}} + 1} \tag{7.70}$$

σ_{bAE}: bending stress of column (A) caused by seismic load (N/mm^2)

$$\sigma_{bAE} = \frac{|M_{O1} - M_{O2}|}{Z_{Cl}} \tag{7.71}$$

σ_{cBE}: compressive stress of column (B) caused by seismic load (N/mm^2)

$$\sigma_{cBE} = \frac{1}{nA_{CL}} \left(F_V + \frac{4F_H H_C}{D_B} \right) C_5 + \frac{0.67 C_4 S_H \tan\theta_e}{A_{CL}} \tag{7.72}$$

σ_{bBE}: bending stress of column (B) caused by seismic load (N/mm^2)

$$\sigma_{bBE} = \frac{\left| 2C_4 M_{O1} \sin^2\left(\dfrac{180^\circ}{n}\right) - M_{O2} \right|}{Z_{CL}} \tag{7.73}$$

3-Yield earthquake coefficient related to damage mode of bracing rod

$$K_{yT} = K_{MH} \frac{F}{\sigma_{TE}} \tag{7.74}$$

K_{yT}: yield earthquake coefficient related to tensile yield of bracing rod

F: yield strength or strength equivalent to 0.2% strain of bracing rod steel (N/mm^2)

σ_{TE}: tensile stress of bracing rod caused by seismic load (N/mm^2)

$$\sigma_{TE} = \frac{S_H}{A_B \cos\theta_e} \tag{7.75}$$

4-Yield earthquake coefficient related to damage mode of pipe bracing

4-1-yield earthquake coefficient related to tensile yield of pipe bracing

$$K_{ytP} = K_{MH} \frac{F + \sigma_{cPN}}{\sigma_{tPE}} \tag{7.76}$$

K_{ytP}: yield earthquake coefficient equivalent to tensile yield of pipe bracing

σ_{cPN}: compressive stress of pipe bracing caused by operation normal load (N/mm^2)

$$\sigma_{cPN} = \frac{W_V \sin^2 \theta_e}{n(2A_B \sin^3 \theta_e + A_{CL})} \tag{7.77}$$

σ_{tPE}: tensile stress of pipe bracing caused by seismic load (N/mm^2)

$$\sigma_{tPE} = \frac{0.5S_H}{A_B \cos\theta_e} - \frac{1}{n}\left\{F_V + \frac{4L(H_C - H_{21})F_H}{D_B^2}\right\}\frac{\sin^2\theta_e}{2A_B \sin^3\theta_e + A_{CL}} \tag{7.78}$$

4-2-yield earthquake coefficient related to buckling or yielding due to pressure

$$K_{ycP} = K_{MH}\frac{F' - \sigma_{cPN}}{\sigma_{cPE}} \tag{7.79}$$

K_{ycP}: yield earthquake coefficient related to buckling and yielding due to pressure

F': compressive pressure for controlling buckling considering the effective slenderness coefficient (N/mm2)

$$\sigma_{cPE} = \frac{0.5S_H}{A_B \cos\theta_e} + \frac{1}{n}\left\{F_V + \frac{4L(H_C - H_{21})F_H}{D_B^2}\right\}\frac{\sin^2\theta_e}{2A_B \sin^3\theta_e + A_{CL}} \tag{7.80}$$

5-yield earthquake coefficient related to damage mode of anchor-bolt

5-1-Yield earthquake coefficient related to tensile yield of anchor-bolt

The minimum value obtained from the following equations would be the yield earthquake coefficient related to tensile yield of anchor-bolt.

$$K_{ytA'} = K_{MH}\frac{F - \sigma_{cPN}}{\sigma_{tA'E}} \tag{7.81}$$

$$K_{ytB'} = K_{MH}\frac{F_A + \sigma_N}{\sigma_{tB'E}} \tag{7.82}$$

$K_{ytA'}$ and $K_{ytB'}$: yield earthquake coefficient related to tensile yield of anchor-bolt of columns A' and B'

σ_N: compressive stress of anchor-bolt caused by operation normal load (N/mm^2)

$$\sigma_N = \frac{W_V}{nn_a A_b} \tag{7.83}$$

$\sigma_{tA'E}$: tensile stress of anchor-bolt of column A' caused by seismic load

$$\sigma_{tA'E} = \frac{1}{n_a A_b}\left\{\frac{1}{n}\left(F_V + \frac{4F_H H_C L}{D_B^2}\right) + 0.4C_4 S_H \tan\theta_e\right\} \tag{7.84}$$

$\sigma_{tB'E}$: tensile stress of anchor-bolt of column B' caused by seismic load

$$\sigma_{tB'E} = \frac{1}{nn_a A_b}\left(F_V + \frac{4F_H H_C}{D_B}\right) \tag{7.85}$$

n_a: number of anchor-bolts for each support

A_b: effective section area of anchor-bolt (mm^2)

5-2-Yield earthquake coefficient related to shear yield of anchor-bolt

The minimum value obtained from equations 7.86 and 7.87 would be the yield earthquake coefficient equivalent to shear yield of anchor-bolt, K_{ysB}.

$$K_{ysA'} = K_{MH}\frac{F/\sqrt{3}}{\tau_{A'E}} \tag{7.86}$$

$$K_{ysB'} = K_{MH}\frac{F/\sqrt{3}}{\tau_{B'E}} \tag{7.87}$$

$K_{ysA'}$: yield earthquake coefficient related to tensile yield of anchor-bolt of column (A')
$K_{ysB'}$: yield earthquake coefficient related to tensile yield of anchor-bolt of column (B')
$\tau_{A'E}$: shear stress of column A' obtained from equation 7.88

$$\tau_{A'E} = \frac{Q_{A'}}{n_a A_b} \tag{7.88}$$

$Q_{A'}$: shear force of anchor-bolt of column (A') obtained from equation 7.89 (N)

$$Q_{A'} = S_H + \frac{K_C F_H}{K} \tag{7.89}$$

$\tau_{B'E}$: shear stress of column B' obtained from equation 7.90 (N/mm^2)

$$\tau_{B'E} = \frac{1}{n_a A_b}\left\{2C_a S_H \sin^2\left(\frac{180°}{n}\right) + \frac{K_C F_H}{K}\right\} \tag{7.90}$$

5-3-Yield earthquake coefficient of anchor-bolt equivalent to combination of tensile and shear stresses

The minimum value obtained from equations 7.91 and 7.92 would be yield earthquake coefficient related to combination of tensile and shear stresses, K_{ym}.

$$K_{ymA'} = K_{MH}\frac{1.4F + \sigma_N}{\sigma_{tA'E} + 1.6\tau_{A'E}} \tag{7.91}$$

$$K_{ymB'} = K_{MH}\frac{1.4F + \sigma_N}{\sigma_{tB'E} + 1.6\tau_{B'E}} \tag{7.92}$$

$K_{ymA'}$: yield earthquake coefficient related to stress combination of anchor-bolt of column A'
$K_{ymB'}$: yield earthquake coefficient related to stress combination of anchor-bolt of column B'

6-Yield earthquake coefficient equivalent to damage mode of shear plate

6-1-Earthquake coefficient equivalent to bending yield of shear plate

$$K_{ybS} = K_{MH}\frac{F}{\sigma_{bE}} \tag{7.93}$$

K_{ybS}: yield earthquake coefficient related to bending yield of shear plate
F: yield strength or strength equivalent to 0.2% strain of shear plate steel (N/mm^2)
σ_{bE}: bending strength of shear plate caused by seismic load

$$\sigma_{bE} = 3\frac{Q_{A'}}{2ab}.\frac{b^2}{t^2} \tag{7.94}$$

α: length of shear plate (mm) (see figure 7.5)
b: height of shear plate (mm) (see figure 7.5)

t: thickness of shear plate (mm)

6-2-Yield earthquake coefficient equivalent to shear yield of shear plate

$$K_{ysS} = K_{MH}\frac{F/\sqrt{3}}{\tau_E} \tag{7.95}$$

K_{ysS}: yield earthquake coefficient equivalent to shear yield of shear plate

τ_E: shear stress of shear plate caused by seismic load (N/mm^2)

$$\tau_E \quad \frac{Q_{A'}}{2at} \tag{7.96}$$

6-3-Yield earthquake coefficient equivalent to combination of bending and shear stresses of shear plate

$$K_{yms} = K_{MH}\frac{F}{\sqrt{\sigma_{bE}{}^2 + 3\tau_E{}^2}} \tag{7.97}$$

7-Yield earthquake coefficient equivalent to damage mode of base plate

7-1-Yield earthquake coefficient equivalent to bending yield of base plate caused by concrete's load-bearing force

$$K_{yBR} = K_{MH}\frac{F - \sigma_{RN}}{\sigma_{RE}} \tag{7.98}$$

K_{yBR}: yield earthquake coefficient equivalent to bending yield of base plate caused by concrete's load-bearing force

σ_{RN}: bending stress of base plate caused by operation normal load (N/mm^2)

$$\sigma_{RN} = 3\frac{W_V\left(D_{bp} - D_c\right)}{n\pi t^2 D_{bp}{}^2} \tag{7.99}$$

σ_{RE}: bending stress of base plate caused by seismic load (N/mm^2)

$$\sigma_{RE} = 3\left\{\frac{1}{n}\left(F_V + \frac{4F_H H_C}{D_{bp}}\right) + 0.67C_4 S_H \tan\theta_e\right\}\frac{\left(D_{bp} - D_C\right)^2}{\pi t^2 D_{bp}{}^2} \tag{7.100}$$

D_{bp}: diameter of base plate (mm) (see figure 7.6)

D_c: external diameter of support (mm) (see figure 7.6)

t: thickness of base plate (mm)

7-2-Yield earthquake coefficient equivalent to bending yield of base plate due to bracing force of bolt

$$K_{yBT} = K_{MH}\frac{F + \sigma_{TN}}{\sigma_{TE}} \tag{7.101}$$

K_{yBT}: yield earthquake coefficient related to bending yield of base plate due to bracing force of bolt

σ_{TN}: reduction range of bending stress of base plate caused by operation normal load (N/mm^2)

$$\sigma_{TN} = \frac{6W_V L'}{nn_a (D_a + 2L')t^2} \qquad (7.102)$$

σ_{TE}: bending stress of base plate caused by seismic load (N/mm^2)

$$\sigma_{TE} = 6\left\{\frac{1}{n}\left(F_V + \frac{4F_H H_C}{D_b}\right) + 0.67C_4 S_H \tan\theta_e\right\}\frac{L'}{n_a(D_a + 2L')t^2} \qquad (7.103)$$

D_a: diameter of anchor-bolt hole (mm)

7-3-3-Ductility factor

The ductility factor of spherical tank for each damage mode could be obtained using equation 7.104.

$$\mu_p = \frac{1}{4C}\left\{\left(\frac{K_{MH}}{K_y}\right)^2 - 1\right\} \qquad (7.104)$$

μ_p: ductility factor for each damage mode, if $K_{MH} \leq K_y$ then $\mu_p = 0$

K_{MH}: modified horizontal seismic factor of design

K_y: yield earthquake coefficient for each damage mode

C: as follows based on characteristics of each damage mode

1-Upper support damage:

-Buckling or compressive and bending yielding, shear yield, and combination of compressive, bending, and shear stresses C = 2.0

2-Lower support damage:

-Buckling or compressive and bending yielding C = 2.0

3-Bracing rod damage:

-Tensile yield C = 1.0

4-Bracing pipe damage:

-Tensile yield, buckling or yielding C = 2.0

5-Anchor-bolt damage:

-Tensile yield and combination of tensile and shear stresses C = 1.0

-Shear yield C = 2.0

6-Shear plate damage:

-Bending yield, shear yield, and combination of bending and shear stresses C = 2.0

7-Base plate damage:

Bending yield caused by concrete’s crippling force and bending yield caused by bracing force of bolt C = 2.0

7-3-4-Allowable ductility factor

The allowable ductility factor is calculated for each damage mode of spherical tank.

1-Upper support damage:

-Buckling or bending yielding, shear yield, and combination of compressive, bending, and shear stresses μ_{pa} = 1.0

2-Lower support damage:

-Buckling or bending yielding μ_{pa} = 1.0

3-Bracing rod damage:

-Tensile yield $\mu_{pa} = 1.8$

4-Bracing pipe damage:

-Tensile yield, $\mu_{pa} = 1.0$

-buckling or yielding $\mu_{pa} = 0.35$

5-Anchor-bolt damage:

-Tensile yield and combination of tensile and shear stresses $\mu_{pa} = 1.8$

Shear yield $\mu_{pa} = 0.35$

6-Shear plate damage:

-Bending yield, shear yield, and combination of bending and shear stresses $\mu_{pa} = 0.35$

7-Base plate damage:

-Bending yield caused by concrete's crippling force and bending yield caused by bracing force of bolt $\mu_{pa} = 0.35$

7-3-5-Acceptance criteria

Equation 7.105 should be satisfied for each damage mode of spherical tank.

$$\mu_p \le \mu_{pa} \tag{7.105}$$

μ_p: ductility factor of each damage mode

μ_{pa}: allowable ductility factor of each damage mode

Chapter 8

Seismic Design and Safety Control of Cylindrical Tanks

8-1-General procedure for seismic designing

When earthquake risk level-1 is in use, the allowable stress, and whenever earthquake risk level-2 is in use, the ductility design method could be applied.

In this chapter, the target facilities of seismic designing are cylindrical tanks which their capacity is 3 tons from weight perspective or 300 m^3 from volume perspective.

In seismic designing of cylindrical tanks, the seismic input should be considered for both each mode of liquid rigid movement or tank and turbulent mode.

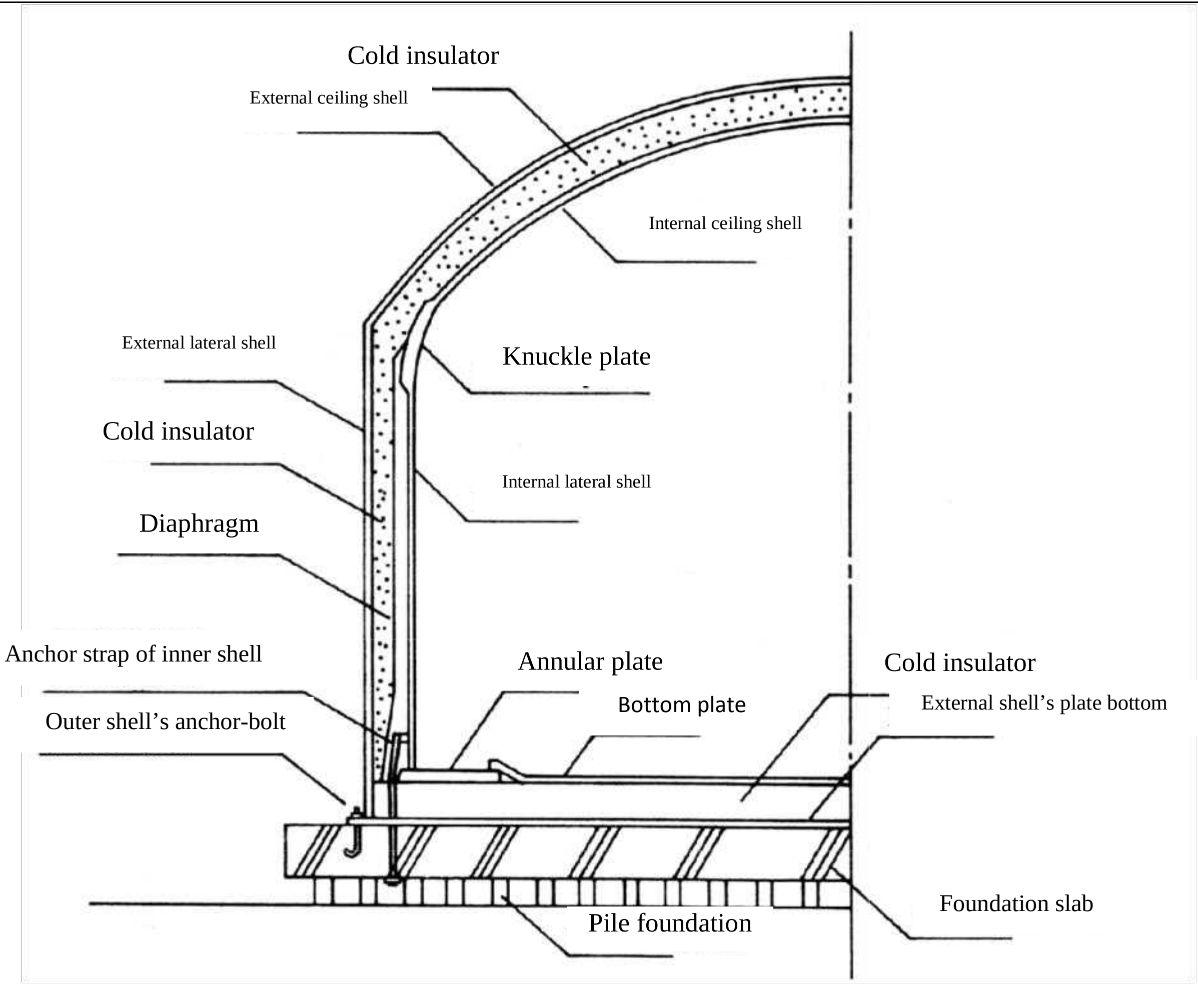


Figure 8.1: Sample cross section of cylindrical tank

Figure (8.1) shows a sample cross-section of cylindrical tanks. Cylindrical tanks which their designing procedure will be described here are pressure tanks with fixed ceiling. Thus, the designing procedure of tanks with floating ceiling would not be presented in this section.

The seismic designing procedure related to turbulent mode of tanks is only applied at risk level-2 for cylindrical tanks.

The designing should be performed for both risk levels 1 and 2 of earthquake. The damage modes are presented in figure (2.8).

1-Wall (shell) damage mode of tank side

a) Buckling caused by risk level-1 earthquake

The elastic buckling of upper parts of lateral wall and the elastic and plastic buckling of lower parts of lateral wall, "elephant's foot buckling", should be examined.

b) Buckling caused by risk level-1 earthquake

The dynamic pressure of fluid on lateral wall is more than the pressure on ceiling and its intensity is less than the pressure imposed at risk level-1 of earthquake, therefore the elastic buckling for lateral walls should be examined at every height.

2-Damage modes of lateral anchor belts

a) Tensile yield caused by risk level-1 earthquake

b) Tensile yield caused by risk level-2 earthquake

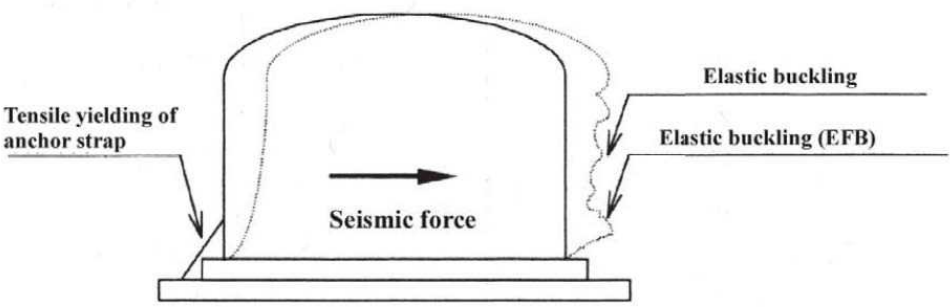


Figure 8.2: Damage modes of cylindrical tank

8-2-Designing using allowable stress method

8–2–1-Response analysis method

Based on size and natural period of cylindrical tank, the seismic performance could be evaluated using the pseudo-static method, modified pseudo-static method, and other methods.

The designing should be controlled for buckling of tank's lateral wall and tensile yield of anchor belt. The classification methods are presented in table (8.1).

Table 8.1: Analysis methods

Dimensions / Risk Level	a) Height and external diameter equal to or less than 10m with medium and low importance	b) Internal diameter equal to or less than 20m and the ratio of lateral wall to internal diameter equal to or less than 1.25	c) otherwise than a) and b) (large dimensions)
Risk level-1	Pseudo-static method, modified pseudo-static method, modal analysis or time-history response analysis	modified pseudo-static method, modal analysis or time-history response analysis	Similar to previous column
Risk level-2	time-history response analysis	time-history response analysis	Similar to previous column

8-2-2-Pseudo-static method

The pseudo-static method could be applied for structures with medium and low importance factor and diameter less than 10m.

$$K_{SH} = \beta_4 K_H \tag{8.1}$$

K_{SH}: static horizontal seismic factor

K_H: horizontal seismic intensity at surface based on loading guide and seismic analysis of vital vessel

β_4: horizontal response magnification factor

$$F_{SH} = K_{SH} W_H \tag{8.2}$$

F_{SH}: static horizontal seismic force of design (N)

W_H: sum of tank weight and fluid's effective weight

The effective weight could be obtained by multiplying fluid's weight to ratio of effective weight, f_1, shown in figure (8.3).

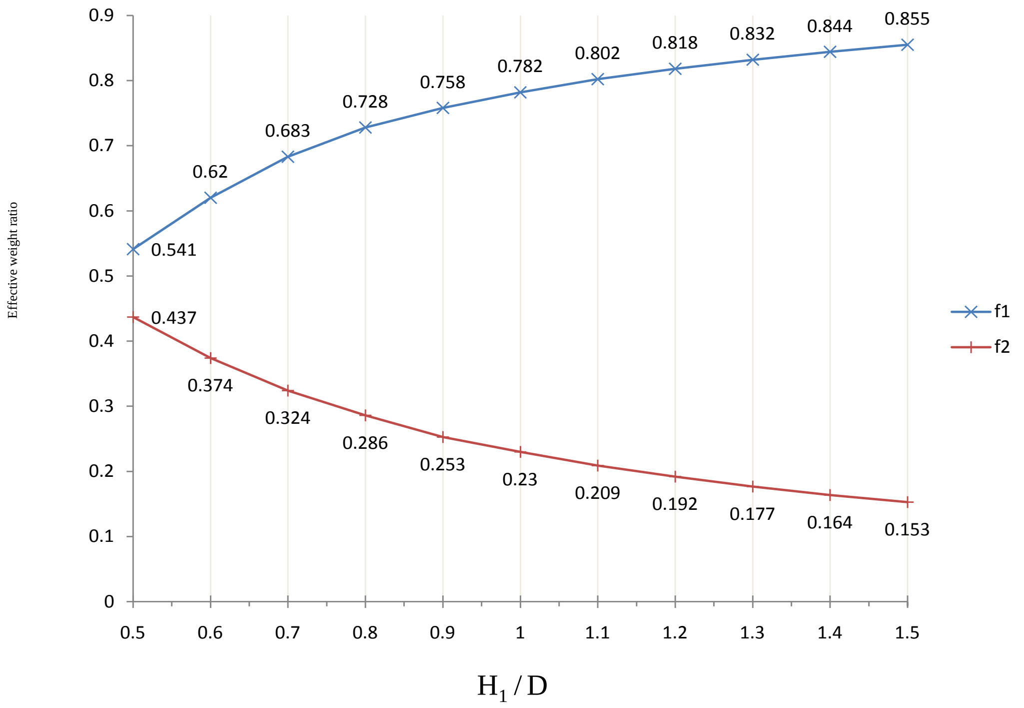


Figure 8.3: Measuring fluid's effective weight

8-2-3-Modified pseudo-static method

For calculating the modified factors of earthquake, the natural period of tank should be calculated as follows.
Calculating the natural period of cylindrical tank:

$$T = \frac{2}{\lambda}\sqrt{\frac{W_0}{\pi g E t_{1/3}}} \tag{8.3}$$

T: Natural period (s)

$$\lambda = 0.067\left(\frac{H_L}{D_0}\right)^2 - 0.30\left(\frac{H_L}{D_0}\right) + 0.46 \tag{8.4}$$

H_L: upmost height of fluid level (m)

E: longitudinal modulus of elasticity of tank's lateral wall (N/mm^2)

D_0: internal diameter of tank (m)

$t_{1/3}$: thickness of lateral wall at one third of tank's lateral wall height (mm)

W_0: operational weight (N)

W_0 is the sum of following weights:

a) Internal lateral wall weight

b) Half of cold insulator weight. If the insulator is separated from the lateral wall, like diaphragm, then its weight is not considered.

c) Internal ceiling weight (if the tank's ceiling is single-wall, then it would be the ceiling's weight itself).

d) Weight of cold insulator in ceiling

e) Total weight of fluid

8–2–4-Stress measurement

The compressive stress of lateral wall and tensile stress of anchor belt should be calculated for earthquake risk level-1.

1-Lateral wall stress

$$\sigma_c = -\frac{P_{0\min} D_0}{4t} + \frac{(1 + K_{MV})(W_s' + W_r)}{\pi D_0 t} + \frac{4M_{10}}{\pi D_0^{\ 2} t} \qquad (8.5)$$

σ_c: compressive stress of lateral wall for controlling buckling (N/mm^2)

t: lateral wall thickness except for allowable corrosion amount (mm)

P_{0min}: minimum internal pressure during normal operation = 0.003MPa

K_{MV}: vertical modified seismic factor of design

W'_s: lateral wall weight imposed on a point where stress is being measured (N)

W_r: total ceiling weight (including snow load) (N)

M_{10}: overturn moment imposed on a point where stress is being measured (N)

$$M_{10} = K\left(1 - \frac{h_P}{H_L}\right)^2 \left(W_s H_s + W_r H_r + W_l H_L\right) \qquad (8.6)$$

W_s: total lateral wall weight (N)

K: equal to K_{MH} or K_{SH}

H_L: maximum height of water elevation (mm)

h_p: height of determined point from bottom plate (mm)

H_s: height of lateral wall's center of mass from bottom plate (mm)

H_r: height of ceiling's center of mass from tank's bottom (mm)

W_l: fluid's effective weight at risk level-1 of earthquake (N)

$$W_1 = f_1 W_l \qquad (8.7)$$

W_1: fluid's weight (N)

f_1: effective weight ratio of fluid (see figure (8.4))

H_1: height of fluid's effective center of mass from tank's bottom plate (mm)

$$H_1 = h_1 H_L \qquad (8.8)$$

h_1: value obtained from figure (8.4)

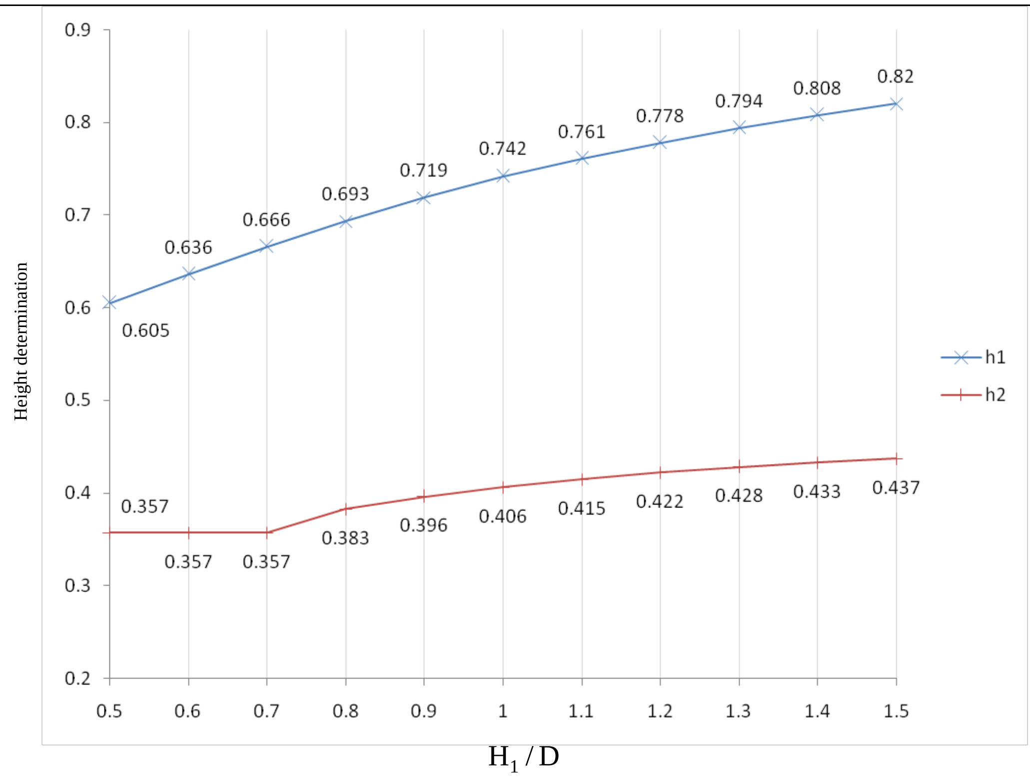


Figure 8.4: h_1 and h_2 factors

2-Caused anchor belt stress

$$\sigma_t = \frac{1}{N_{as} A}\left\{\frac{\pi D_0{}^2 P_0}{4} - (1 - K_{MV})(W_s + W_r) + \frac{4M_{10}}{D_0}\right\} \tag{8.9}$$

σ_t: tensile stress of anchor belt (N/mm^2)

N_{as}: number of anchor belts

A: section area of anchor belt (mm^2)

P_0: normal pressure (MPa)

K_{MV}: vertical modified seismic factor of design

3-Stresses except for cases described in previous section

The stresses should be calculated for the point shown in table (8.2) based risk level-1 of earthquake.

Table 8.2: Determined stresses

Type of Stress	Stress Evaluation Part
Primary membrane stress	Tank's wall, knuckle plate, ceiling plate, and annular plate except for stress concentration part
Compressive stress	
Primary local membrane stress	Connections between ceiling and knuckle plate, knuckle plate and shell plate, shell plate and annular plate
Primary bending stress	
Secondary membrane stress plus bending stress	
Tensile stress	Anchor belt

8-2-5-Allowable stress

The allowable stress of lateral shell is defined similar to compressive members and for anchor belt similar to support members.

1-Allowable compressive stress of tank's lateral wall

$$S' = \frac{Et}{3D_0} \tag{8-10}$$

S': allowable compressive stress (N/mm^2)

t: lateral wall thickness except for allowable corrosion amount (mm)

E: longitudinal modulus of elasticity at designing temperature (N/mm^2)

2-Allowable tensile stress of anchor belt

The allowable tensile stress is the minimum value between yield stress, strength equivalent to 0.2% strain of steel, and 70% of tensile strength (N/mm^2).

8-2-6-Acceptance criteria

All measured stresses should be smaller than allowable stresses.

8-3-Ductility method design

8-3-1-Risk level-2 of earthquake

The risk level-2 of earthquake on ground surface could be defined as displacement and given velocity, and based on natural period of turbulence which is obtained from the equation between fluid's maximum height and internal diameter.

1-Natural period of turbulence

$$T_S = 2\pi\sqrt{\frac{D_0}{3.682g}}\coth\left(\frac{3.682H_l}{D_0}\right) \tag{8.11}$$

T_S: natural period of turbulence (sec)

D_0: tanks internal diameter (m)

H_l: fluid's maximum height (m)

2-Velocity and ground displacement for risk level-2 at horizontal direction

When $T_s \leq 7.5$:

$$V_H = 100\beta_1\beta_2 \tag{8.12}$$

When $T_S > 7.5$:

$$D_H = 120\beta_1\beta_2 \quad (8.13)$$

V_H: ground velocity for risk level-2 of earthquake at horizontal direction (cm/s)

D_H: half of ground displacement domain for risk level-2 of earthquake at horizontal direction (cm)

β_1: importance factor

β_2: design's basis acceleration ratio

The horizontal acceleration at α2 level (m/s^2) could be measure based on turbulence period from either equation (8.14) or equation (8.15).

$$a_2 = \frac{2\pi}{T_S} V_H \quad (8.14)$$

$$a_2 = \left(\frac{2\pi}{T_S}\right)^2 D_H \quad (8.15)$$

The design's horizontal seismic intensity, KH2, for risk level-2 of earthquake could be obtained from equation (8.17).

$$K_{MH2} = 9K_{H2} \quad (8.17)$$

8-3-2-Damage modes

1-Wall failure of lateral shell

- Buckling caused by risk level-2 earthquake

2-Failure of anchor belt

-Tensile yielding caused by risk level-2 earthquake

8-3-3-Yield earthquake coefficient

1-Yield earthquake coefficient for wall buckling of lateral shell

$$K_{ycS2} = K_{MH2} \frac{S_c + \sigma_p - \sigma_o}{\sigma_{E2H}} \quad (8.18)$$

S_C: buckling stress (N/mm^2)

$$S_C = \frac{Et}{2.5D_0} \quad (8.19)$$

σ_p: mean axial tensile stress caused by internal pressure (N/mm^2)

$$\sigma_p = \frac{P_{0\min} D_0}{4t} \quad (8.20)$$

σ_o: mean axial compressive stress caused by tank's weight (N/mm^2)

$$\sigma_o = \frac{(W_s + W_r)}{\pi D_0 t} \quad (8.21)$$

σE2H: compressive stress at a point with height hp caused by overturn moment, when the seismic factor is imposed at risk level-2. (N/mm^2)

$$\sigma_{E2H} = K_{MH2} \frac{4(1 - h_p / H_1)^{1.3} W_2 H_{20}}{\pi D_0{}^2 t} \quad (8.22)$$

K_{MH2}: horizontal modified factor of design caused by risk level-2 earthquake

$$K_{MH2} = \frac{9\alpha_2}{g} \tag{8.23}$$

α_2: horizontal acceleration at ground level based on turbulence's natural period (cm/s^2) (see table (8.3))

Table 8.3: seismic acceleration of risk level-2

TS	α2
.5 or less	$V_H \frac{2\pi}{T_S}$
More than 7.5	$D_H \left(\frac{2\pi}{T_S}\right)^2$

The fluid's effective weight (N) is obtained from equation (8.24).

$$W_2 = f_2 W_1 \tag{8.24}$$

f_2: see figure (8.3)

$$H_{20} = h_2 H_L \tag{8.25}$$

h_2: see figure (8.4)

2-Yield earthquake coefficient of lowest part of lateral wall caused by circumferential stress and axial compressive stress

At the lowest wall of lateral shell, the yield earthquake coefficient due to yield caused by compressive stress at circumferential direction and axial compressive stress is calculated from equation (8.26).

$$K_{ybS1} = kK_{MH} \tag{8.26}$$

k: ratio factor against design's modified seismic factor. It is the minimum value between k_1 and k_2. If k_2 is negative, then k is k_1.

$$k_1 = \frac{S_{cr} - \sigma_{1o} - S_{cr}\frac{\sigma_{0h}}{S_y}}{\sigma_{E1H} + \sigma_{E1V} + S_{cr}\frac{\sigma_{Eh}}{S_y}} \tag{8.27}$$

$$k_2 = \frac{{}_bS_{cr} - \sigma_{1o} - {}_bS_{0.3}\frac{\sigma_{0h}}{S_y}}{\sigma_{E1H} + \sigma_{E1V} + {}_bS_{0.3}\frac{\sigma_{Eh}}{S_y}} \tag{8.28}$$

σ_{1o}: mean axial compressive stress caused by tank's weight (N/mm^2)

$$\sigma_{1o} = \frac{W_r + W_{1s}}{2\pi Rt} \tag{8.29}$$

σ_{Oh}: circumferential membrane stress of tank's lateral shell during normal operation

$$\sigma_{0h} = \frac{R}{t}\left(P_s + P_{0\max}\right) \tag{8.30}$$

σ_{Eh}: circumferential membrane stress of tank's lateral shell during earthquake

$$\sigma_{Eh} = \frac{R}{t} P_s \left\{ K_{MV} + K_{MH} \frac{\sqrt{3}}{2} \tanh(\sqrt{3} \frac{R}{H_L}) \right\} \tag{8.31}$$

σ_{ElH}: axial compressive stress caused by overturn moment when modified horizontal seismic factor, K_{MH}, is imposed.

$$\sigma_{E1H} = K_{MH} \frac{(W_{1s} + W_r + W_1) H_G}{\pi R^2 t} \tag{8.32}$$

σ_{ElV}: axial compressive stress caused by design's modified vertical seismic factor, K_{MV} (N/mm^2)

$$\sigma_{E1V} = K_{MV} \frac{W_r + W_{1s}}{2\pi Rt} \tag{8.33}$$

R: radius of tank's lateral shell (mm)

t: thickness of lowest part of tank's lateral shell sheet (mm)

P_{0max}: maximum pressure during normal operation (MPa)

P_s: fluid's hydrostatic pressure (MPa)

H_G: height of tank's center of gravity (mm)

W_r: total weight of ceiling (including snow) (N)

W_{ls}: total wall weight of tank's lateral shell

W_l: fluid's effective weight (see previous section) (N)

S_{cr}: value obtained from equation (8.34) (N/mm^2)

If $0.807 \leq \frac{R}{t}$:

$$S_{cr} = \frac{0.8E}{\sqrt{3(1-\nu^2)}} . \frac{t}{R} \tag{8.34}$$

If $0..69 \frac{E}{S_y} \leq \frac{R}{t} \leq 0.807 \frac{E}{S_y}$:

$$S_{cr} = 0.6S_y + 0.4S_y \frac{0.807 - \frac{R}{t} . \frac{S_y}{E}}{0.738} \tag{8.35}$$

If $\frac{R}{t} \leq 0.69 \frac{E}{S_y}$:

$$S_{cr} = S_y \tag{8.36}$$

${}_bS_{cr}$: value obtained from equation (8.37) (N/mm^2)

If $2.106 \left(\frac{E}{S_y} \right)^{0.78} \leq \frac{R}{t}$:

$${}_bS_{cr} = 0.6E \left\{ 1 - 0.731 \left[1 - \exp\left(-\frac{1}{16} \sqrt{\frac{R}{t}} \right) \right] \right\} \frac{t}{R} \tag{8.37}$$

If $0.274 \left(\frac{E}{S_y} \right)^{0.78} \leq \frac{R}{t} \leq 2.106 \left(\frac{E}{S_y} \right)^{0.78}$:

$$ {}_{b}S_{cr} = 0.6S_y + 0.4S_y \left[2.106 - \frac{R}{t}\left(\frac{S_y}{E}\right)^{0.78} \right] \frac{1}{1.832} \tag{8.38} $$

If $\frac{R}{t} \le 0.274\left(\frac{E}{S_y}\right)^{0.78}$:

$$ {}_{b}S_{cr} = S_y \tag{8.39} $$

${}_{b}S_{0.3}$: value obtained from equation (8.40) (N/mm^2)

$$ {}_{b}S_{0.3} = \frac{0.7S_{cr} - {}_{b}S_{cr}}{0.3} \tag{8.40} $$

S_y: yield stress of lateral shell (N/mm^2)

v: Poisson's ratio

3-Yield earthquake coefficient for tensile stress of anchor belt

$$ K_{yA2} = K_{MH2} \frac{S_y - \sigma_{Ap} + \sigma_{A0}}{\sigma_{AEH2}} \tag{8.41} $$

σ_{Ap}: tensile stress caused by internal pressure (N/mm^2)

$$ \sigma_{Ap} = \frac{\pi D_0{}^2 P_{0\max}}{4N_{as}A} \tag{8.42} $$

σ_{A0}: compressive stress caused by tank's weight (N/mm^2)

$$ \sigma_{A0} = \frac{W_s + W_r}{N_{as}A} \tag{8.43} $$

σ_{AEH2}: tensile stress caused by overturn moment when risk level-2 earthquake is imposed (N/mm^2).

$$ \sigma_{AE2H} = K_{MH2} \frac{4W_2H_{20}}{D_0N_{as}A} \tag{8.44} $$

N_{as}: number of anchor belts

A: section area of anchor belt (mm^2)

8-3-4-Ductility factor

The ductility factor, μ_p, is calculated from equation (8.45) for buckling of tank's lateral shell and anchor belt yield.

$$ \mu_P = \frac{1}{4C}\left\{\left(\frac{K_{MH}}{K_y}\right)^2 - 1\right\} \tag{8.45} $$

μ_p: ductility factor for each damage mode, if $K_{MH} < K_y$, then $\mu_p = 0$

K_{MH}: modified horizontal seismic factor of design for risk level-2 earthquake

K_y: yield earthquake coefficient for each damage mode

Based on damage mode, the value of C is defined as follows:

(1)Buckling of lateral shell caused by risk level-2

(2)Tensile yield of anchor belt caused by risk level-2

Assuming the fully elastoplastic behavior, C for lateral shell would be equal to 2.

Assuming the plastic behavior, C for anchor belt would be equal to 1.

8-3-5-Allowable ductility coefficient

The allowable ductility coefficient, μ_{pa}, should be calculated for each damage mode of risk level-2 earthquake.

1-The allowable ductility factor for buckling of tank's lateral shell

If $\frac{\sigma_0}{S_c} \leq 0.2$:

$$\mu_{pa} = 0.35 \tag{8.46}$$

If $\frac{\sigma_0}{S_c} > 0.2$:

$$\mu_{pa} = 0.13 \tag{8.47}$$

σ_0: mean axial compressive stress (N/mm^2)

$$\sigma_0 = \frac{(W_r + W_s) - P_{0\min}\pi R^2}{2\pi Rt} \tag{8.48}$$

S_C: elastic buckling stress when mean axial compressive stress is not available

$$S_C = \frac{Et}{2.5D_0} \tag{8.49}$$

2-The allowable ductility factor for tensile yield of anchor belt

$$\mu_{pa} = \frac{\pi._a q_y R}{K_{yA2}{}^2 (W_s + W_r + W_2) g} \left(\frac{2\pi}{T}\right)^2 \frac{0.617 t_b S_{yb}{}^2}{E_a P_b} \tag{8.50}$$

Provided $0.75 \leq \mu_{pa} \leq 2.5$

t_b: thickness of annular plate (mm)

S_{yb}: yield stress of annular plate (N/mm^2)

E_a: longitudinal modulus of elasticity of anchor belt (N/mm^2)

P_b: pressure of annular plate (MPa)

T: tank's natural period (Sec)

a_{qy}: yield strength of anchor belt per width unit (N/mm^2)

$${}_a q_y = \frac{NAS_{ya} - \pi R^2 P_0 \min}{2\pi R} \tag{8.51}$$

R: tank's lateral shell radius (mm)

S_{ya}: yield stress of anchor belt (N/mm^2)

8-3-6-Acceptance criteria

For each damage mode the equation (8.52) should be satisfied.

$$\mu_p \leq \mu_{pa} \tag{8.52}$$

μ_p: ductility factor

μ_{pa}: allowable ductility factor

Chapter 9

Seismic Design and Safety Control of Tower and Vertical Vessel

9-1-Seismic design procedure of tower and vertical vessel

Towers with height, H_t, equal to 5m or more (distance between tangents) are the target facilities for seismic design of this chapter.

9-2-Allowable stress design method

9-2-1-Response analysis

The seismic performance is evaluated, based size and natural period of tower, using the pseudo-static method, modified pseudo-static method, and modal analysis methods. When risk level-1 is in use, the allowable stress method, and when risk level-2 is in use, the ductility method is applied.

9-2-1-1-Pseudo-static method

The pseudo-static method could be applied to structures with medium or low importance factor, and height lower than 20m (height from the lower side of base plate to highest contact point of skirt support and leg support and for ring supports, the distance between tangents) from the base for tower and horizontal tank. The total weight is assumed equal to dead weight of part receiving earthquake and weight of contents.

9-2-1-2-Modified pseudo-static method

This is a method to determine the seismic force based on the natural period. For towers with skirt support, in which the ratio of mean diameter (D_m) to height from bottom (H_t) is lower than 4, the magnification factor of horizontal response (β_4) could be considered equal to 2.

If the natural period of tower is larger than the value presented in table (9.1), then the response analysis should be carried out based on the next modal analysis.

Table 9.1: Natural period range

Type of Ground	Natural Period (Sec)
Type 1	0.5
Type 2 and 3	1
Type 4	1.5

1-Natural period of towers independent from skirt support

In cases where the height of tower is equal or larger than 4 times of mean diameter, provided the ratio of maximum to minimum internal diameter is lower than 2 and the ratio of upper to lower shell thickness if larger than 0.5 and lower than 4, then the tower's natural period could be obtained using equation (9.1).

$$T = \frac{CH_t}{\sqrt{K_{MH} D_m}} \quad (9.1)$$

T: natural period (sec)

K_{MH}: modified seismic factor of design

D_m: mean diameter of shell (m)

C: assumed about 0.03 for towers

H_t: height from tower bottom (m)

2-Natural period of towers with leg supports

$$T = 2\pi\sqrt{\frac{W_0}{Kg}} \qquad (9.2)$$

T: natural period (sec)

g: gravity acceleration (mm/s^2)

W_0: operational weight (N)

K: horizontal rigidity of equipment (N/mm) obtained from equation (9.3)

$$K = \frac{1}{\frac{\lambda}{K_1} + \frac{1}{K_2}} \qquad (9.3)$$

K_1: total cyclic rigidity obtained from equation (9.4) (N/mm)

$$K_1 = \frac{3nEA_1D_1^{\,2}}{2H_1^{\,3}} \qquad (9.4)$$

K_2: total shear rigidity obtained from equation (9.5) (N/mm)

$$K_2 = \frac{nK_c}{1 + \frac{H_1K_c}{GA_1}} \qquad (9.5)$$

K_c: bending rigidity of each leg obtained from equation (9.6) (N/mm)

$$K_c = \frac{4E(I_1 + I_2)}{H_1^{\,3}} \qquad (9.6)$$

λ: modified factor which is obtained from equation (9.7) based on height of gravitational center:

$$\lambda = \left(\frac{H_2}{H_1}\right)^2 - \frac{H_2}{H_1} + 4 \qquad (9.7)$$

Where, in above equations, the notations are as follows.

H_1: height from lower side of bottom sheet to installation welding center (mm)

H_2: height from lower side of bottom sheet to tower's center of gravity (mm)

n: number of legs

E: longitudinal modulus of elasticity of leg materials (N/mm^2)

D_1: diameter of created circle by center of legs (mm)

G: shear transverse modulus of elasticity of leg materials (N/mm^2)

A_1: section area of leg (mm^2)

I_1: second moment of area relative to leg's axis tangential to tank's perimeter (mm^4)

I_2: second moment of area relative to leg's axis in the radial direction of section (mm^4)

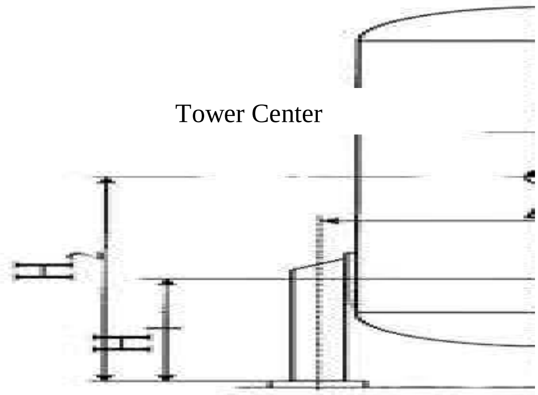


Figure 9.1: Towers with leg supports

9-2-1-3-Modal analysis

It is necessary for modal analysis to calculate the natural period of each tower and tank (vessel) according to various support conditions, including skirt, leg, and ring, for each mode.

9-2-2-Stress measurement

The stress measurement method is as following based on type of support structure:

1-Towers with skirt and ring support

1-1-created stress in shell (body)

a) Tensile stress

$$\sigma_t = \left(\frac{P_o D_m}{4t} - \frac{W_V - F_v}{\pi D_m t} + \frac{4M}{\pi D_m{}^2 t} \right) \frac{1}{\cos\theta} \tag{9.8}$$

σ_t: created tensile stress in shell (N/mm^2)

t: thickness of shell considering the allowable corrosion amount (mm)

P_o: normal operation pressure (MPa)

D_m: mean diameter (mm)

W_V: sum of structure weight and contents' weight (N)

F_V: design's vertical seismic force (N)

M: sum of created moments in modified horizontal seismic factor (if there is eccentric loads, then the moment of load should be considered) (N.mm)

θ: ½ angle of cone apex (deg)

b) Compressive stress

$$\sigma_c = \left(\frac{P_o D_m}{4t} - \frac{W_V - F_v}{\pi D_m t} + \frac{4M}{\pi D_m{}^2 t} \right) \frac{1}{\cos\theta} \tag{9.9}$$

σ_c: created compressive stress in shell (N/mm^2)

1-2-Stress created in skirt support

$$\sigma_c = \left\{ \frac{W_V + F_V}{(\pi D_m - Y_s)t} + \frac{4M}{(\pi D_m{}^2 - 2D_m Y_s)t} \right\} \frac{1}{\cos\theta} \tag{9.10}$$

σ_c: created compressive stress at skirt support (N/mm^2)

D_m: mean diameter (mm)

Y_s: maximum horizontal length of skirt opening (mm)

t: thickness of skirt plate (mm)

θ: ½ angle of cone-shaped skirt apex (deg)

1-3-Stress created in anchor-bolt

$$\sigma_t = \frac{1}{n_{ab} A_b}\left(-W_V + F_V + \frac{4M}{D_1}\right) \quad (9.11)$$

σ_t: created tensile stress in anchor-bolt (N/mm^2)

n_{ab}: number of anchor-bolts

A_b: effective section area of anchor bolt (mm^2)

D_1: area of created circle by center of anchor-bolts (mm)

1-4-Created stress in bottom sheet

a) Non-block type base

$$\sigma_b = \frac{3{L_m}^2}{t^2}\left(\frac{W_V + F_V}{A_{bp}} + \frac{M}{Z}\right) \quad (9.12)$$

σ_b: created bending stress in bottom sheet (N/mm^2)

t: thickness of sheet (mm)

L_m: maximum value between l1 and l2 shown in figure (9.2) (mm)

A_{bp}: area of bottom sheet (mm^2)

Z: sectional modulus relative to radial direction of bottom sheet (mm^3)

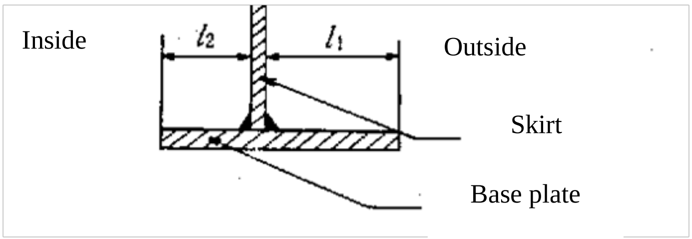


Figure 9.2: Base plate

b) Block type base

The stress measurement is similar to section (a). The created bending stress in bottom plate could be calculated by considering the created stress in block base based on structure of block base.

2-Stress measurement of towers with leg support

2-1-Created stress in shell

The calculation procedure is similar to (1.1). In this case, the vertical cylindrical storage tank with length lower than 5m is assumed as rigid between the highest and lowest contact points of legs and shell tangential line.

2-2-Stress created in leg

a) Tensile stress:

$$\sigma_t = \frac{1}{n_1 A_1}(-W_0 + F_V + \frac{4F_H H_2}{D_1}) \quad (9.13)$$

σ_t: tensile stress created in leg (N/mm^2)

n_1: number of legs

A_1: section area of leg (mm^2)

W_V: operational weight (N)

F_V: design's vertical seismic load (N)

D_1: diameter of circle created by center of legs (mm)

H_2: height from bottom sheet to mass center of towers (mm)

b)Compressive stress:

$$\sigma_t = \frac{1}{n_1 A_1}(W_V + F_V + \frac{4F_H H_2}{D_1}) \quad (9.14)$$

σ_c: compressive stress created in leg (N/mm^2)

c) Bending stress:

$$\sigma_b = \frac{1.2F_H H_1 e_f}{n_1(I_1 + I_2)} \quad (9.15)$$

σ_b: bending stress created in leg (N/mm^2)

I_1: inertia moment of area relative to leg's axis in the tangential direction of shell cross-section (mm^4)

I_2: inertia moment of area relative to leg's axis in the radial direction of shell cross-section (mm^4)

e_f: maximum distance from neutral axis of leg to arc part of tower shell (mm)

d) Shear stress:

$$\tau = \frac{F_H}{n_1 A_1} \quad (9.16)$$

τ: shear stress created in leg (N/mm^2)

2-3-Stress created in anchor-bolt

This stress is presented by equation (9.17) based on pullout force, P_b, imposed on base plate.

$$P_b = -W_v + F_V + \frac{4F_H H_2}{D_1} \text{ (N)} \quad (9.17)$$

a) Tensile stress:

For $P_b \geq 0$

$$\sigma_t = \frac{e_f + \frac{a}{2} - c}{a - 2c} \cdot \frac{2P_b}{n_a A_b} \quad (9.18)$$

For $P_b < 0$

$$\sigma_t = \frac{0.48\left(e_f + \frac{a}{6} + \frac{c}{3}\right)}{a - c}\left\{\frac{15}{b(a-c)} + \frac{2}{n_a A_b}\right\} P_b \tag{9.19}$$

σ_t: tensile stress created in anchor-bolt, if it is negative, then it would be assumed zero.

a, b, and c: values which are shown in figure (9.3) (mm)

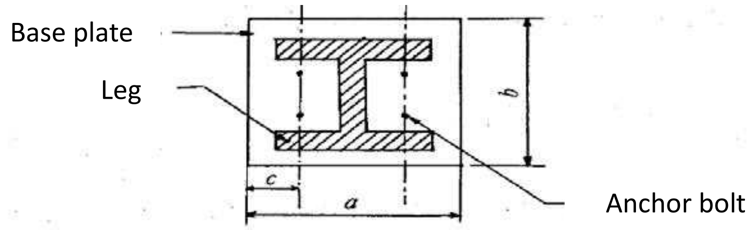


Figure 9.3

n_a: number of anchor-bolts in each leg

e_f: value obtained from equation (9.20) (mm)

$$e_f = \frac{1}{P_b} \cdot \frac{1.2 I_1}{I_1 + I_2} \cdot F_H H_1 \tag{9.20}$$

b) Shear stress

$$\tau = \frac{F_H}{n_1 n_a A_b} \tag{9.21}$$

τ: shear stress created on anchor-bolt (N/mm^2)

3-Calculated stress of towers with ring (annular) support

3-1-Stress created in shell plate: it could be measured for shell using the equation mentioned in section for towers with skirt support.

3-2-Stress created in bolt: it could be measured for bolt using the equation mentioned in section for towers with skirt support.

Location of stress calculation in towers and vertical vessels is shown in table (9.2).

Table 9.2: Positions for stress measurement and stress type

Tower Type	Part which its stress is being determined	Type of Stress				
		Tension	Shear	Bending	Compressive	Buckling
Skirt Support	Body	○			○	○
	Shell				○	○
	Anchor-bolt	○				
	Base plate			○		
Leg support	Body	○			○	○
	Leg	○	○	○	○	○
	Anchor-bolt	○	○			
Ring support	Body	○			○	○
	Bolts	○				

Positions for stress measurement in towers is limited to shell, skirt, leg, anchor-bolt, base plate, and other important parts of structures such as support system, but this does not mean that other parts of structure do not need seismic design.

If the importance factor is medium and low, then the evaluation of vertical seismic force could be only carried out for compressive stress.

9-2-3-Allowable stress

In this method, the designing is performed based on stresses obtained in compressive part materials and support structure and considering the allowable stresses.

The allowable stresses of materials are presented in section 4.3.

9-2-4-Acceptance criteria

If the calculated stress is smaller than the allowable stress, the seismic performance for earthquake is confirmed.

9-3-Ductility method designing

9-3-1-Towers with shell support

9-3-1-1-Damage modes

1-Shell failure
- -Shell tensile yield
- -Shell compressive buckling

2-Skirt failure
- -Skirt compressive buckling

3-Anchor-bolt failure
- -Anchor-bolt tensile yield

4-Base plate yield
- -Bending yield of base plate

9-3-1-2-Yield earthquake coefficient

The yield earthquake coefficient should be calculated for each damage mode.

1-Yield earthquake coefficient related to shell yielding mode

1-1-Yield earthquake coefficient related to shell tensile yield

$$K_{yts} = K_{MH} \frac{S_y - \sigma_{tOs}}{\sigma_{tHs} + \sigma_{tVs}} \tag{9.22}$$

K_{yts}: yield earthquake coefficient for shell's tensile yield in stress calculation position

K_{MH}: design's modified seismic factor of structure

σ_{tOs}: tensile stress created in shell during normal operation obtained from equation (9.23) (N/mm^2)

$$\sigma_{tOs} = \left(\frac{P_o D_m}{4t} - \frac{W_v}{\pi D_m t} \right) \frac{1}{\cos\theta} \tag{9.23}$$

σ_{tHs}: tensile stress created in shell caused by design's horizontal force obtained from equation

(9.24) (N/mm^2)

$$\sigma_{tHs} = \frac{4M}{\pi D_m{}^2 t} \cdot \frac{1}{\cos\theta} \tag{9.24}$$

σ_{tVs}: tensile stress created in shell caused by design's vertical force obtained from equation (9.25) (N/mm^2)

$$\sigma_{tVs} = \frac{F_V}{\pi D_m t} \cdot \frac{1}{\cos\theta} \tag{9.25}$$

M: total moments imposed on position of modified horizontal factor (if the load is eccentric, then the moments should be considered) (N.mm)
D_m: mean diameter (mm)
t: thickness of shell (considering the allowable corrosion amount, mm)
P_0: normal operation pressure (MPa)
θ: ½ angle of cone apex (deg)
W_V: total weight of structure and its contents in seismic design position (N)
F_V: vertical seismic force obtained from equation (9.26)

$$F_V = K_{MV} W_V \tag{9.26}$$

KMV: modified vertical seismic factor
S_y minimum value between yield strength at design temperature or materials' normal temperature and strength equivalent to 0.2% strain of steel (N/mm^2)

1-2-Yield earthquake coefficient related to shell compressive buckling

$$K_{ycs} = K_{MH} \frac{S_c - \sigma_{co}}{\sigma_{cH} + \sigma_{cV}} \tag{9.27}$$

K_{ycs}: yield earthquake coefficient for shell compressive buckling at position of stress measurement
σ_{co}: compressive stress created in shell due to normal operation load obtained from equation (9.28) (N/mm^2)

$$\sigma_{co} = \left(-\frac{P_o D_m}{4t} + \frac{W_v}{\pi D_m t} \right) \frac{1}{\cos\theta} \tag{9.28}$$

σ_{cH}: compressive stress created in shell caused by design's horizontal force obtained from equation (9.29) (N/mm^2)

$$\sigma_{cH} = \frac{4M}{\pi D_{m^2} t} \cdot \frac{1}{\cos\theta} \tag{9.29}$$

σ_{cV}: compressive stress created in shell caused by design's vertical seismic force obtained from equation (9.30) (N/mm^2)

$$\sigma_{cV} = \frac{F_v}{\pi D_m t} \cdot \frac{1}{\cos\theta} \tag{9.30}$$

P_o: minimum pressure during normal operation (MPa)

$$S_c = \min\left(S_y, S_f S'\right) \tag{9.31}$$

S': value obtained from equation (9.32)

$$S' = \frac{0.6Et}{\left(1 + 0.004\dfrac{E}{S_y}\right)D_m} \tag{9.32}$$

E: longitudinal modulus of elasticity at normal temperature (N/mm^2)

S_f: 1.5

2-Yield earthquake coefficient related to skirt yield mode

2-1-Yield earthquake coefficient related to skirt compressive buckling

$$K_{yck} = K_{MH}\frac{S_c - \sigma_{cO}}{\sigma_{cH} + \sigma_{cV}} \tag{9.33}$$

K_{yck}: yield earthquake coefficient for shell compressive buckling at stress measurement position

σc0: compressive stress created in skirt support due to normal operation load obtained from equation (9.34) (N/mm^2)

$$\sigma_{cO} = \frac{W}{(\pi D_m - Y_s)t} \cdot \frac{1}{\cos\theta} \tag{9.34}$$

σ_{CV}: compressive stress created in shell caused by design's vertical seismic force obtained from equation (9.35) (N/mm^2)

$$\sigma_{CV} = \frac{F_v}{(\pi D_m - Y_s)t} \cdot \frac{1}{\cos\theta} \tag{9.35}$$

σ_{CH}: compressive stress created in shell caused by design's horizontal seismic load obtained from equation (9.36) (N/mm^2)

$$\sigma_{CH} = \frac{4M}{(\pi D_m - 2D_m Y_s)t} \cdot \frac{1}{\cos\theta} \tag{9.36}$$

Y_s: maximum horizontal length of shell opening (mm)

t: thickness of plate at the desired position (mm)

θ: ½ angle of cone-shaped skirt apex at the desired position (deg)

3-Yield earthquake coefficient related to yield mode of anchor-bolt

3-1-Yield earthquake coefficient related to tensile yield of anchor-bolt

$$K_{ytB} = K_{MH}\frac{S_y + \sigma_{tOb}}{\sigma_{tHb} + \sigma_{tVb}} \tag{9.37}$$

K_{ytB}: yield earthquake coefficient related to tensile yield of anchor-bolt

σ_{tOb}: stress created in anchor-bolt due to normal load obtained from equation (9.38) (N/mm^2)

$$\sigma_{tOb} = \frac{W_V}{NA_1} \tag{9.38}$$

σ_{tHb}: stress created in anchor-bolt by horizontal seismic force obtained from equation (9.39) (N/mm^2)

$$\sigma_{tHb} = \frac{1}{NA_1} \cdot \frac{4M}{D_1} \tag{9.39}$$

σ_{tVb}: stress created in anchor-bolt by vertical seismic force obtained from equation (9.40) (N/mm^2)

$$\sigma_{tVb} = \frac{F_v}{NA_1} \quad (9.40)$$

N: number of anchor-bolts

D_I: diameter of created circle by center of anchor-bolts (mm)

S_y: yield strength of anchor-bolt materials (N/mm^2)

4-Yield earthquake coefficient related to yield mode of base plate

4-1-Yield earthquake coefficient related to bending yield of base plate for non-block type

$$K_{ybb} = K_{MH}\frac{S_y - \sigma_{bO}}{\sigma_{bH} + \sigma_{bV}} \quad (4.91)$$

K_{ybb}: yield earthquake coefficient related to bending yield of base plate

σ_{tbO}: bending stress created in base plate by normal load obtained from (9.42) (N/mm^2)

$$\sigma_{bO} = \frac{3L^2}{t^2}.\frac{W_V}{A_b} \quad (9.42)$$

σ_{bH}: bending stress created in base plate by horizontal seismic force obtained from equation (9.43) (N/mm^2)

$$\sigma_{bH} = \frac{3L^2}{t^2}.\frac{M}{Z} \quad (9.43)$$

σ_{bV}: bending stress created in base plate by vertical seismic force obtained from equation (9.44) (N/mm^2)

$$\sigma_{bV} = \frac{3L^2}{t^2}.\frac{F_V}{A_b} \quad (9.44)$$

t: thickness of plate (mm)

L: maximum value of l1 and l2 shown in figure (9.4) (mm)

A_b: area of base plate (mm^2)

Z: section modulus relative to radial direction of base plate (mm^3)

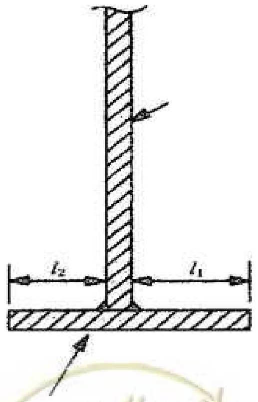


Figure 9.4: Base plate

4-2-Yield earthquake coefficient related to bending yield of base plate for block type

The calculation equation from section (1.1) could be applied. The bending yield earthquake coefficient of base plate could be calculated considering the stress created in block base based on block base structure.

When believed that the allowable stress method is suitable, if the measured stress is equal to yield stress or compressive buckling stress, the seismic factor would be considered as the yield earthquake coefficient.

The yield earthquake coefficient is obtained by replacing the measured stress by elastic response related to each failure mode using the determined seismic factor in evaluation of design via the allowable stress method, instead of yield stress.

In such cases, equation (9.45) is used for simplification.

$$\frac{\text{Horizontal yield coefficient, } K_{yH}}{\text{Modified horizontal seismic factor, } K_{MH}} = \frac{\text{Vertical yield coefficient, } K_{yV}}{\text{Modified vertical seismic factor, } K_{MV}} \quad (9.45)$$

After calculating the structure's natural period, the modified vertical and horizontal seismic factors could be obtained. The yield earthquake coefficient is obtained using the above-mentioned equation for each damage mode.

9-3-1-3-Response ductility factor

The response ductility factor is obtained from design's modified horizontal seismic response and yield seismic coefficient. It is calculated from equation (9.46) for each damage mode of skirt support.

$$\mu_p = \frac{1}{4C}\left\{\left(\frac{K_{MH}}{K_y}\right)^2 - 1\right\} \quad (9.46)$$

μ_p: response ductility factor for each damage mode, if $K_{MH} \le K_y$ then $\mu_p = 0$

K_{MH}: modified horizontal seismic factor of design for the desired structure under seismic design.

K_y: yield earthquake coefficient related to yield mode

C: the following values could be used for yield mode of skirt support.

1-Shell damage mode

-Damage caused by shell's tensile stress C = 2.0

-Yield caused by shell's compressive buckling C = 2.0

2-Skirt support damage

-Damage caused by compressive buckling of skirt support C = 2.0

3-Anchor-bolt damage

-Damage caused by tensile yield of anchor-bolt C = 1.0

4-Seat's sheet damage (base plate)

-Damage caused by bending yield of seat's sheet C = 2.0

In towers with skirt support, C = 1 for seat's sheet yield with slip anchor-bolts, and C = 2 for other cases.

9-3-1-4-Allowable ductility factor

The allowable ductility factor for damage mode of tower with skirt support is as following.

1-Shell damage

-Shell's tensile damage $\mu_{pa} = 1.0$

-Shell's compressive buckling μ_{pa} = 0.35

2-Skirt support damage

-Skirt compressive buckling μ_{pa} = 0.35

3-Anchor-bolt damage

-Tensile yield of anchor-bolt μ_{pa} = 1.8

4-Base plate damage

-Bending yield of base plate μ_{pa} = 0.35

The allowable ductility factor could be obtained assuming structural characteristics D_S = 0.5 for shell buckling, D_S = 0.35 for anchor-bolt, D_S = 0.35 for shell bending, and D_S = 0.5 for plate bending.

9-3-1-5-Acceptance criteria

If the calculated response ductility factor is smaller than the allowable ductility factor, the seismic performance of structure would be suitable.

9-3-2-Towers with leg and ring supports

9-3-2-1Damage mode

1-Towers with leg support

The seismic performance of towers with leg support should be examined for the following damage modes.

1-1-Shell plate failure

a) Tensile yield of shell plate

b) Compressive buckling of shell plate

1-2-leg failure

a) Leg tensile yield

b) Leg compressive buckling

c) Leg bending yield

d) Leg shear yield

e) Yielding due to combination of above stresses in leg

1-3-Bracing rod failure

a)Bracing rod tensile yield

b) Bracing rod shear yield

c) Yielding due to combination of tensile and shear stresses of bracing rod

1-4-Leg's attachment part failure

a) Bending yield of leg's attachment part

2-Tower with ring support

The seismic performance of towers with ring support should be examined for the following damage modes.

2-1-Shell plate failure
- a) Tensile yield of shell plate
- b) Compressive buckling of shell plate

2-2-Bolts failure
- a) Tensile yield of bolts

The seismic design procedure using ductility design method for towers with leg and ring supports is presented in figure (9.5).

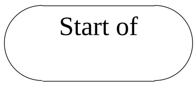


Figure 9.5: Seismic design procedure using ductility method for towers with leg and ring supports

9-3-2-2-Yield earthquake coefficient of towers with leg support

9-3-2-2-1Yield earthquake coefficient in damage mode of shell plate

1-Tensile yield earthquake coefficient of shell

$$K_{ytS} = K_{MH}\frac{S_y + \sigma_O}{\sigma_{EL}} \quad (9.47)$$

K_{MH}: modified lateral seismic factor

K_{ytS}: tensile yield seismic factor of shell in designing conditions

σ_O: operation stress obtained from equation (9.48)

σ_{EL}: seismic stress caused by K_{MV} and K_{MH} obtained from equation (9.49)

$$\sigma_O = \left(\frac{W_v}{\pi D_m t} - \frac{P_o D_m}{4t}\right)\frac{1}{\cos\theta} \quad (9.48)$$

$$\sigma_{EL} = \left(\frac{F_v}{\pi D_m t} - \frac{4M}{\pi D_m{}^2 t}\right)\frac{1}{\cos\theta} \quad (9.49)$$

M: total moments imposed at design position for modified lateral seismic factor, K_{MH} (if the load is eccentric, then its moment is considered) (N.mm)

D_m: mean diameter at designing position (mm)

P_0: normal pressure (MPa)

θ: ½ angle of cone apex at designing conditions (deg)

W_V: total of dead weight and structure's contents imposed on the position under-study (N)

F_V: vertical seismic force obtained from equation (9.50)

$$F_V = K_{MV} W_V \quad (9.50)$$

K_{MV}: modified vertical seismic factor

t: thickness of plate at designing position (without considering the allowable corrosion amount, mm)

S_y: minimum value between tensile yield strength at designing temperature or normal temperature and strength equivalent to 0.2% strain of steel (N/mm^2)

2-Yield earthquake coefficient in shell compressive buckling

$$K_{ycs} = K_{MH}\frac{S_c - \sigma_O}{\sigma_{EL}} \quad (9.51)$$

K_{ycs}: yield earthquake coefficient of shell's compressive buckling at designing position

$$S_c = \min\left(S_y, S_f S'\right) \quad (9.52)$$

S': value obtained from equation (9.53) at designing position (N/mm^2)

$$S' = \frac{0.6Et}{\left(1 + 0.004\frac{E}{S_y}\right)D_m} \quad (9.53)$$

E: longitudinal modulus of elasticity at materials' designing temperature (N/mm^2)

S_f: 1.5

9-3-2-2-2Yield earthquake coefficient of leg's damage mode

1-Tensile yield seismic coefficient of leg

$$K_{ytL} = K_{MH} \frac{F + \sigma_{LO}}{\sigma_{aLE}} \quad (9.54)$$

K_{MH}: modified lateral seismic factor

K_{ytL}: tensile yield earthquake coefficient of leg at designing position

F: yield stress of leg (N/mm^2)

σ_{LO}, σ_{aLE}: value obtained from equation (9.55)

$$\sigma_{LO} = \frac{W_V}{n A_1} \quad (9.55)$$

$$\sigma_{aLE} = \frac{1}{nA_1}\left(F_v + \frac{4F_H H_2}{D_1}\right) \quad (9.56)$$

n: number of legs

A_1: section area of leg (mm^2)

F_V: design's vertical seismic force and value obtained from equation (9.57) (N)

$$F_v = K_{MV} W_V \quad (9.57)$$

FH: design's horizontal seismic force (N)

2-Leg's compressive buckling yield earthquake coefficient

$$K_{ycL} = K_{MH} \frac{F' - \sigma_{LO}}{\sigma_{aLE}} \quad (9.58)$$

K_{ycL}: leg's compressive buckling yield earthquake coefficient at designing position

F': leg's buckling stress (N/mm^2)

3-Bending yield earthquake coefficient of leg

$$K_{ybL} = K_{MH} \frac{F}{\sigma_{bLE}} \quad (9.59)$$

K_{ybL}: leg's bending yield earthquake coefficient at designing position

σ_{bLE}: value obtained from (9.60)

$$\sigma_{bLE} = \frac{1.2 F_H H_1 e_t}{n_1 (I_1 + I_2)} \quad (9.60)$$

I_1: inertia moment relative to circumferential direction of leg (mm^4)

I_2: inertia moment relative to radial direction of leg (mm^4)

H_1: length of leg (mm)

e_t: maximum length from leg's neutral axis to arc part of tower's shell

4-Shear yield earthquake coefficient of leg

$$K_{ysL} = K_{MH} \frac{F/\sqrt{3}}{\sigma_{sLE}} \quad (9.61)$$

K_{ysL}: shear yield earthquake coefficient of leg at designing position

σ_{sLE}: value obtained from equation (9.62)

$$\sigma_{sLE} = \frac{F_H}{n_1 A} \quad (9.62)$$

A, n1, FH: similar to previous definitions

5-Stress combination yield earthquake coefficient of leg

a)Yield earthquake coefficient for combination of compressive and bending combination from equation (9.63)

$$K_{ycbL} = K_{MH}\frac{1-\frac{\sigma_{LO}}{F'}}{\frac{\sigma_{aLE}}{F'}+\frac{\sigma_{bLE}}{F}} \tag{9.63}$$

b)Yield earthquake coefficient for combination of tensile and bending stresses from equation

$$K_{ytbL} = K_{MH}\frac{F+\sigma_{LO}}{\sigma_{LE}} \tag{9.64}$$

σ_{LE}: value obtained from equation (9.65)

c)Yield earthquake coefficient for combination of compressive, bending, and shear stresses

K_{ycbL}: yield earthquake coefficient for compressive and bending stresses at leg's designing position

K_{ytbL}: yield earthquake coefficient for tensile and bending stresses at leg's designing position

K_{ymL}: yield earthquake coefficient for combination of compressive, bending, and shear stresses at leg's designing position

9-3-2-2-3-Yield earthquake coefficient in damage mode of anchor-bolt

1-Tensile yield earthquake coefficient of anchor-bolt

$$K_{ytB} = K_{MH}\frac{F+\sigma_{BO}}{\sigma_{tBE}} \tag{9.67}$$

K_{ytB}: Tensile yield earthquake coefficient of anchor-bolt at designing position

K_{MH}: design's modified lateral seismic factor

F: yield stress of bolt (N/mm^2)

σ_{BO}: stress of anchor-bolt due to operation weight obtained from equation (9.68)

$$\sigma_{BO} = j_B W_V \tag{9.68}$$

σ_{tBE}: overturn stress of anchor-bolt due to design's seismic force

$$\sigma_{tBE} = j_B\left(F_V + \frac{4F_H H_2}{D}\right) \tag{9.69}$$

j_B: from equations (9.83) and (9.84) and based on P_b from equation (9.17)

If $P_b \geq 0$:

$$j_B = \frac{e_d + \frac{a}{2} - c}{a-2c}\frac{2}{n_a A_b} \tag{9.70}$$

If $P_b < 0$

$$j_B = \frac{1}{n}\frac{0.48\left(e_d + \frac{a}{6}+\frac{c}{3}\right)}{a-c}\left\{\frac{15}{b(a-c)}+\frac{2}{n_a A_b}\right\} \tag{9.71}$$

a, b, c: values shown in figure (9.6) (mm)

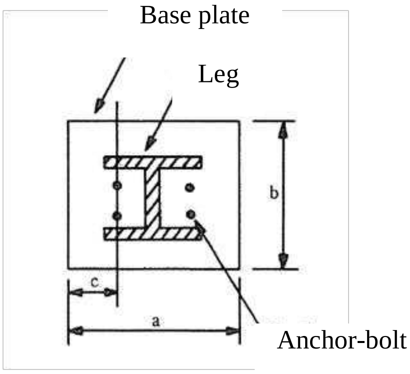


Figure 9.6: Leg's cross-section

n_a: number of anchor-bolts in each leg

e_d: value obtained from equation (9.72) (mm)

$$j_B = \frac{1}{P} \cdot \frac{1.2 I_1}{I_1 + I_2} . F_H H_1 \quad (9.72)$$

2-Shear yield earthquake coefficient of bracing rod

$$K_{ysB} = K_{MH} \frac{F/\sqrt{3}}{\sigma_{sBE}} \quad (9.73)$$

K_{ysB}: shear yield earthquake coefficient of bracing rod at designing position

σ_{sBE}: value obtained from equation (9.74)

$$\sigma_{sBE} = \frac{F_H}{n n_a A_b} \quad (9.74)$$

3-Tensile and shear stresses combination yield earthquake coefficient of bracing rod

$$K_{ytsB} = K_{MH} \frac{1.4F + \sigma_{BO}}{\sigma_{tBE} + 1.6\sigma_{sBE}} \quad (9.75)$$

K_{ytsB}: stress combination yield earthquake coefficient of bracing rod

9-3-2-2-4-Yield earthquake coefficient in damage mode of leg attachments

1-

a)Leg attached to cylindrical part

The bending yield earthquake coefficient of leg attachments is a value obtained from equation (9.76).

$$K_{ybc} = \frac{Q_{ybc}}{W_H} \quad (9.76)$$

K_{ybc}: bending yield earthquake coefficient of leg attachments

W_H: operational weight (N)

Q_{ybc}: yield strength determined by bending yield of shell panel of leg's attachment, and is obtained from equation (9.77) (N)

$$Q_{ybc} = \frac{n\beta D_m t^2}{8 H_1 i_c} S_y \quad (9.77)$$

S_y: tensile yield strength of panel materials (N/mm^2)

β: value obtained from equation (9.78)

$$\beta = h_{sp} / D_m \tag{9.78}$$

h_{sp}: height of hardening plate (mm)
i_c: stress index shown in figure (9.7)
r: value obtained from equation (9.79)

$$r = 0.5 D_m t \tag{9.79}$$

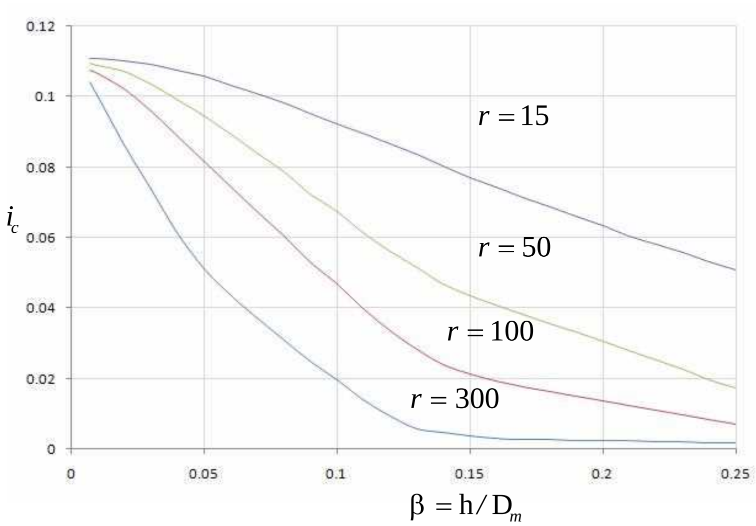


Figure 9.7: value of i_c

b)Leg attached to panel

The bending yield earthquake coefficient of leg is a value obtained from equation (9.80).

$$K_{ybH} = \frac{Q_{ybH}}{W_H} \tag{9.80}$$

K_{ybH}: leg's bending earthquake coefficient
Q_{ybH}: yield strength determined by bending yield of panel at external facilities of leg shell
Q_{ybH}: minimum value between Q_{y1} and Q_{y2}. This value is three times of minimum value of Q_{y1} and Q_{y2} in double-shell storage tanks (N)
Q_{y1}: panel's bending strength due to bending moment imposed on leg which is obtained from equation (9.81) (N).

$$Q_{y1} = \frac{nM_{LG}}{H_1} \tag{9.81}$$

M_{LG}: value obtained from equation (9.82)

$$M_{LG} = \frac{M_x \sqrt{R_m t_h}}{i_{s2}} \tag{9.82}$$

M_x: value obtained from equation (9.83)

$$M_x = 1.5\frac{t_h^2}{6}S_y \tag{9.83}$$

t_h: thickness of panel shown in figure (9.8) (mm)

R_m: value obtained from equation (9.84) (mm)

$$R_m = \frac{D_0}{8}\left[4-3\left(\frac{D}{D_0}\right)^2\right]^{3/2} \tag{9.84}$$

D: leg's central diameter shown in figure (9.8) (mm)

D_0: shell's external diameter shown in figure (9.8) (mm)

i_{S2}: stress index related to leg's bending moment, shown in figure (9.9)

Q_{y2}: bending yield strength of panel due to axial force imposed on leg obtained from equation (9.85) (N).

$$Q_{y2} = \frac{M_x nD\sqrt{4B^2+D^2}}{i_{s1}\left[8(H_2-H_1)B+D^2\right]} \tag{9.85}$$

i_{S1}: stress index related to leg's axial force obtained from figure (9.10)

B: value obtained from equation (9.86) (figure 9.8)

$$B = D_0\left[1-\left(\frac{D}{D_0}\right)^2\right]^{1/2} \tag{9.86}$$

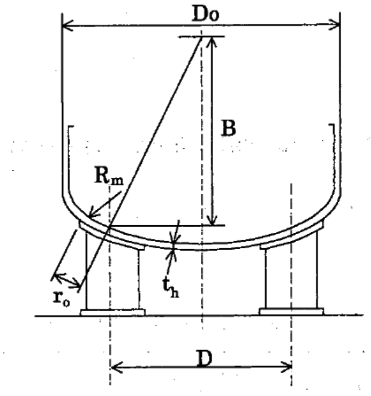


Figure 9.8: Tower with leg support

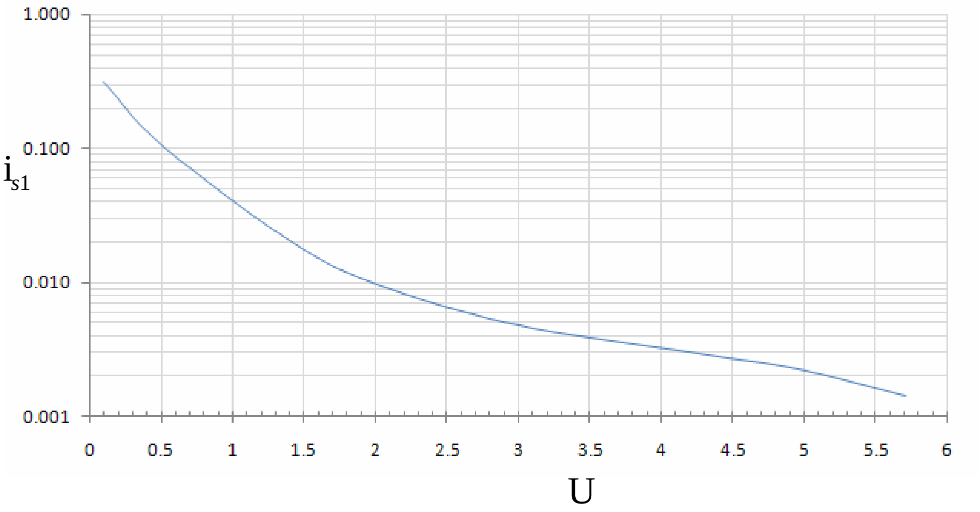


Figure 9.9: Determining index i_{S1}

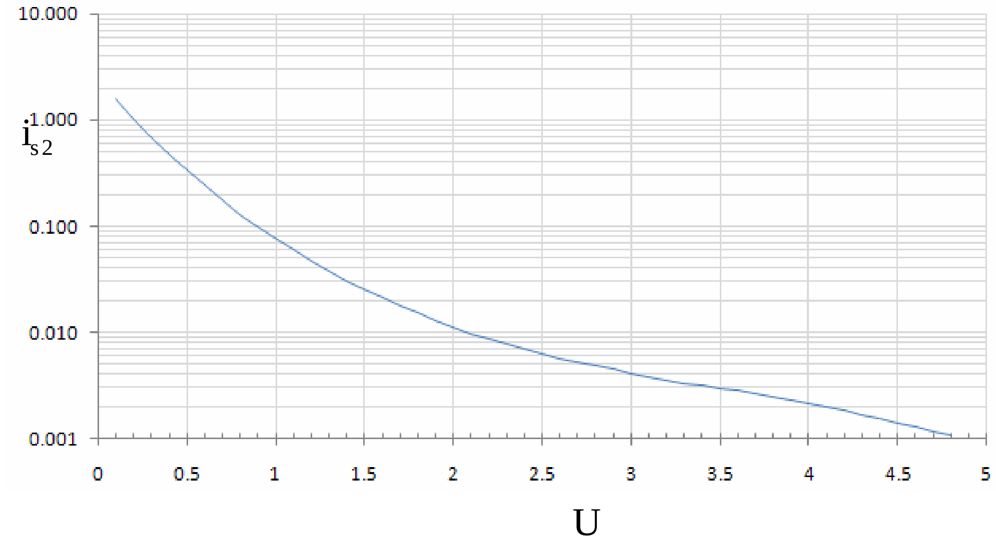


Figure 9.10: Determining index i_{S2}

U in figures (9.9) and (9.10) is a coefficient obtained from equation (9.87).

$$U = \frac{r_O}{\sqrt{R_m t}} \tag{9.87}$$

r_0: leg's corrosion plate radius (mm)

9-3-2-3-Yield earthquake coefficient of towers with ring support

9-3-2-3-1-Yield earthquake coefficient in damage mode of shell plate

1-Tensile yield earthquake coefficient of shell plate

It complies with rules related to tensile yield earthquake coefficient of shell plate in towers with skirt support.

2-Compressive buckling yield earthquake coefficient of shell plate

It complies with rules related to compressive buckling yield earthquake coefficient of shell plate in towers with shell support.

9-3-2-3-2-Yield earthquake coefficient in damage mode of bolt

It complies with rules related to yield earthquake coefficient of damage mode of anchor-bolt in towers with skirt support.

9-3-2-4-Response ductility factor

The response ductility factor is a value obtained from equation (9.88) for each damage modes of towers with leg or ring supports.

$$\mu_p = \frac{1}{4C}\left\{\left(\frac{K_{MH}}{K_Y}\right)^2 - 1\right\} \qquad (9.88)$$

μ_p: response ductility factor in damage mode, for cases in which $K_{MH} \leq K_y$, then $\mu_p = 0$.

K_{MH}: design's modified seismic factor

K_y: yield seismic factor in damage mode

C: is as following for each damage mode:

1-Towers with leg support

1-1-Shell plate damage

a) Damage due to tensile yield of shell plate C = 2.0

b) Damage due to compressive buckling of shell plate C = 2.0

1-2-Leg damage

a) Damage due to tensile yield of leg C = 2.0

b) Damage due to compressive buckling of leg C = 2.0

c) Damage due to bending yield of leg C = 2.0

d) Damage due to shear yield of leg C = 2.0

e) Damage due to combination of compressive and bending stresses of leg C = 2.0

f) Damage due to combination of tensile and bending stresses of leg C = 2.0

g) Damage due to combination of compressive, bending, and shear stresses of leg C = 2.0

1-3-Bracing rod damage

a) Damage due to tensile yield of bracing rod C = 1.0

b) Damage due to shear yield of bracing rod C = 2.0

c) Damage due to combination of tensile and shear stresses of bracing rod C = 1.0

1-4-Leg's attachments damage

a)Damage due to bending yield of leg's attachments C = 2.0

2-Towers with ring support

2-1-Shell plate damage

a) Damage due to tensile yield of shell plate C = 2.0

b) Damage due to compressive buckling of shell plate C = 2.0

2-2-Bolts head damage

a) Damage due to tensile yield of bolts C = 1.0

Among different damage modes of towers with leg and ring supports, C is assumed equal to 1 for tension of bracing rod and tension of bolts head (for combination of tensile and shear stresses is also equal to 1) and in other cases C = 2. ($D_s = 0.5$)

9-3-2-5-Allowable ductility factor

The allowable ductility factor, μ_{pa}, is as follows for different damage modes of towers with leg and ring supports:

1-Towers with leg support

1-1-Shell plate damage

a) Damage due to tensile yield of shell plate $\mu_{pa} = 1.0$

b) Damage due to compressive buckling of shell plate $\mu_{pa} = 0.35$

1-2-Leg damage

a) Damage due to tensile buckling of leg $\mu_{pa} = 1.0$

b) Damage due to compressive buckling of leg $\mu_{pa} = 0.35$

c) Damage due to bending yield of leg $\mu_{pa} = 1.0$

d) Damage due to shear yield of leg $\mu_{pa} = 1.0$

e) Damage due to combination of compressive and bending stresses of leg $\mu_{pa} = 0.35$

f) Damage due to combination of tensile and bending stresses of leg $\mu_{pa} = 1.0$

g) Damage due to combination of compressive, bending, and shear stresses of leg $\mu_{pa} = 0.35$

1-3-Bracing rod damage

a) Damage due to tensile yield of bracing rod $\mu_{pa} = 1.8$

b) Damage due to shear yield of bracing rod $\mu_{pa} = 1.0$

c) Damage due to combination of tensile and shear stresses of bracing rod $\mu_{pa} = 1.8$

1-4-Leg's attachments damage

a) Damage due to bending yield of leg's attachments $\mu_{pa} = 2.0$

2-Towers with ring support

2-1-Shell plate damage

a) Damage due to tensile yield of shell plate $\mu_{pa} = 1.0$

b) Damage due to compressive buckling of shell plate $\mu_{pa} = 0.35$

2-2-Bolts head damage

a) Damage due to tensile yield of bolts $\mu_{pa} = 1.8$

The Allowable ductility factor is determined by assuming the structural characteristics factor, $D_s = 0.5$ for shell buckling and $D_s = 0.35$ for tensile yield of anchor-bolt and bolt-head. The structural characteristics factor is assumed $D_s = 0.35$ for shell's bending tensile yield

9-3-2-6-Criteria

Equation (9.89) should be satisfied for each damage modes of towers with leg and ring supports.

$$\mu_p \le \mu_{pa} \tag{9.89}$$

μ_p: response ductility factor in each damage mode
μ_{pa}: allowable ductility factor in each damage mode

Chapter 10

Seismic Design and Safety Control of Pseudo-Building Structures

10-1-Cover structures of equipments

10-1-1-Seismic design methods for cover and frame structures

The cover and frame structures of refinery equipments are designed using the allowable stress method or the ductility method based on hazard level.

10-1-2-Allowable stress method

In designing of cover and frame structures using the allowable stress method, the obtained stress in member should not exceed the allowable stress.

10-1-2-1-Calculating horizontal seismic force imposed on cover and frame structures

The response analysis is carried out using the pseudo-static method or the modified pseudo-static method, and the imposed design seismic force is calculated.

1-Natural period of frame structure of towers and tanks

Ratio of frame structure's weight = operation weight of tower and tanks/total weight (10.1)

1-1-Maximum ratio of frame structure's weight: 0.1 or less

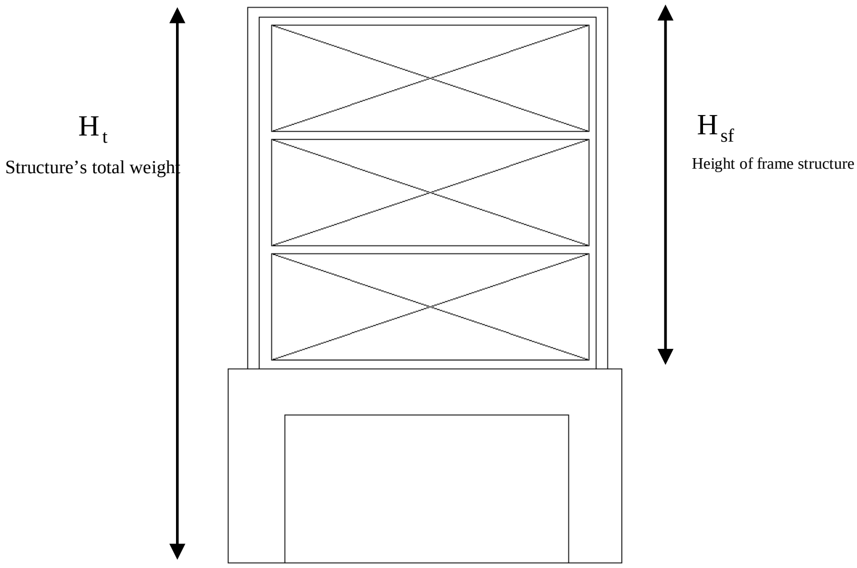


Figure 10.1: H_{sf} and H_t

$$T_{fs} = 0.01H_{sf} + 0.02H_t \tag{10.2}$$

T_{fs}: natural period

H_{sf}: height of metallic part of frame structure (m)

H_t: height of frame structure (m)

1-2-Maximum ratio of frame structure's weight: more than 0.1

$$T_{fs} = 0.057\sqrt{n} \tag{10.3}$$

η: maximum displacement of frame structure in case the frame structure's weight is used horizontally. In this case, tower or tanks are assumed as rigid. (mm)

2-Vertical and horizontal modified seismic force

2-1-Tower and tank held by frame structures

$$F_{MH} = \beta_7 \mu K_{MH} W_H \tag{10.4}$$

$$F_{MV} = K_{MV} W_V \tag{10.5}$$

F_{MH}: modified horizontal seismic force

F_{MV}: modified vertical seismic force

W_H: operational weight

W_V: total weight of contents and structure held by frame in position where the vertical seismic force is being measured

β_7: response magnification factor of frame structure for towers and tanks, obtained from equation (10.6).

λ is a value which could be obtained from the calculative equations of table (10.1), based on natural period, T_a, of towers and tanks.

Table 10.1: value of λ

Perio (sec)	Λ
$T_{fs} \le 0.36$	$\sqrt{0.52 + 0.48\lambda}$
$0.36 < T_{fs} < 0.9$	$\sqrt{1-(1-\lambda)\left(\frac{1.8T_a T_{fs}}{T_a^2 + 1.21T_{fs}^2}\right)}$
$0.9 < T_{fs} < 1.1$	$\sqrt{y}$
$T_{fs} \ge 1.1$	$\sqrt{1-(1-\lambda)\left(\frac{2.2T_a T_{fs}}{T_a^2 + 1.21T_{fs}^2}\right)^2}$

T_{fs}: natural period of frame structure

γ: ratio of frame structure's weight

h_e: value obtained from figure (10.2) based on damping constant of towers and tanks

T_a: value obtained from equation (10.7)

$$T_a = 0.057\sqrt{\eta_a} \tag{10.7}$$

η_a: maximum displacement (mm) when the weight of tower or tank is used horizontally and the frame structure is assumed as rigid.

For the following cases, Ta = 0:

- Horizontal cylindrical storage tank with capacity less than 100 tons
- Vertical storage tank held by ring with distance between tangents less than 5m

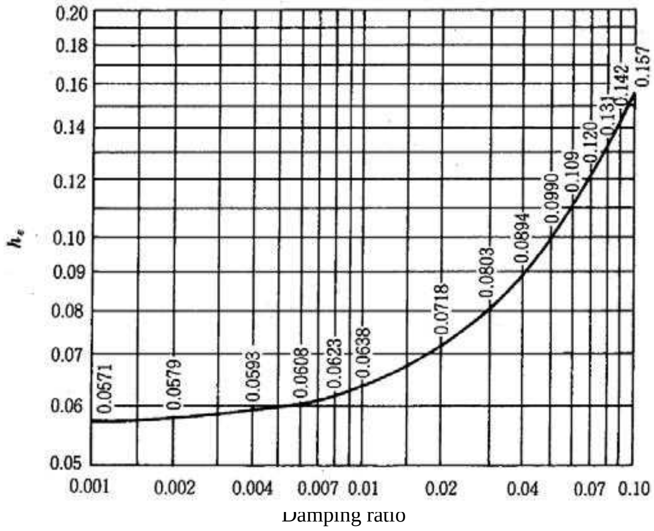


Figure 10.2: Value of h_e

μ: seismic distribution factor which is the maximum value obtained from equations (10.8) and (10.9), and is larger than 1.

$$\mu = \frac{1.5H}{H_t} \tag{10.8}$$

$$\mu = \frac{1.5H_o}{H_t} \tag{10.9}$$

H_t: maximum height of structure

H: height of point in which the design's modified seismic force, F_{MH}, of tower or tank is being measured.

H_0: height of lowest support point of towers and tank

K_{MH}: design's horizontal modified seismic factor. In this case, the response magnification factor, β_5, is obtained for maximum natural period between T_1 and T_2. The damping factor is also assumed equal to a value related to tower or tank.

$$T_1 = \sqrt{(1+\lambda)\frac{T_a^2 + T_S^2}{2}} \tag{10.10}$$

$$T_2 = \sqrt{(1+\lambda)\frac{T_a^2 + T_S^2}{2}} \tag{10.11}$$

K_{MV}: modified vertical seismic factor

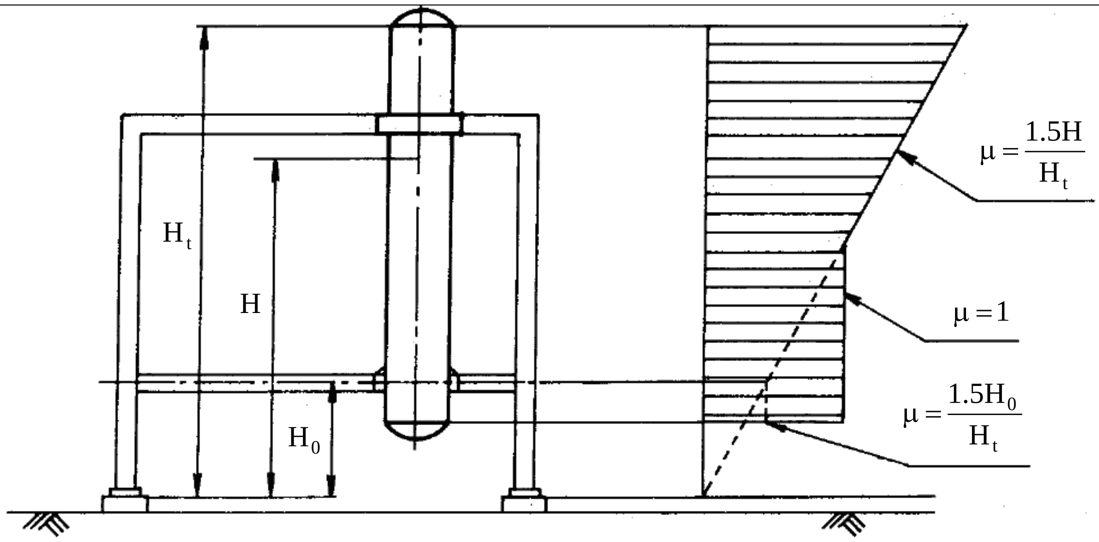


Figure 10.3: distribution of seismic factor of towers and tanks on frame structure

2-2-Frame structure of towers and tanks

The modified seismic force is obtained from equations (10.12) and (10.13). For holder structures with medium and low importance factor and support structures for which the pseudo-static method is allowable, the static method could be used for design.

$$F_{MH} = \mu K_{MH} W_H \tag{10.12}$$

$$F_{MV} = K_{MV} W_V \tag{10.13}$$

μ: distribution factor of seismic force. This factor is obtained using equation (10.14). If this equation is smaller than 1 or the height of frame is lower than 10m (figure 10.4), this value is assumed equal to 1.

$$\mu = \frac{1.5H}{H_t} \tag{10.14}$$

H_t: height of frame structure

H: height in which the design's modified seismic force of frame is measured.

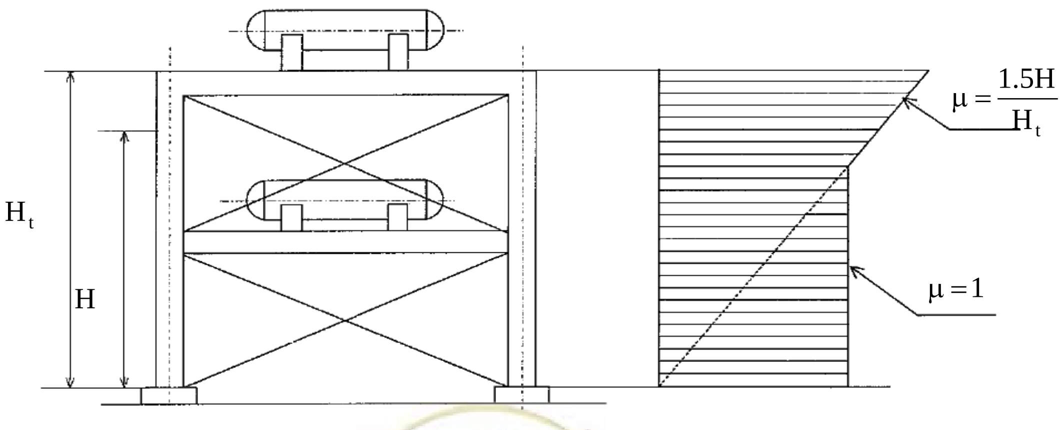


Figure 10.4: seismic distribution of frame structure (H_t > 10m)

For response analysis, the pseudo-static method or modified pseudo-static method are carried out based on the natural period of frame structure.

10-1-2-2-Calculated stress of cover and frame structures

Stresses are calculated using the seismic force of structure's design.

Parts in which the stress is calculated and the stress type is presented in table (10.2).

Table 10.2: Parts in which stress is calculated and stress type

Part in which stress is calculated	Type of Calculated Stress				
	Tension	Shear	Bending	Compressive	Buckling
Column			O	O	O
Beam	O	O	O	O	O
Bracing	O			O	O
Bracing rod	O	O			

10-1-2-2-3-Evaluation of calculated stresses

If the calculated stress is smaller than the allowable stress of design, the required evaluation of seismic performance via the allowable stress method is acceptable. If the calculated stresses are larger than allowable stresses, the characteristics of structure would change and the seismic performance evaluation is repeated.

10-1-3-Ductility design method

The ductility design of pseudo-building structures, such as equipments' frame structures, holder of furnace and chimney towers, is performed using the yield strength evaluation. The procedure of yield strength seismic design is presented in figure (10.5).

Required seismic performance

Figure 10.5: Procedure of yield strength seismic design

10-1-3-1-Design's horizontal seismic force calculation imposed on cover and frame structures

The horizontal seismic force using the natural period of structure and obtained modified seismic factor is compared and controlled by yield strength.

The structure's natural period is measured by analysis of characteristic value in frame analysis, but the natural period of structures with simple shapes could be obtained using the proper method described in section (10.1.2.1).

10-1-3-2-Damage mode

The damage mode of frame is the state in which part of the frame or the entire beam becomes instable and/or yields due to seismic force (plastic joint).

If the seismic force (horizontal force) imposed on frame is gradually increased, the plastic joint in each beam and column group would be consecutively created and the entire frame or part of it falls. On this basis, the shear force of storey is determined given the obtained yield strength, beam and column failure.

10-1-3-3-Current seismic demand

The current seismic demand, Q_{un}, for each frame storey is calculated using design's seismic force and equation (10.15).

$$ {}_{i}Q_{un} = D_s K_{MH} \sum_{i}^{n} \mu W_i \quad (10.15) $$

W_i: weight of each storey (N)
i_{Qun}: current seismic demand of i_{th} storey (N)
μ: seismic distribution factor in the direction of height (at least one)

$$ \mu = 1.5H / H_t \quad (10.16) $$

H_t: height of frame (mm)
H: height in which the modified seismic factor of frame design is calculated (mm)
K_{MH}: modified horizontal seismic factor obtained from equation (10.17)

$$ K_{MH} = \beta_5 K_H \quad (10.17) $$

β_5: horizontal response reinforcement obtained from equation (3.7) of seismic loading guideline of vital vessels
K_H: horizontal seismic factor at ground level elevation

The current seismic demand, Q_{un}, is obtained from sum of upper stories' demand. The amount of D_s for each storey could be obtained from table (10.3) and the maximum value (D_s = 0.5) could be used for each storey.

Table 10.3: Structure's characteristic factor, D_s

Frame characteristic \ Frame type		(a) Frame with rigid connection or frames of this type	(b) Cases other than (a) and (c)	(c) Frame with bracing
(1)	Frame with special plastic deformation like local buckling do not occur easily. (sudden decrease of strength against local buckling in response to imposed stresses is impossible)	0.25	0.3	0.35
(2)	Frames except for category 1 and frame with high plastic deformation (high ductility)	0.3	0.35	0.4
(3)	Frames except for category 1 and 2 and frame which its yield strength does not decrease rapidly. (local buckling caused by forces in members does not cause elastic deformations) (medium ductility)	0.35	0.4	0.45
(4)	Frames except for category 1 to 3 (low ductility)	0.4	0.45	0.5

10-1-3-4-Ultimate seismic demand

The ultimate seismic demand of frame could be obtained provided the following cases.

In calculating the ultimate seismic demand, Q_u, the practical simplified method could be used based on structure besides the elastoplastic analysis method. The calculation conditions are as follows:

1-The ultimate seismic demand is calculated for each flat frame separately.

2-It is assumed that the flat frame is a frame consists of axial linear member disregarding the beam and column connection type (panel connection).

3-It is necessary to relate the seismic demands to member's connection.

4-If both ends of beam reach their self-plastic state; the horizontal rigidity should be created for beam in order to prevent from lateral buckling.

5-Beam and column plastic moments are created in nodes.

6-The seismic demand of frames with bracing is obtained from total demands of bracing and frame seismic demands.

10-1-3-5-Yield strength evaluation

If equation (10.18) is satisfied for each frame storey, the frame would satisfy the seismic performance required by seismic force.

$$Q_{un} \le Q_u \tag{10.18}$$

Q_{un}: current seismic demand (N)

Q_u: ultimate seismic demand (N)

10-1-3-6-Pedestal yield strength evaluation

The pedestal must have sufficient yield strength and deformation capacity for non-destruction against seismic forces in accordance to section 23 of the appendix.

10-2-Chimney

10-2-1-Chimney seismic design procedure

The seismic design of chimney is performed using the allowable stress method for seismic forces.

10-2-2-Design notes

Chimneys have various types, like independent chimney and chimney held by metallic tower. The independent chimney is designed using the methods related to towers with skirt support described in section 4.1.5.5. On the other hand, since chimneys held by metallic tower, are similar towers with skirt support, therefore this type of chimney is designed similarly to towers supported by frame.

10-3-Furnace

10-3-1-Furnace seismic design procedure

The seismic design of furnace is performed using the allowable stress method or the ductility method. A furnace is a frame structure of beam and column in which a metal plate is installed on frame and is isolated from inside, bracing pipe, torch, vapors, and fire.
The analysis of heating pipe stress, heating pipe support, furnace wall, furnace frame, and torch could be performed independently.

Notes for designing each part are as follows.

1-The furnace wall should be able to bear the furnace's frame deformation and also have sufficient strength.

2-The furnace frame should be rigid as far as possible, and parts such as heating pipe, heating pipe support, furnace wall, or chimney should be designed so that could bear the design's seismic force.

3-The heating pipe should be a structure which prevents from resonance with furnace as far as possible, and also must have sufficient yield strength so that it would not fall due to inertia force and furnace frame deformation caused by earthquake.

4-The independent design of chimney is performed based on design procedure of chapter 9 and if it is held by furnace structure, it could be designed as an integrated structure.

5-Duct is a structure which absorbs the relative deformation between furnace and chimney and is designed based on the design procedure of piping system presented in chapter 5.

6-The vapor pipe for putting out fire is designed based on design procedure of piping system presented in chapter 5.

10-3-2-Furnace response analysis

10-3-2-1-Analytic modeling

Since furnaces are structures made of beam and plate (shell plate for installing thermal isolation materials), there are two modeling methods for their analysis. First model is the combined model of spatial frame in which the structural members are spatially modeled and the second method is the multi-mass spring model in which the mass is assumed at a virtual suitable position and are attached by spring members. In the first state it could be modeled as a two dimensional frame consistent with the furnace structure, and in the second model, the concentrated masses are virtually placed on the furnace's main beams position.

10-3-2-2-**Mass**

To analyze the stresses imposed on column, beam, bracing, and other components, it is necessary to model the mass as rigid. When the mass is exposed to concentrated loads or members' distributed load, the evaluation is usually automatically performed using structural information. In this model in order to analyze response, other components of furnace are only considered as masses and are evaluated. The exhaust pipe, isolation (cover fiber and non-organic (ceramic), etc.), heating pipe, fireplace, torch, installation base, and the main piping are examples of main components, which are evaluated as a mass. The mass of these components is focused on the node of member installed on. If the magnitude and location for addition of this mass is precisely determined, the modeling would be performed with good accuracy. The member's mass is imposed in a concentrated manner on the node.

10-3-2-3-Damping constant

Since various components of furnaces, such as pipes and isolations, are installed on frame, the furnace could be considered as a frame structure. The following values are obtained according to the damping constant of other equipments and it could be equal to damping constant of furnace.

(a) Non-organic fiber (ceramics) used in isolation: 15%

(b) Covering and fireproof brick used in isolation: 10%

Chapter 11

Seismic Design and Safety Control of Pipelines

11-1-Target Components

These lines could be divided into the following groups:

(a) On-ground (pipes on support or pipe bridge)

(b) Buried pipes including:

-Straight pipes

-Curve pipes

-Junctions (T-shaped)

(c) Gas riser

- Generally, on-ground pipes are on support, and their seismic design include base and pipe support.
- Buried gas pipes are made of steel and polyethylene with proper welds, have sufficient strength against effects caused by earthquake waves, and do not receive damages easily.
- The geotechnical hazards of ground inflict damages to pipes exposed to such hazards.
- The effects of very local ground deformation on buried pipes, especially service pipes at subscribers' junctions, are specifically examined.
- Gas risers are resistant against earthquake by themselves and cause no problems, however due to existence of threaded junctions in them, if the rotation angle increases from a specific amount, there would be leakage. This rotation limit is presented in this guideline.

11-2-Seismic calculations of on-ground pipeline

On-ground pipelines have not received any particular problems during earthquakes.

In this guideline, the seismic design of support legs is presented according to the damage mode of these pipes in previous earthquakes.

11-2-1-Loads and their combinations

1-The imposed loads in seismic design include:

1-1-Primary loads

a) Dead load

b) Pre-stressing force

c) Effect of weight, creep, and contraction of concrete

d) Earth pressure

e) Floating or uplift

1-2-Secondary loads

- Seismic effects (including the effect caused by propagation of earthquake waves and the effect caused by permanent deformations of ground)

2-The load combination is as follows:

- Primary loads

- Primary loads + earthquake effects (the loads' factor is one).

3-Loads should be applied in such a way that the most unfavorable stress, deformation, and other effects would be created.

The following items should be considered as seismic effects. But whenever the seismic design is performed by ductility method, there would no need to include items 1 and 2:

1-The created seismic force due to the weight of structure which is obtained by multiplying the weight of structure to seismic factor and in both horizontal and vertical directions, as shown in figure (11.1).

2-Ground pressure during earthquake

3-Effect of liquefaction and lateral spreading

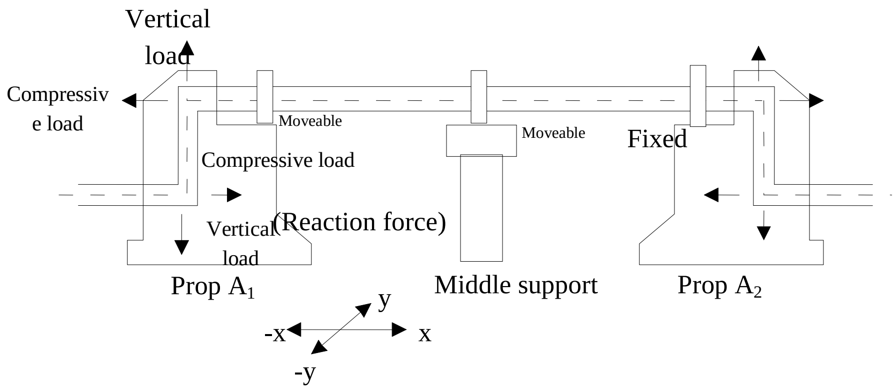


Figure 11.1: Model for a bridge carrying pipeline

The imposed loads on the above sample are presented in table (11.1).

Table 11.1: Structural analysis for a bridge with two spans

Dead weights: Side support A_2 (fixed)	Dead weights: Middle support	Dead weights: Side support A_1 (moveable)	Loading components	Loads
$3/16\sum W$	$5/8\sum W$	$3/16\sum W$	Vertical	Non-seismic loads
$\pm 3/16\sum W.\mu$	$\pm 5/8\sum W.\mu$	$\pm 3/16\sum W.\mu$	Horizontal (x direction): Friction force	Non-seismic loads
$P_1 A_r$	—	$P_1 A_r$	Horizontal (x direction): Pressure	Non-seismic loads
$3/16\sum W$	$5/8\sum W$	$3/16\sum W$	Vertical	Seismic: Axial
$\pm\sum W_0.K_{SH}$	$\text{Min}\begin{cases}\pm 5/8\sum W.\mu \\ \pm 5/8\sum W.K_{SH}\end{cases}$	$\text{Min}\begin{cases}\pm 3/16\sum W.\mu \\ \pm 3/16\sum W.K_{SH}\end{cases}$	Horizontal (x direction): Friction or inertia	Seismic: Axial
$P_2 A_r$	—	$P_2 A_r$	Horizontal (x direction): Pressure	Seismic: Axial
$3/16\sum W$	$5/8\sum W$	$3/16\sum W$	Vertical	Seismic: Transverse
$\pm 3/16\sum W.K_{SH}$	$\pm 5/8\sum W.K_{SH}$	$\pm 3/16\sum W.K_{SH}$	Horizontal (x direction): Inertia force	Seismic: Transverse
$\pm 3/16\sum W.K_{SH}.H_g$	$\pm 5/8\sum W.K_{SH}.H_g$	$\pm 3/16\sum W.K_{SH}.H_g$	Rotation moment	Seismic: Transverse

ΣW: total weight on bridge system structure
K_{SH}: horizontal seismic factor
ΣW_0: ΣW – weight of water
μ: friction factor
H_g: height of bridge's side support center of mass
P_1: fluid's pressure
P_{2l}: fluid's dynamic pressure
A_r: pipe's section area

11-2-2-Design seismic force of load bearing supports

The design's seismic force which is expanded upwards and downwards is considered as follows.

$$R_L = R_{D_f} + \sqrt{R_{HEQ}^2 + R_{VEQ}^2} \tag{11.1}$$

$$R_U = R_{D_f} - \sqrt{R_{HEQ}^2 + R_{VEQ}^2} \tag{11.2}$$

R_1: upwards seismic force (kN)

R_U: downwards seismic force (kN)

The amount of RU should not exceed -0.3RDF for hazard level-2, and -0.1RDF for hazard level-1.

R_{DF}: reaction force (kN) of support caused by dead weight on the structure. The downwards reaction force must be assumed positive.

The upwards and downwards reaction force, RHEQ, could be obtained using equation (11.3).

$$R_{HEQ} = \frac{K_H \cdot R_{VP} \cdot H_p + K_H \cdot R_{VW} \cdot H_W}{B_W} \tag{11.3}$$

K_H: horizontal seismic intensity factor

R_{VP}: weight of pipe and fluid

R_{VW}: weight of passageway (there is a passage above pipe's section, holding sheath, and its support)

H_P: height from center of pipe

H_w: height from mass center of passageway

B_W: width of support

R_{VEQ}: upwards or downwards reaction force (kN) created by vertical seismic intensity factor, KV, which could be obtained as follows:

$$R_{VEQ} = K_V R_{D_f} \tag{11.4}$$

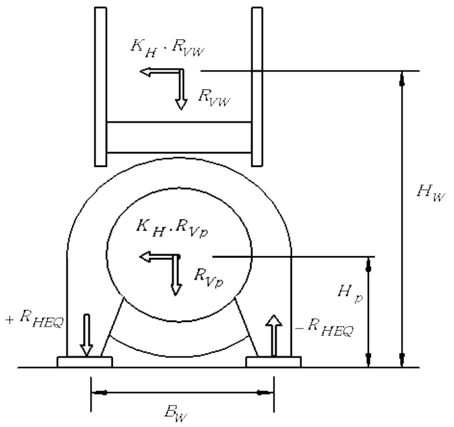


Figure 11.2: upwards and downwards force created due to lateral force

11-2-3-Safety control of load-bearing supports

1-Strength of support frame, on- and foundation equipments attached to support should not be less than lateral seismic force.

2-The support strength should not be too high that the expansion connection could not have a good performance.

3-The expansion connection should have sufficient ductility to absorb the support's vertical displacement.

11-2-4-Length of seat

The minimum distance to anchor-bolt in support should be less than the amount of S_1 obtained from equation (11.5).

$$S_1 = 0.20 + 0.005L \tag{11.5}$$

L is the length of span (m).

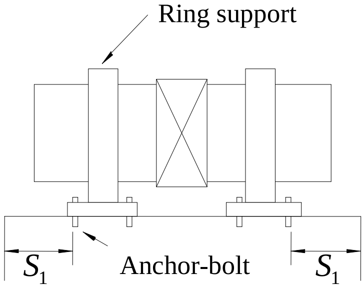


Figure 11.3: Minimum length to anchor-bolt in support

11-2-5-Bracing constraint and clamp

- The constraints and clamps should be installed on structures to prevent from leakage and overturn.
- The components of constraints and clamps should be properly selected considering the pipe bridge type, load-bearing type, ground conditions, etc.
- Constraints and clamps perpendicular to pipe bridge axis are installed in connection support, bridges' overlapping parts, and middle supports of bridges' girder.
- Constraints and clamps should include the following structural components:

  1-Superstructure to foundation connection

  2-Attachment piece to superstructure and foundation

  3-Superstructures connected to both sides

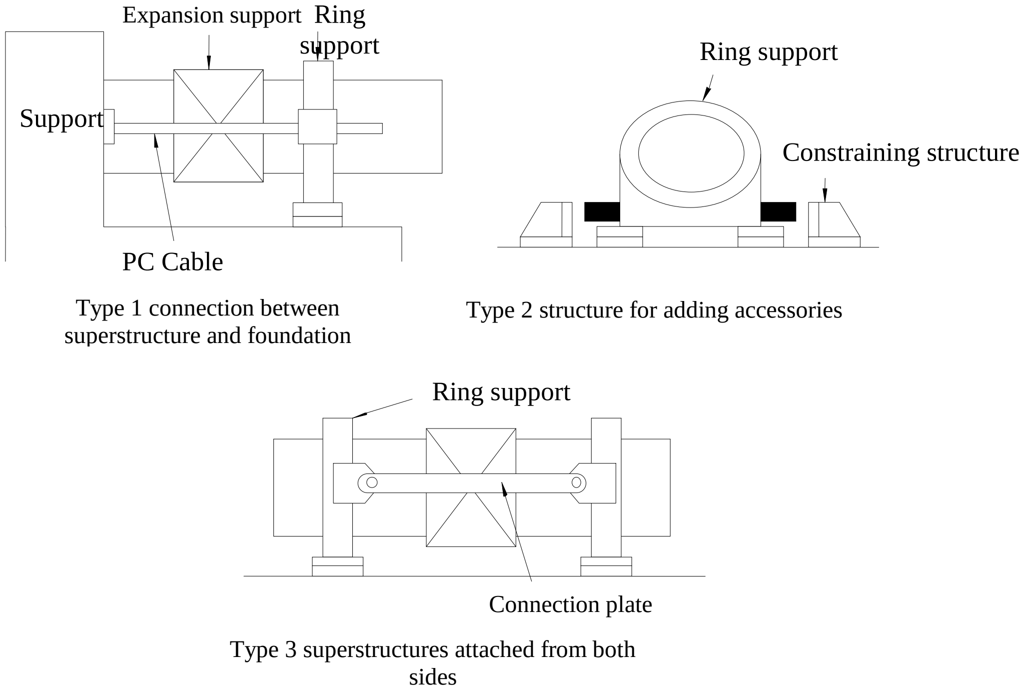


Figure 11.4: Details of constraints and clamps

11-2-5-1-Design force of constraints and clamps

The resistant force of constraints and clamps should not be smaller than the design's seismic force, H_F, determined from equation (11.6).

The maximum displacement of constraints should be equal to the value of S_F given from equation (11.7).

$$H_F = 1.5R_d \tag{11.6}$$

$$S_F = c_F S_E \tag{11.7}$$

H_F: design's seismic force of constraint (kN)

R_d: reaction of dead weight (kN)

S_F: design's maximum displacement of constraint (cm)

S_E: length of girder's seat in support (cm)

C_F: design's displacement factor of constraint equal to 0.75

11-2-5-2-Displacement limitation of constraints and clamps

The maximum displacement of constraint should not exceed the allowable displacement of expansion connection.

The maximum displacement of constraint could be estimated as follows:

1-For no-earthquake state

$$\delta_x = \delta_T + \Delta_x \tag{11.8}$$

2-For hazard level-1 earthquake

$$\delta_{x1} = \delta_T + U_R + \Delta_x \tag{11.9}$$

3-For hazard level-2 earthquake

$$\delta_{x2} = \frac{\delta_T + U_R + U_G}{n_1} + \Delta_x \tag{11.10}$$

δ_x: displacement of expansion connection (mm)

Δ_x: allowable length for unknown excess displacement (mm)

δ_T: displacement from temperature increase (mm)

$$\delta_T = \alpha_t \cdot \Delta T \cdot L \tag{11.11}$$

α_t: thermal expansion factor

ΔT: temperature increase

L: average support structure span length

U_R: relative displacement (mm) between adjacent support structures

$$U_R = \sqrt{U_{R1}^2 + U_{R2}^2} \tag{11.12}$$

U_{R1}: response of support structure 1 (mm)

U_{R2}: response of support structure 2 (mm)

U_G: relative displacement (mm) caused by soil deformation between adjacent support structures

$$U_G = \varepsilon_G \cdot L \tag{11.13}$$

ε_G: strain of ground surface free field

n_1: number of expansion connections

11-3-Buried pipelines

In this section, the buried pipelines are divided into three parts, namely straight, curve, and T-shaped branches, and the seismic design as well as safety control of each against effects caused by wave propagation and geotechnical hazards are presented, respectively.

The behavior evaluation methods and seismic strains calculations in buried pipelines are based on the theory of beam on elastic bed and interaction between soil and pipe.

The main method of this guideline in seismic analysis of buried lines is the displacement response method which its detailed description is presented in the guide for seismic loading of vital vessels.

11-3-1-Straight pipelines

11-3-1-1-Design for seismic wave propagation

The displacement response method is used for estimating the stresses of buried pipe.

This method is presented in the guide for seismic loading of vital vessels.

Based on this method, the ground strain, ε_G, could be obtained.

The longitudinal strain in pipe's transverse section could be obtained from the following equations:

For elastic parts:

$$\varepsilon_{pl} = \alpha \varepsilon_G \tag{11.14}$$

For non-elastic parts:

$$\varepsilon_{pl} = \varepsilon_G \tag{11.15}$$

$$\alpha = q \cdot \alpha_0 \tag{11.16}$$

α_0: factor for converting ground stress to pipe stress at longitudinal direction

$$\alpha_0 = \frac{1}{1+\left(\dfrac{2\pi}{\lambda_1 \cdot L_A}\right)^2} \quad , \lambda_1 = \sqrt{\frac{K_1}{E \cdot A_r}} \quad , L_A = V \cdot T_p \tag{11.17}$$

L_A: apparent wave length (m)
T_G: ground dominant period (s)
V: wave transmission speed (m/s)
K_1: soil's spring constant around longitudinal direction (N/m^2)
A_r: transverse section area (m^2)
E: Young's modulus (N/m^2)
q: slippage factor

$$\begin{aligned} q &= 1-\cos\xi + \Omega \cdot \left(\frac{\pi}{2} - \xi\right)\sin\xi & \tau_G \geq \tau_{cr} \\ q &= 1 & \tau_G < \tau_{cr} \end{aligned} \tag{11.18}$$

Ω: modification factor of q, which is equal to 1.5

$$\xi = \arctan\left(\frac{\tau_{cr}}{\tau_G}\right) \tag{11.19}$$

τ_{cr}: critical shear stress at the beginning of slippage (N/m^2)
τ_G: applied shear stress on pipe's surface (N/m^2)

$$\tau_G = \frac{2}{L_A} \cdot \frac{EA_r}{D_i} \cdot \alpha_0 \varepsilon_G = K_1 \cdot (1-\alpha_0) \cdot U_h \tag{20.11}$$

D_i: diameter of pipe (m)
The definition of K_L is presented in equation (10.17) and U_h is utterly explained in guide for seismic loading of vital vessels.

11-3-1-1-2-Allowable strain

In hazard level-1, for straight pipes with average diameter (D_m) and thickness (t), the allowable strain is the minimum value between $0.35t/D_m$ and 1%.
This strain for curve is equal to 1%.
In hazard level- 2, the critical strain for straight, curve, and tee-shaped pipes is 3%.

11-3-1-1-3-Maximum strain of pipe on ground's transmission region (special case of crossing of pipes from boundary of two grounds)

When the pipeline is crossing hard soil to reach soft soil, the boundary area response would be more than response in uniform soil.
This increase in response could be estimated by the approximate equation (11.21):

$$\varepsilon_{G2} = \sqrt{\varepsilon_{G1}^2 + \varepsilon_{G3}^2} \tag{11.21}$$

ε_{G1}: strain in uniform soil
ε_{G3}: soil's excess strain in transmission area is equal to 0.003.

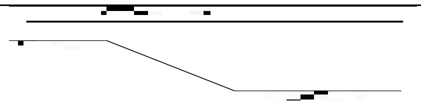


Figure 11.5: Pipeline in soil's transmission area

11-3-1-1-3-1-Soil's resilience at longitudinal direction

In figure (11.6), the applied shear stress on pipe's surface on axis length has bilinear characteristics with soil resilience, K_1.

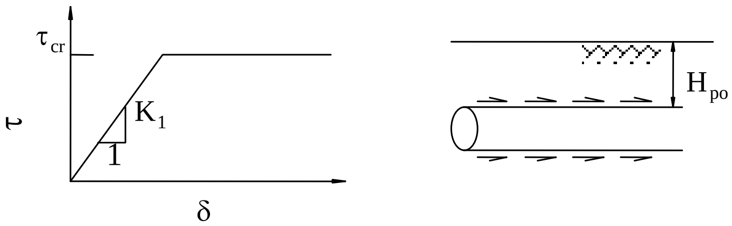


Figure 11.6: Soil rigidity at length of pipe's axis

When the pipeline is installed in different depths, H_{po}, the shear stress could be converted using equation (11.22). In this equation, H_{po} is in meter

$$\tau_{cr} = 1.5\left(\frac{H_{po}}{1.8}\right) \qquad (N/cm^2) \tag{11.22}$$

11-3-1-1-3-2-Soil rigidity at transverse direction

The soil's resilience, k_2, could be obtained by dividing the maximum limited stress of soil, σ_{cr}, to yield displacement, δ_{cr}, table (11.2).

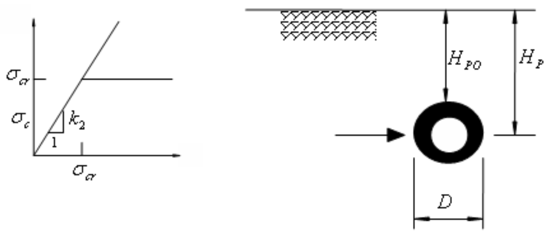


Figure 11.7: Soil rigidity perpendicular to pipe's axis

Table 11.2: Limiting stress perpendicular to pipe's axis

Soil rigidity (N/cm^3)	Yield displacement (cm)	Maximum limiting stress (N/cm^2)	Nominal diameter (mm)
20	2.6	53	100
20	2.6	51	150
18	2.6	48	200
16	2.7	42	300
14	2.8	39	400
13	2.8	36	500
12	2.9	34	600
11	2.9	33	750
10	3.1	30	900

11-3-1-2-Location of maximum pipe's strain caused by permanent ground deformation (PGD)

The maximum pipe's strain for liquefaction occurs at the interface of liquefied and non-liquefied materials.
The maximum pipe's strain for faulting emerges at first failure path.
The maximum pipe's strain for landslip occurs at the boundary of slopes and at the largest non-uniform settlement.

11-3-1-2-1-Design for horizontal displacement caused by liquefaction

For straight pipe perpendicular to slope's axis, the curvature angle of straight pipe (deg) could be obtained from equation (11.23):

$$P_1 = D_i.\gamma_k.\sigma_c \quad \text{for} \quad \omega_s = \frac{180}{\pi}.127.D_i.\sqrt{\frac{P_1.\gamma_\delta.\delta_h}{EI}} \tag{11.23}$$

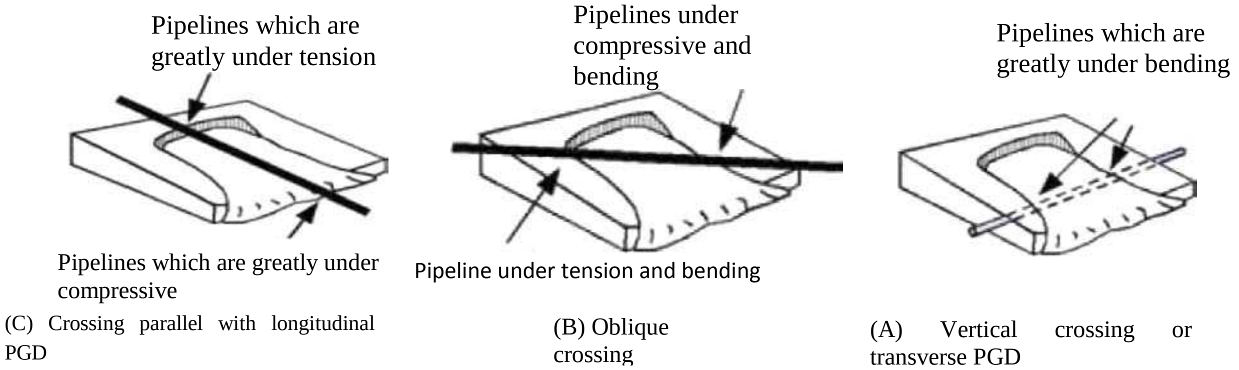


Figure 11.8: Crossing situations from liquefied area

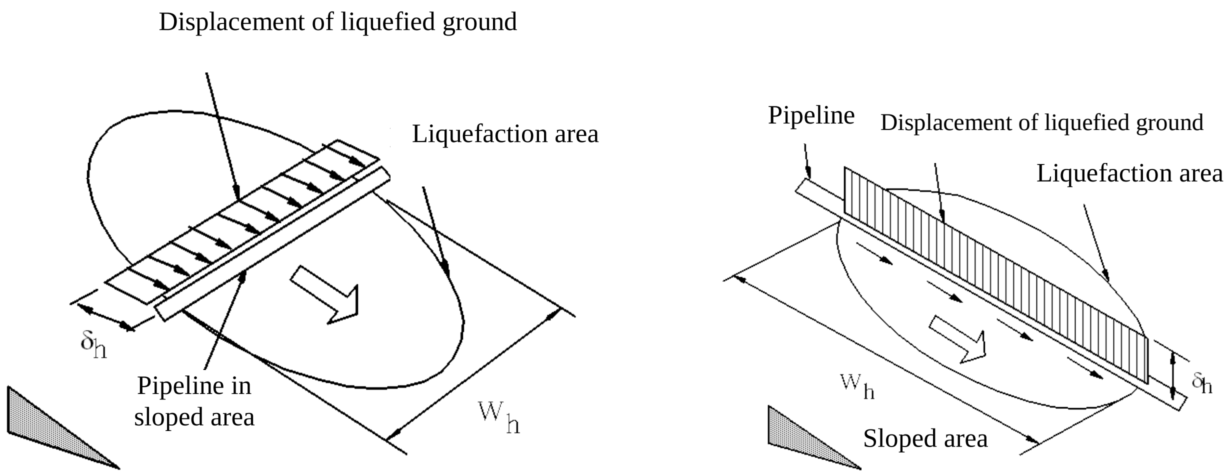


Figure 11.9: Longitudinal and transverse loading of buried straight pipe in crossing from liquefaction area

Table 11.3: Partial safety factors

Partial safety factors			Members	Displacement mode	Standard value
Load factor	γ_δ, for ground displacement	Slope	Pipe and curve	All modes	1.8
		Pier, dock			1.3
		Settlement			1.0
	γ_k, for ground resistant force	Axial	Pipe and curve	All modes	1.2
		Transverse			1.2
		Perpendicular			1.1

11-3-1-2-2-Design in crossing with fault

The created strain in pipes due to ground displacement, PGD, at the crossing with fault could be calculated using equation (11.24).

$$\varepsilon_{pipe} = 2\left[\frac{PGD}{2L_a}\cos\beta + \frac{1}{2}\left(\frac{PGD}{2L_a}\sin\beta\right)^2\right] \quad (11.24)$$

Where, β and L_a are the contact angle with fault relative to pipe's axis and the effective length of pipe displacement caused by fault's displacement, respectively, figure (11.10).

If the structural analysis implies that the standard design of pipelines could not safely bear the amount of fault displacement at intersection with the fault, then it is necessary to use the site-specific design.

The site-specific design includes such items as follows:

- Increase in thickness and rigidity of pipe
- Using modified backfill methods
- Considering displacements, surface topography, fault's width or fault's area, soil conditions, environmental effects, and closeness of neighboring structures
- Control of drainage and erosion

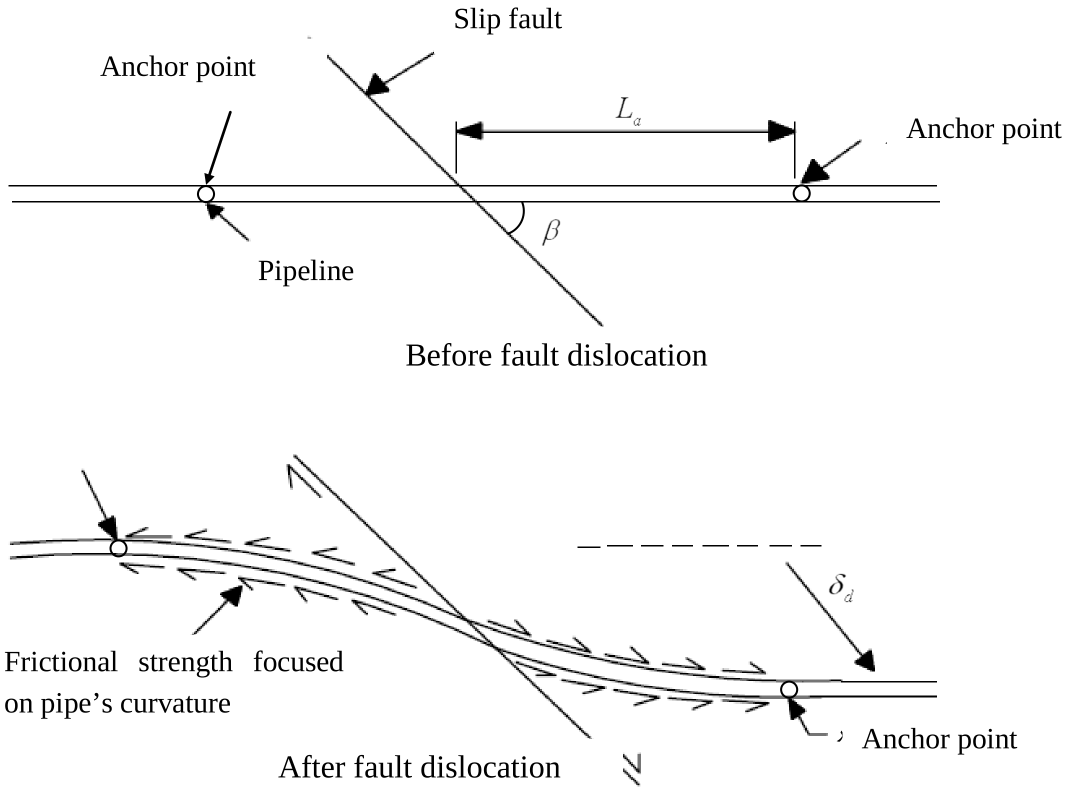


Figure 11.10: Buried straight pipe in intersection with fault

11-3-1-2-3-Design in landslip

The pipe's response depends on the direction of PGD of soil bulk. Figures (11.11) and (11.12)

1-If the movement of soil bulk is parallel to pipe's axis, it is called longitudinal PGD.

2-If the movement of soil bulk is perpendicular to pipe's axis, it is called transverse PGD.

- Usually, the longitudinal PGD inflict more damage in pipes than transverse PGD.
- In cases of pipes with high importance factor which are exposed to large PGDs (more than 30cm) (such as land slip or surface fault displacement), the FEM method with more details is used.

11-3-1-2-3-1-Response of buried pipe to longitudinal PGD

The displacement and maximum force of pipe occur in boundary locations of the dislocated soil bulk which lead to tension (pullout in point A at the beginning of dislocated soil bulk in figure (11.11)) or compressive in point B in figure (11.12) in pipe.

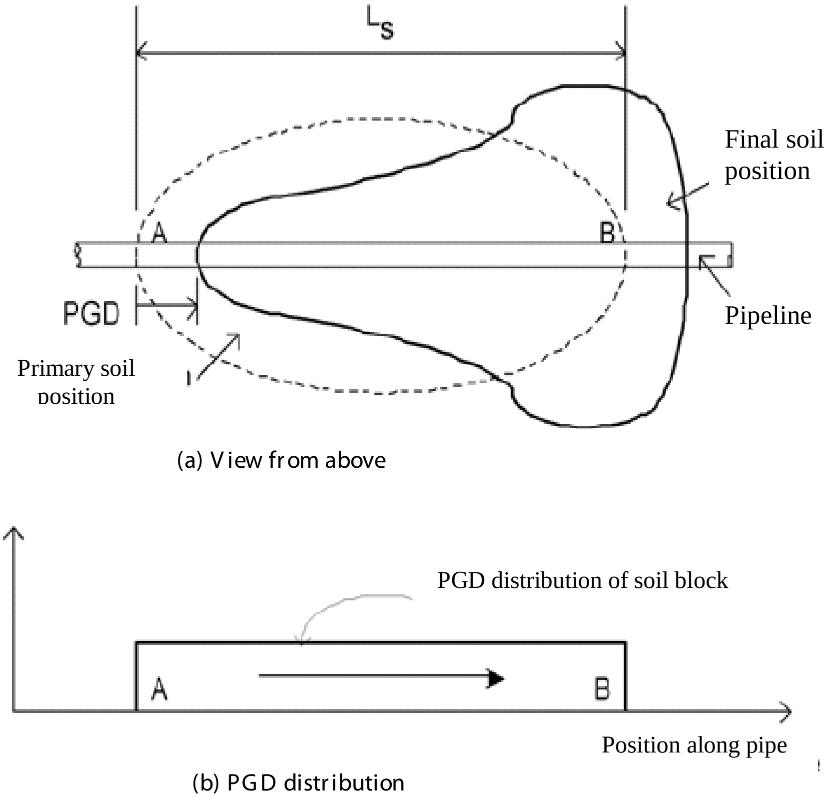


Figure 11.11: Response of buried straight pipe to longitudinal PGD

The design force of body and pipe connection could be obtained from the smallest F_1 and F_2 which indicates upper boundary of axial force in pipe. F_1 is calculated assuming the completing matching of pipe's behavior with soil, and F_2 is the ultimate transmitted force from soil to pipe.

$$F = \min(F_1, F_2) \tag{11.25}$$

$$F_1 = \frac{A_r E \delta}{L_{sp}} \tag{11.26}$$

$$F_2 = \pi D_0 L_{SP} \tau_{cr} \tag{11.27}$$

A_r: Pipe's wall area (cm^2)
E: elastic rigidity (N/cm^2)
L_{sp}: length of pipe in dislocated soil bulk (cm)
D_0: external diameter of pipe (cm)
δ: ground permanent displacement (cm)
τ_{cr}: shear (tangential) stress between soil and pipe (N/cm^2)

11-3-1-2-3-2-Buried pipe response to transverse PGD

The displacement of pipe is considered as a loaded beam which its maximum displacement occurs at the middle of the span.
As shown in points A, B, and C of figure (11.12), the maximum bending strain of pipe occurs both in the center and near to soil bulk boundaries which is transversely displaced.
The value of equation (11.28) could be conservatively considered as the bending strain.

$$\varepsilon_b = \pm \frac{\pi D_o \delta}{W_G^{\ 2}} \tag{11.28}$$

Where, W_G is the soil bulk width, D_0 denotes the pipe's external diameter, and δ is the maximum PGD.

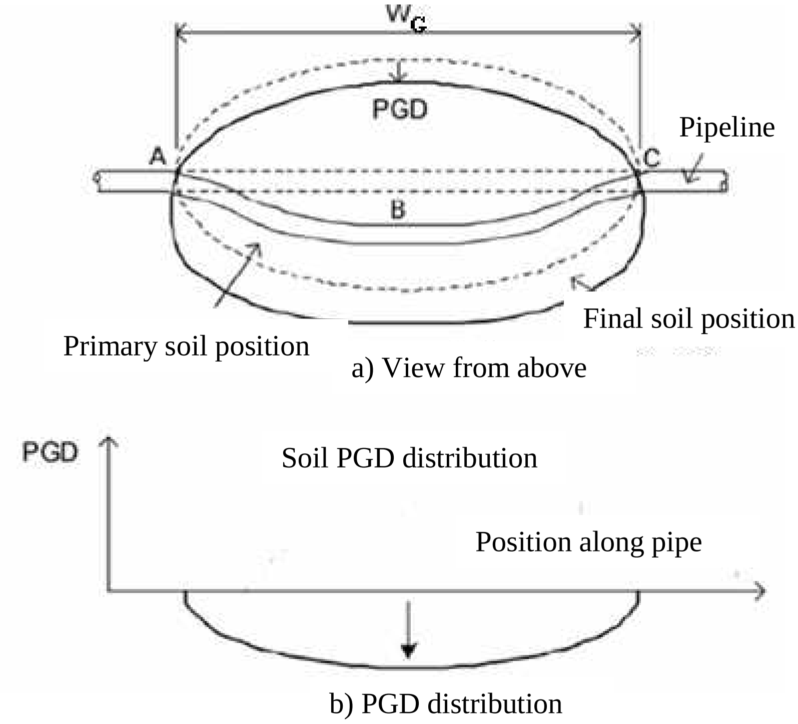


Figure 11.12: Response of buried straight pipe to transverse PGD

- The pipe's bending strain, which is obtained from the above method, is highly overestimated.
- If the design using the above strain is not practical, it is suggested to use the finite element method.

11-3-2-Buried straight pipes in local displacements of ground (gap local settlement)

In loading caused by seismic wave's propagation or geotechnical hazards, large length of pipe is affected. In this section, the design is presented for a section of pipe in ground's local landmarks.

11-3-2-1-Axial displacement capacity, Δu, in straight pipeline

Sometimes there might be gaps and local leakage at junctions of service pipes to station structures. These pipes usually contain low pressure and sometimes medium pressure (large industrial applications) gases.

As shown in figure (11.13), for the highest pipe length increase is considered as the capacity of displacement, Δu.

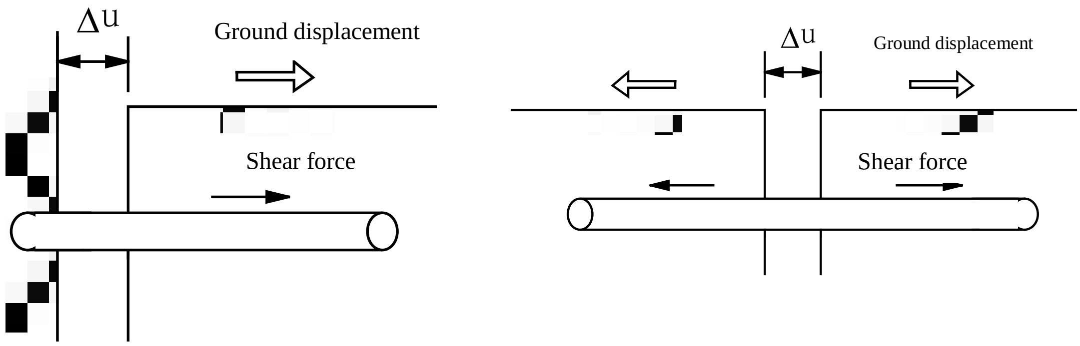


Figure 11.13: Axial displacement of a straight pipe (gap)

(a) Critical axial displacement of polyethylene pipes

$$\Delta u = \frac{A\bar{E}\varepsilon_0^2}{\pi D \tau} \quad \text{(cm)} \tag{11.29}$$

D: external diameter of pipe (cm)

A: pipe's wall area (cm^2)

$\bar{E}$: Equivalent elastic rigidity table (11.8) (N/cm^2)

ε_0: critical strain table (11.8)

τ: ground shear stress (N/cm^2)

(b) Critical axial displacement in steel pipes or ductile pipes

$$\Delta u = \frac{AE\left|\varepsilon_y^2 + \lambda\left(\varepsilon_0^2 - \varepsilon_y^2\right)\right|}{\pi D \tau} \tag{11.30}$$

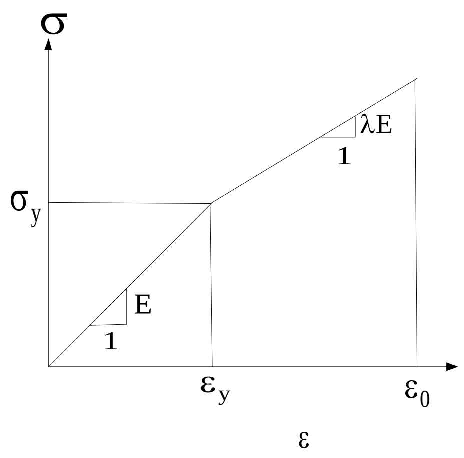


Figure 11.14: Determining E and λE

ε_y: yield strain

σ_y: yield stress (N/cm^2)

λ_E: strain hardening factor

Table (11.4) shows the standard values of shear stress for different types of pipes.

Table 11.4: Standard value of shear stress, τ (N/cm^2)

Pipe	Shear stress, τ (N/cm^2)
Steel pipe with plastic cover	0.98
Polyethylene pipe	
Polyvinyl chloride pipe	
Steel pipe without cover	1.47

11-3-2-1-2-Transverse displacement capacity, Δv, of straight pipeline

Two common types of soil conditions are presented in figure (11.15).

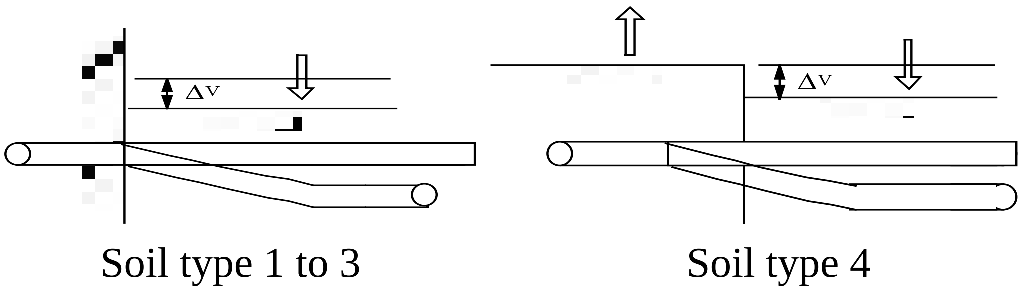


Figure 11.15: Ground vertical displacement (settlement)

(1) Vertical displacement in soils type 1 to 3

1) Steel pipe welded using electric arc method and polyethylene pipe:

$$\Delta v = \frac{2\sqrt{2}e^{\frac{\pi}{4}}}{D}\sqrt{\frac{4\bar{E}I}{kD}}\varepsilon_0 \tag{11.31}$$

e: Napier's constant

$\bar{E}$: equivalent rigidity (modulus) of elasticity from table (11.8) (N/cm^2)

I: section inertia moment (cm^4)

k: ground reaction factor (N/cm^3)

ε_0: critical strain from table (11.8)

D: external diameter of pipe (cm)

2) Vertical displacement in soil type 4

1) Steel pipe welded using electric arc method and polyethylene pipe:

$$\Delta v = \frac{1}{D}\sqrt{\frac{4\bar{E}I}{kD}}\varepsilon_0\,(\mathrm{cm}) \tag{11.32}$$

2) Pipe fixed in wall:

$$\Delta v = \frac{1}{2EI}\sqrt{\frac{4EI}{kD}}M_0\,(\mathrm{cm}) \tag{11.33}$$

$$M_0 = 269000 \quad (N.cm)$$

3) Pipe with ductile connection:

$$\Delta v = \mathrm{Min}(\Delta v_1, \Delta v_2, \Delta v_3) \tag{11.34}$$

Where, (dimensions in cm):

$$\Delta v_1 = \frac{e^{\beta l}}{D\beta^2\left\{\sin(\beta l) - \dfrac{\mu\theta_0}{2(\overline{E}I\beta + \mu\,\theta_0)}\cos(\beta l)\right\}} \tag{11.35}$$

$$\Delta v_2 = \frac{\left\{2(\overline{E}I\beta + \mu\theta_0)\right\}}{D\beta^2\mu\theta_0}\varepsilon_0 \tag{11.36}$$

$$\Delta v_3 = \frac{\left\{2(\overline{E}I\beta + \mu\theta_0\right\}}{D\overline{E}\beta^2}\theta_0 \tag{11.37}$$

$$\beta = \sqrt[4]{\frac{kD}{4\overline{E}I}} \tag{11.38}$$

$$\beta \cdot l = \tan^{-1}\left(1 + \frac{\mu\theta_0}{\overline{E}I\beta}\right) \tag{11.39}$$

μ: relative factor between rotation angle of a connection and imposed moment (N.cm/rad)
θ_0: critical angle of connection (rad)

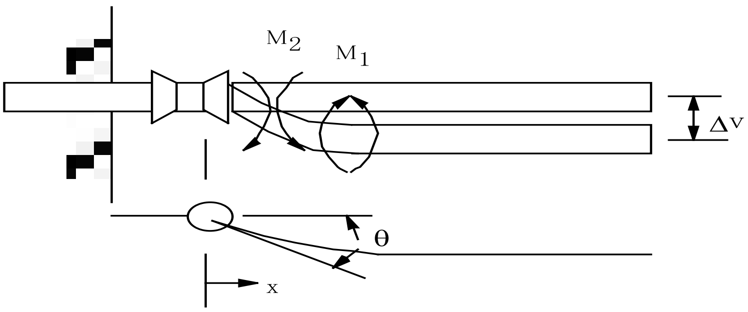


Figure 11.16: Piping model with ductile connection near wall

The maximum moment, M_{max}, is as follows:

$$M_{max} = \text{Max}(M_1, M_2) \tag{11.40}$$

$$M_1 = 2\overline{E}I\beta^2 e^{-\beta l}\left\{\sin(\beta l) - \frac{\mu}{2\overline{E}I(\beta + \mu)}\cos(\beta l)\right\}\Delta v \tag{11.41}$$

$$M_2 = 2\overline{E}I\beta^2 \frac{\mu}{2\overline{E}I(\beta + \mu)}\Delta v \tag{11.42}$$

According to empirical results, the μ factor is as follows:

$$\mu \approx \frac{M_0}{\theta_0} = 1.1 \times 10^6 \qquad \text{(N.cm/rad)} \tag{11.43}$$

$M_0 = 269000 \quad$ (N.cm)

$\theta_0 = 0.25 \quad$ rad

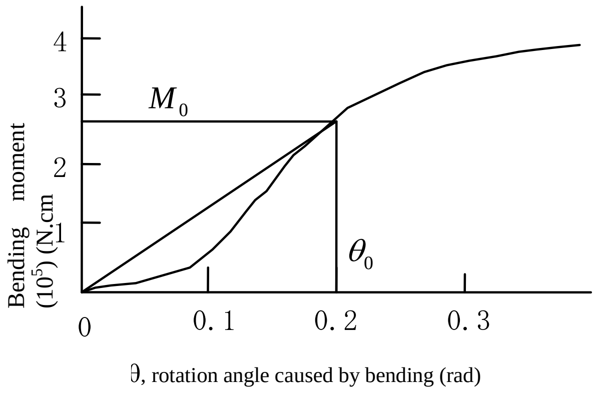


Figure 11.17: Moment-rotation angle diagram in a pipe's connection

4) Ground resilience, k, in transverse direction

$$k = \frac{1}{3} K_{30} \left(\frac{D}{17}\right)^{-\frac{3}{4}} \quad (N/cm^3) \tag{11.44}$$

D and K30 are pipe's diameter and ground's reaction factor from plate test with 30cm diameter, respectively.

If the value is K30 is unknown, it could be considered as K30 = 29.4 N/cm^3.

11-3-3-Curve pipes (buried lines)

In this section, similar to seismic design of buried straight pipelines, the seismic design of buried curve piped is presented.

11-3-3-1-Design for seismic wave propagation

The bending strain in curve pipes could be calculated as follows:

$$\begin{aligned} \varepsilon_B &= \beta_B \Delta & \beta_B \Delta &\le 1.27\varepsilon_y \\ \varepsilon_B &= C_B \beta_B \Delta & \beta_B \Delta &> 1.27\varepsilon_y \end{aligned} \tag{11.45}$$

β_B: pipe's curvature conversion factor from equation (11.47)

C_B: bending strain modification factor in complete-plastic area

For pipes with diameters lower than 600mm, CB = 2

For pipes with diameters larger than 600mm, CB = 1

Δ: relative displacement between free field and straight pipe connected to curve

$$\Delta = (1-\alpha) \cdot U_h \tag{11.46}$$

α could be obtained from equation (11.16).

the conversion factor for curve could be calculated as follows:

$$\beta_B = \frac{2 i_B A_r \lambda^2 D_o \left|(5 + R_r \lambda) b_1\right| + 4\lambda^3 I_r \left|5(1 + b_2) - b_1\right|}{10A_r + 5L_A I_r \lambda^3 (1 + b_2) + 10A_r b_3} \tag{11.47}$$

$$b_1 = -\frac{1 + 2R_r \lambda + (\pi - 2) n_b R_r{}^2 \lambda^2}{(1 + R_r \lambda)\left|2 + \pi n_b R_r \lambda + (4 - \pi) n_b R_r{}^2 \lambda^2\right|} \tag{11.48}$$

$$b_2 = \frac{1 - 2n_b R_r^{\,2}\lambda^2 - (4-\pi)n_b R_r^{\,3}\lambda^3}{(1+R_r\lambda)\left|2+\pi n_b R_r\lambda + (4-\pi)n_b R_r^{\,2}\lambda^2\right|} \quad (11.49)$$

$$b_3 = n_b R_r^{\,3}\lambda^3\left\{\frac{\pi}{2} + \frac{\pi I_r}{2n_b A_r R_r^{\,2}} + \left(1 - \frac{I_r}{n_b A_r R_r^{\,2}}\right)b_1 + \left(\frac{2}{R_r\lambda} + \frac{\pi}{2} + \frac{\pi I_r}{2n_b A_r R_r^{\,2}}\right)b_2\right\} \quad (11.50)$$

Uh: width of ground's horizontal displacement (cm)

β_B: Curve's conversion factor (1/cm)

i_B: factor (without dimension) of stress resonance in curve

$$i_B = \max\left\{\frac{1.95}{h^{2/3}}, 1.5\right\} \quad (11.51)$$

n_b: curve's flexibility factor (without dimension)

$$n_b = \frac{1.65}{h} \quad (11.52)$$

h: pipe's factor

$$h = \frac{tR_r}{r_m^{\,2}} \quad (11.53)$$

t: thickness of pipe (cm)

r_m: pipe's mean diameter (cm)

A_r: pipe's cross-section area (cm^2)

R_r: curve's curvature radius (cm)

I_r: inertia moment of section area (cm^4)

L_A: apparent wave length (cm)

λ: parameter (1/cm)

$$\lambda = \sqrt[4]{\frac{K_2}{4EI_r}} \quad (11.54)$$

K_2: transverse soil resilience relative to pipeline (N/cm^2)

E: elastic modulus of steel pipe (N/cm^2)

- The bending strain relation, ε_B, could be obtained using the relative displacement, Δ, between the free field and the straight pipe connected to curve, caused by seismic wave propagation along the straight pipeline.
- If the plastic strain exceeds the critical limit, $1.27\varepsilon_y$, plastic joint would be created.
- The plastic strain is determined considering the safety margin using parameter C_B from the equivalent linear approximate method.

11-3-3-2-Seismic design in liquefaction

11-3-3-2-1-Deformation of pipe's curve in sloped areas

In sloped areas, the GPD caused by liquefaction, δ_h, is downwards.

This displacement, as shown in figure (11.18), decreases from the upstream point to the downstream point.

The distribution of ground displacement is assumed to be triangular.

The maximum rotation angle, W_{bs}, caused by δ_h is given by equation (11.55). this value should not exceed from the allowable rotation given in table (11.6).

$$w_{bs} = \frac{180}{\pi}\left|\frac{3}{4}\pi - 2\sin^{-1}\left\{0.92 - 35.4\frac{\delta_h}{L_{ps}}\left(0.88\frac{D_o}{D_{600}} + 0.12\right)\right\}\right| \qquad (11.55)$$

$$L_{ps} = \sqrt{2}\sqrt{\frac{M_{pbs} + M_{ps}}{P_1}} \qquad (11.56)$$

$$M_{ps} = \frac{4}{\pi}M_{ys} \qquad (11.57)$$

$$M_{ys} = \frac{\pi}{32}\frac{D_o^4 - (D_o - 2t_s)^4}{D_o}\sigma_y \qquad (11.58)$$

D_{600}: standard 600mm-pipe diameter (mm)
D_0: internal diameter of pipe (mm)
σ_y: pipe's yield stress (N/cm^2)
M_{ys}: pipe's yield moment (N.mm)
L_{ps}: parameter (mm)
M_{ps}: complete-plastic bending moment of pipe (N.mm)
M_{pbs}: complete-plastic bending moment of pipe and curve (N.mm)
t_s: thickness of pipe (mm)
P_1: soil pressure on pipe as a spread load (N/mm)

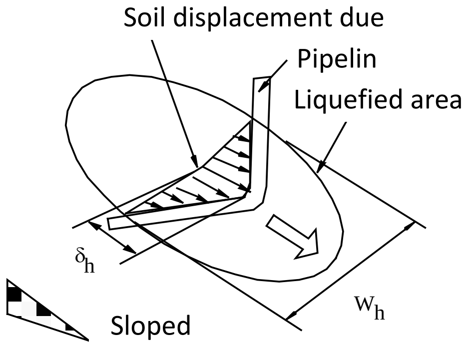


Figure 11.18: Analytical model of pipe's curve in sloped liquefied area

11-3-3-2-2-Pipe's curve deformation in shore areas

The rotation angle (deg) at the outer side (with D_o/t_o less than 50) in shore areas is calculated as follows; this value must not exceed the allowable rotation given in table (11.6).

$$w_{bs} = \frac{180}{\pi}\frac{150\delta_h}{L_{po1}}\left(0.49\frac{D_o}{D_{600}} + 0.69\right) \qquad (11.59)$$

$$L_{po1} = \sqrt[4]{\frac{1200EI_r\delta_h}{P_1}} \qquad (11.60)$$

According to figure (11.19), in shore areas, the GPD caused by liquefaction, δ_h, is spreaded to seaside, so that this displacement decreases from seaside towards land.

The ground displacement distribution diagram is also triangular for this case.

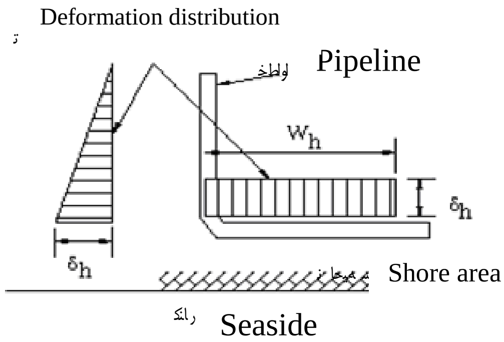


Figure 11.19: Analytic model of curve in liquefied shore area

11-3-4-Design of branches with T-shaped connection

In T-shaped connection of pipe, the bending strain is calculated as follows:

$$\begin{aligned} \varepsilon_T &= \beta_T \Delta & \beta_T\Delta &\le 1.27\varepsilon_y \\ \varepsilon_T &= 2\beta_T \Delta & \beta_T\Delta &> 1.27\varepsilon_y \end{aligned} \quad (11.61)$$

β_T: Conversion factor of T-shaped connection
ε_T: structural strain of T-shaped connection
ε_y: yield strain of junction pipes connected to main pipes
Δ: relative displacement between free area and straight pipe connected to T-shaped connection
The conversion factor is presented as follows:

$$\beta_T = \frac{4\bar{\lambda}_1^{2} D_1 A_2 (C-1)}{4A_2 + LI_1\bar{\lambda}_1^{3} C} \quad (11.62)$$

$$C = \frac{1+4\left(\frac{\bar{\lambda}_1}{\bar{\lambda}_2}\right)^3\left(\frac{D_2}{D_1}\right)}{1+2\left(\frac{\bar{\lambda}_1}{\bar{\lambda}_2}\right)^3\left(\frac{D_2}{D_1}\right)} \quad (11.63)$$

$$\lambda_i = \sqrt[4]{\frac{K_2}{4EI_i}} \quad i=1,2 \quad (11.64)$$

D_i, A_i, and I_i are, respectively, the diameter, area of cross-section, and inertia moment of junction pipe (i = 1) and main pipe (i = 2).
K_2: surrounding soil's resilience modulus for soil's vertical reaction (N/cm^2)

11-3-5-Riser pipe's design

Risers have sufficient strength against vibrations caused by earthquake and do not need seismic design.
The damage mode of risers in earthquake is leakage due to wall or building collapses on them.
If the building moves in the horizontal direction, there would be bending deformation in the riser's pipe connected to it.
I order to prevent from leakage and breakage of riser's pipe, it should be prevented from large deformations.
In order to control the gas flow in every states, it is mandatory to comply with section 17 of national building regulations and safety of risers and other parts of internal piping.

The bending rotation angle could be calculated as follows:

$$\beta = \arctan\left(\frac{\Delta u}{h}\right) \qquad (11.65)$$

Δu and h are horizontal displacement and riser's pipe length between the fixed point and the upper point in figure (11.20), respectively.

The allowable rotation angle, β_α, is given equal to ½ of empirical value, i.e. about 10 degrees.

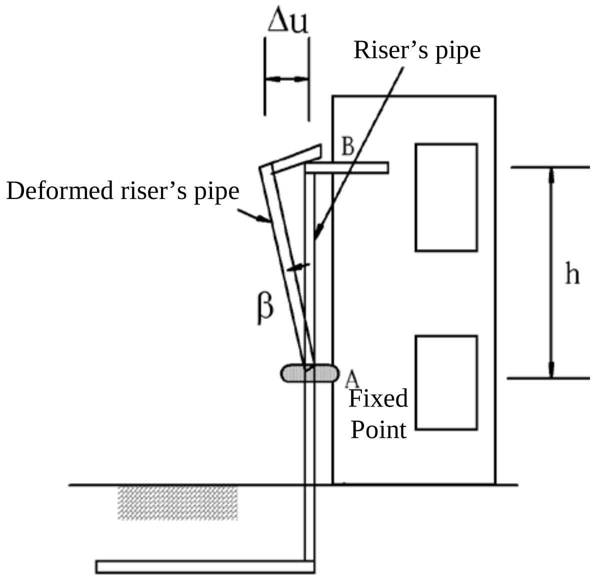


Figure 11.20: Riser's pipe

The relative displacement, Δu, could be obtained from the relative displacement between points A and B in figure (11.20).

11-4-Amount of allowable stress in pipelines

In this section, the allowable stresses for mechanical design against different loads and combination with earthquake for on-ground and buried pipes are presented. These values, in regards to elastic state, are as follows:

1) Ring stress caused by operation pressure

Table 11.5: Ring stress

Class Location	Allowable Stress
1	$0.72\ F_Y$
2	$0.60\ F_Y$
3	$0.50\ F_Y$
4	$0.40\ F_Y$

2) Combination of longitudinal and torsion stresses in on-ground pipes (ANSI B31.8)

a) The expansion stress caused by thermal changes (the combination of longitudinal bending stress with torsion stress) $0.72F_Y$

b) The expansion stress caused by thermal changes, membrane stress caused by internal pressure, and longitudinal bending stress caused by operation's dead weight, snow or ice load and wind load, $1.00F_Y$

c) The longitudinal membrane stress caused by internal pressure and longitudinal bending stress caused by operation's dead weight, snow or ice and wind load, $0.72F_Y$

3) The longitudinal bending stress and combined membrane stress in on-ground pipe

a) Stress caused by internal pressure, thermal changes, operation's dead weight, snow or ice load, $0.90F_Y$

b) Stress caused by internal pressure, thermal changes, operation's dead weight, and maximum hazard level-1 earthquake, $1.00F_Y$

c) Stress caused by internal pressure, thermal changes, operation's dead weight, and maximum hazard level-2 earthquake, $1.10F_Y$

d) Stress caused by experimental pressure, experimental thermal changes, experimental dead weight, $1.10F_Y$

4) Combined membrane allowable stress in buried pipe

a) Stress caused by internal pressure and thermal changes, $0.90F_Y$

b)Stress caused by internal pressure, thermal changes, and maximum hazard level-1 earthquake, $1.00F_Y$

c) Stress caused by internal pressure, thermal changes, and maximum hazard level-2 earthquake, $1.10F_Y$

5) Combined allowable bending and membrane stress in buried pipes:

a) Stress caused by internal pressure, thermal changes, and circumferential bending stress caused by overburden or longitudinal bending stress caused by buoyancy, $1.00F_Y$

b) Stress caused by internal pressure, thermal changes, maximum earthquake, and circumferential bending stress caused by overburden or longitudinal bending stress caused by buoyancy, $1.15F_Y$

c) Maximum effective stress caused by experimental pressure, experimental thermal changes, and circumferential bending stress caused by overburden layer and wheel load, $1.10F_Y$

d) Circumferential bending stress caused by overburden layer and ring membrane stress caused by design's pressure or zero pressure, $0.90F_Y$

e) Circumferential bending stress caused by overburden layer, wheel load, and circumferential membrane stress caused by design's pressure or zero pressure (except for outdoor intersections with roads), $0.80F_Y$

F_Y is the materials' yield strength.

11-5-Acceptance strain criteria proportionate with seismic damage modes

11-5-1-High-pressure pipelines

According to objectives of buried pipes design, the maximum allowable strain or its equivalent value is allocated to pipeline.

In hazard level-1 earthquake, the critical strain in failure mode caused by fatigue in low cycles due to the wave propagation is equal to at least two values of 1% and buckling threshold strain, 35t/D (D is the external diameter of pipe and t is the pipe's thickness).
Also, the critical values for hazard level-2 earthquake are presented in table (11.6).

Table 11.6: Critical values for hazard level-2 earthquake

Seismic load	Component	Failure mode	Unit	Criterion
Wave effects	Pipe curve Straight pipe T-shaped junction	Failure due to fatigue with low cycles	Strain	3%
Permanent Ground Displacement (PGD)				
Liquefaction	Straight pipe	Local buckling	Rotation angle	W_{SC}
	Pipe's curve	Local buckling	Rotation angle	W_{bC}
Intersection with fault	Straight pipe	Tension	Strain	ε_{cr}^{PGD}
Landslip	Straight pipe	Tension	Strain	ε_{cr}^{PGD}

W_{sc}: critical rotation angle caused by bending (deg) in straight pipe which is as follows.

$$w_{sc} = \left\{ 0.44 \frac{t_s}{D_i} \left(8k - \frac{2k^2}{3} \right) + \frac{3.44}{\sqrt{\frac{2D_i}{t_s}}} \left(1 + \frac{\varepsilon_f}{2} \right) \right\} \cdot \frac{180}{\pi} \tag{11.66}$$

W_{bc}: bending critical angle (deg) in curve which is as follows:
W_{bc} is for internal bending and W_{boc} is for external bending.

$$w_{bsc} = 0.9 \frac{\sqrt{D_i / t_b} \cdot \sqrt{\phi_b}}{R_c / D_i} + w_{sc1} \quad , \quad w_{boc} = 2.24 \frac{\phi_b}{\sqrt{D_i / t_b} \cdot \left(\frac{R_c}{D_i} \right)^{0.25} \cdot \eta} \tag{11.67}$$

k: ratio of $L_s/2$ to D_i
t_s: thickness of straight pipe
t_b: thickness of curve
ϕ: curve angle
R_c: curvature radius
ε_f: ultimate strain, 0.35
w_{scl}: critical rotation angle caused by bending (deg) in straight pipe for k = 1
η: parameter related to materials which is presented in table (11.7)

table 11.7: Parameter related to materials, η

Parameter η	Pipe materials based on API 5L
0.77	X42
0.81	X46
0.87	X52
0.87	X56
0.88	X60
0.93	X65

ε_{cr}^{PGD} : Critical strain, 0.35

11-5-2-Seismic design criterion in medium and low pressure pipelines

The critical strain (ε) and the equivalent elastic modulus $(\overline{E})$ in medium and low pressure pipes is as follows.

Table 11.8: Critical strain and equivalent elastic rigidity

Pipe	Critical strain %	Equivalent elastic modulus N/cm²	Strain hardening factor
Steel	3	$\frac{1}{7}E$	7.0×10^{-3}
Soft cast-iron	2	$\frac{1}{5}E$	--
Polyethylene	20	$\frac{1}{2}E$	--

If it is impossible to apply the equivalent elastic modulus, the following moduli of elasticity should be considered:

For steel pipes: $E = 2.06 \times 10^7$ (N/cm^2)

For soft cast-iron pipes: $E = 1.57 \times 10^7$ (N/cm^2)

The hypotenuse elastic modulus could be obtained from the modulus of elasticity, E, and strain hardening factor, λ, as in equation (11.68):

$$E_2 = \lambda E \tag{11.68}$$

The equivalent elastic modulus is presented to determine the non-elastic response of medium and low pressure pipes.

Using this equivalent elastic modulus, the non-elastic response could be obtained using the simplified elastic relation.

(1) Critical strain

The critical strain of steel is considered equal to 3%.

The selected value of critical strain for polyethylene pipes is 20%.

(2) Equivalent elastic modulus

For steel pipes:
$$\overline{E} = \frac{1}{7}E \approx 2.94 \times 10^6 \left(N/cm^2\right)$$

For polyethylene pipes:
$$\overline{E} = \frac{1}{2}E \approx 2.94 \times 10^4 \left(N/cm^2\right)$$

11-5-3-Critical angles in riser pipes

The critical angles in riser pipes should according to the following values:

For hazard level-1 earthquake β_{cr}^{MOE}= 10 (deg)

For hazard level-2 earthquake β_{cr}^{MCE}= 20 (deg)

Appendix

1-Flowchart of allowable span in seismic design of piping

When importance factor of structure is intermediate or low, allowable span method is used and if importance factor is very high or high, allowable stress must be used.

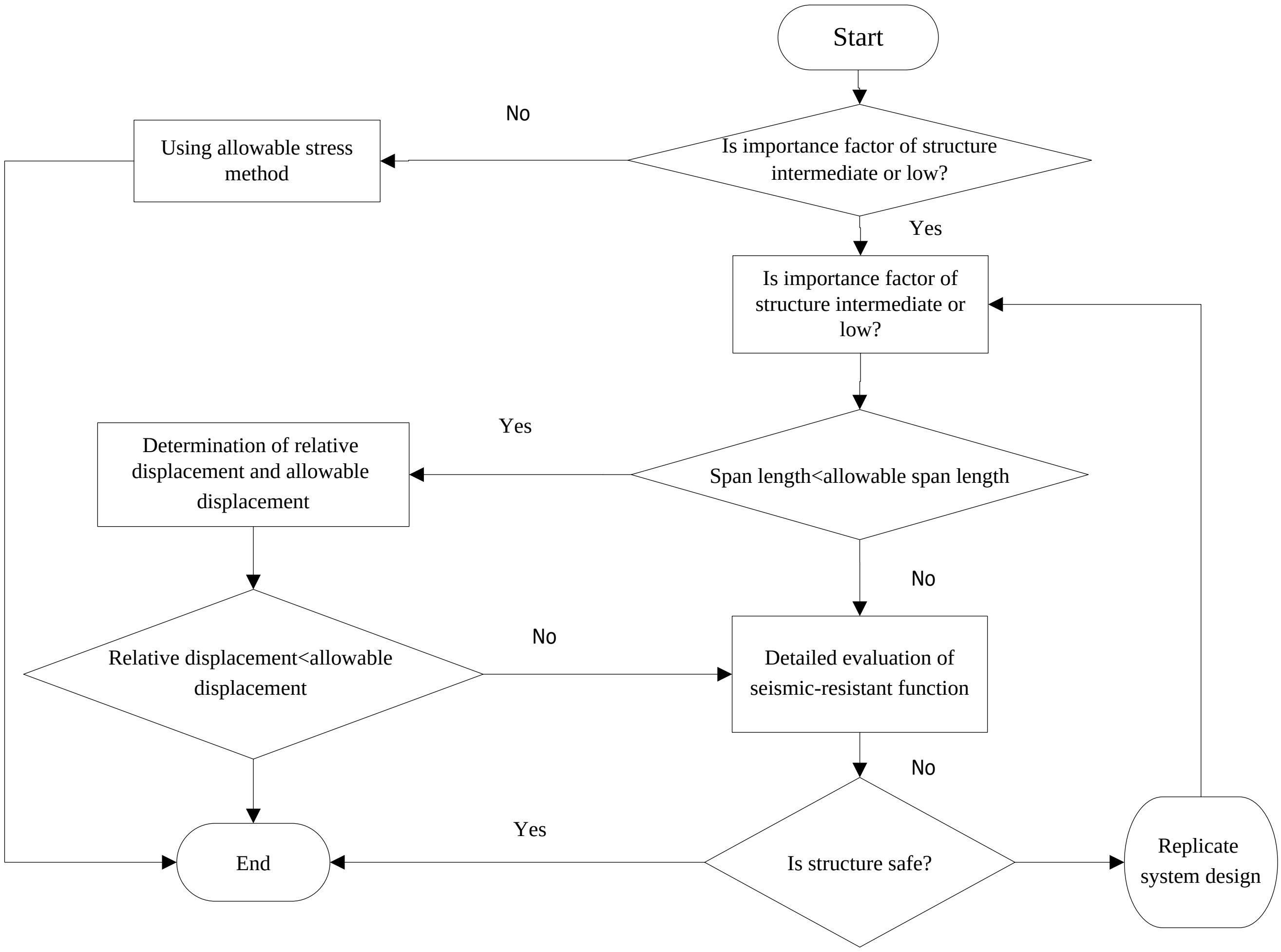


Figure 1-Flowchart of allowable span

1-1-Method of span length calculation

Pipe span length is computed with following general method

1-Each pipe span is evaluated in two horizontal directions and one vertical direction of earthquake

2-For Evaluation in a direction, pipe span length between two support point is equal to sum of pipe projections between those two support points in the direction perpendicular to earthquake.

3-in section 2, if pipe axis direction is in the main direction of earthquake, calculations don't be done in other direction.

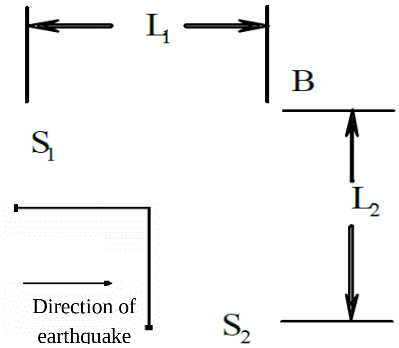


Figure 2-pipe span with support point in the direction of pipe axis

In the example of figure 2, S_1 and S_2 are support points. Since pipe axis of S1-B is coordinated with direction of earthquake so it isn't required to L_1 be added to length of pipe span, so $L_{PS} = L_2$.

4-In the case that external diameter of pipe is various in the pipe span, span length of L is computed using equation (1) and converted into maximum diameter of pipe span.

$$L = \ell_p + \ell_1 \sqrt{\frac{d}{d_1}} \tag{1}$$

Where

L	length of pipe span (m)
d	maximum external diameter of pipe span (mm)
d1	external diameter of pipe (mm)
l_p	length of pipe with external diameter of d (m)
l_1	length of pipe with external diameter of d_1 (m)

5-In the case that the pipe has more than one junction:

Allowable span length between two supports must be obtained. For figure (3) $(L_1 + L_2)$, $(L_1 + L_b)$ and $(L_2 + L_b)$ must be less than allowable span length. In this figure, if the junction diameter is less than half of main pipe diameter, main pipe length must be less than allowable span length.

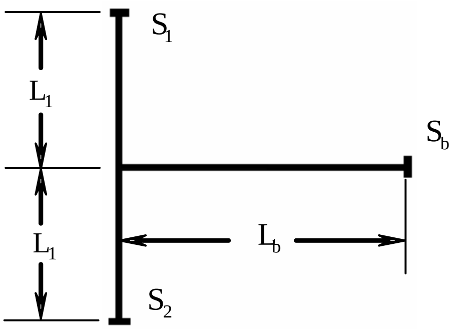


Figure 3-Span of pipe involving junction

1-2-Calculation of allowable span length

Table 1 presents basic allowable span length. If external diameter is not available in the table, it can be derived by interpolation. If external diameter is less than 48.6 mm, equal to 48.6 mm or more than 609.6 mm, it is considered to be 609.6 mm. But it is not applicable if external diameter is more than 1000 mm.

Table 1-Allowable span length of pipe

Standard concentrated weight (compressed gas) W_a (N)	Standard concentrated weight (liquid gas) W_a (N)	basic allowable span length (compressed gas) L_a (m)	basic allowable span length (liquid gas) L_a (m)	External diameter (mm)
304	407	7	6.6	48.6
445	605	7.8	7.1	60.5
839	1116	8.7	7.9	76.3
1126	1545	9.5	8.6	89.1
1414	1986	10.1	9	101.6
1775	2532	10.7	9.5	114.3
2616	3802	11.7	10.2	139.8
3616	5357	12.7	10.8	165.2
6349	9629	14.8	12.2	216.3
9863	15208	16.4	13.2	267.4
14281	22361	18	14.2	318.5
18110	28851	19	15	355.6
25339	40325	20.3	16	406.4
33995	53612	21.5	16.8	457.2
42112	67633	22.7	17.8	5.8
51141	83563	23.8	18.4	558.8
64243	103946	24.9	19.1	6.9.6

In the case that broad weight of heat-insult ant material or concentrated weight of valve is added, allowable span length is computed from multiplication of basic allowable span length, presented in tables 1 and 2, in broad weight correction factor ϕ_d and concentrated weight correction factor ϕ_c.

1-when the road weight of heat-insultant material is added, broad weight correction factor ϕ_d is computed from formula (2):

$$\phi_d = \left(1 + \frac{\Gamma}{\Gamma_p}\right)^{-0.25} \qquad (2)$$

Where

ϕ_d broad weight correction factor, when $\phi_d = 1.0$, $\Gamma / \Gamma_p \leq 0.5$

Γ_p Sum of longitudinal weight of pipe and content weight in each meter (N/m)

Γ Broad weight of heat-insultant material in one meter of pipe length (N/m)

2-when the concentrated weight of valves is added, concentrated weight correction factor ϕ_c is computed based on table 3. Overweight rate γ_w is computed from formula (3)

$$\gamma_w = \frac{w_s}{W_a}\left(1+\frac{\Gamma}{\Gamma_p}\right)^{-\frac{3}{4}} \tag{3}$$

Where

γ_w overweight rate

w_s concentrated weight of span (N)

W_a standard concentrated weight (N) for liquid and concentrated gas piping indicated in table (1)

Table 3-concentrated weight correction factor

Overweight rate limit γ_w	Concentrated weight correction limit ϕ_c
$\gamma_w \le 0.25$	$\phi_C = 1$
$0.25 < \gamma_w \le 1$	$\phi_C = 1.13 - 0.53\gamma_w$
$1 < \gamma_w$	$\phi_C = 0.636 - 0.036\gamma_w$

1-3-Calculation of piping displacement capacity

1-Piping displacement capacity

Piping span displacement capacity is computed from equation (4). Support displacement must be lesser than allowable displacement capacity (Δ)

$$\delta_a = L_{PS} \cdot f_p \tag{4}$$

Where

L_{PS} : allowable span length (mm)

δ_a : piping displacement capacity in the direction of earthquake (mm)

f_p : displacement capacity of piping length in each millimeter that its value obtained from equation (5)

$$f_p = C \cdot \varepsilon_y \cdot L_{PS} / d \tag{5}$$

Where

C displacement constant of allowable piping span that is equal to 0.67

d maximum external diameter of pipe span (mm)

ε_y least value of yield strain for design temperature and normal temperature of piping material which is obtained from equation (6)

$$\varepsilon_y = \frac{S_y}{E} \tag{6}$$

Where

S_y yield strength or 0.2% strength in design temperature of material (N/mm^2)

E Yang module in temperature of material (N/mm[2])

2-Capacity of expansional connection displacement

Capacity of pipe span displacement with corresponding expansional connection with mentioned allowable strain in expansional connection specifications.

3-For figure 3, control is as following:

3-1-when external diameter of junction is more than half of main pipe diameter.

Displacement capacity of Figure length $L_j(12) = (L_1 + L_2)$, $L_j(1b) = (L_1 + L_b)$ and $L_j(2b) = (L_2 + L_b)$ is assumed $\delta_a(12)$, $\delta_a(1b)$ and $\delta_a(2b)$ respectively and relative displacement of support point $S_1 - S_2$, $S_1 - S_2$ and $S_2 - S_b$ is assumed $\Delta(12)$, $\Delta(1b)$ and $\Delta(2b)$ respectively. Then evaluation of displacement capacity is performed through confirmation of $\Delta(12) \le \delta_a(12)$, $\Delta(1b) \le \delta_a(1b)$ and $\Delta(2b) \le \delta_a(2b)$. It is assumed in this figure that earthquake direction must be perpendicular to paper direction and S_1, S_2 and S_3 are support points which halter the direction perpendicular to paper.

3-2-When external diameter of junction is equal or lesser than the half of main pipe diameter. Evaluation of displacement capacity is performed through confirmation of following equations.

$$\frac{\Delta(1b) + \Delta(2b)}{2} + 20\left(\frac{L_{12}}{L_{PS}}\right) \le \delta_a(T_b) \text{ و } \Delta(12) \le \delta_a \qquad (7)$$

It is assumed that $\delta_a(T_b)$ is displacement capacity of L_{PS}, L_{PS} is allowable span and L_{12} is the length of pipe span.

1-4-Calculation of relative displacement

1-Relative displacement

For evaluation of capacity of pipe span displacement, it is assumed that displacement in the direction of earthquake is equal to the displacement of pipe support structure that is obtained by following method based on height of support point.

Relative displacement of piping span Δ is obtained from equation (8).

$$\Delta = \delta_1 + \delta_2 \qquad (8)$$

Where

Δ	relative displacement of piping span
δ_1	displacement of support point 1 in earthquake (mm)
δ_2	displacement of support point 2 in earthquake (mm)

2- Calculation steps of displacement piping support structure in earthquake

Calculation steps are given in figure 4

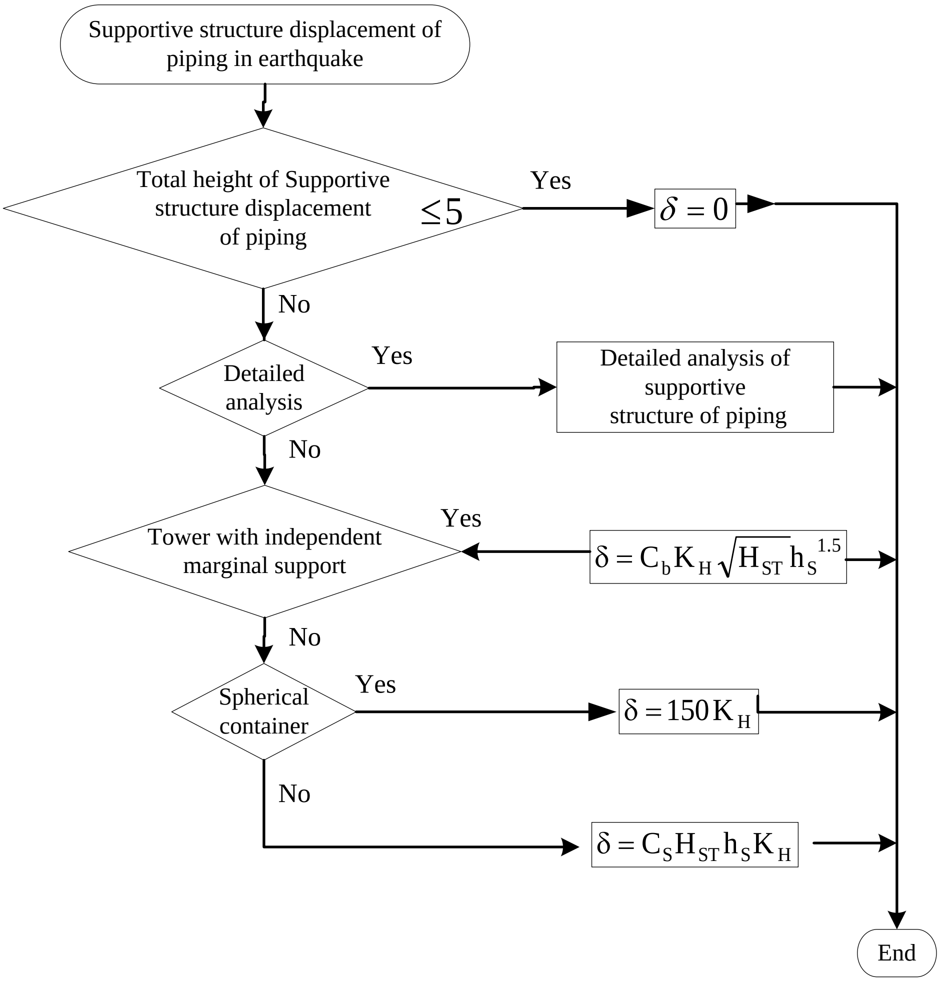


Figure 4-calculation steps of support structure displacement of piping in earthquake

As indicated in the displacement evaluation, if support structure of piping is equal or less than 5m, that evaluation can be neglected.

1-Support displacement of tower piping with independent marginal support in any height in earthquake can be computed in terms of millimeter through equation (9).

$$\delta = C_b K_H \sqrt{H_{st}} \cdot h_s^{1.5} \tag{9}$$

Where

K_H Horizontal intensity of earthquake in ground level with consideration of importance level of piping system

C_b 1

H_{st} Total height of support structure of piping (m)

h_s Height of support point of piping (m)

2-Spherical container displacement in earthquake is computed using equation (10)

$$\delta = 150 K_H \tag{10}$$

Where

K_H Horizontal seismic intensity in the ground level with consideration of importance level of piping system.

3-support displacement of piping of other support structures in earthquake in any height is given by equation (11):

$$\delta = C_S \cdot K_H H_{st} \cdot h_s \qquad (11)$$

Where

K_H Horizontal seismic intensity in the ground level related to importance level of piping system.

C_S 0.7

2-Standard structure of seismic design of piping system

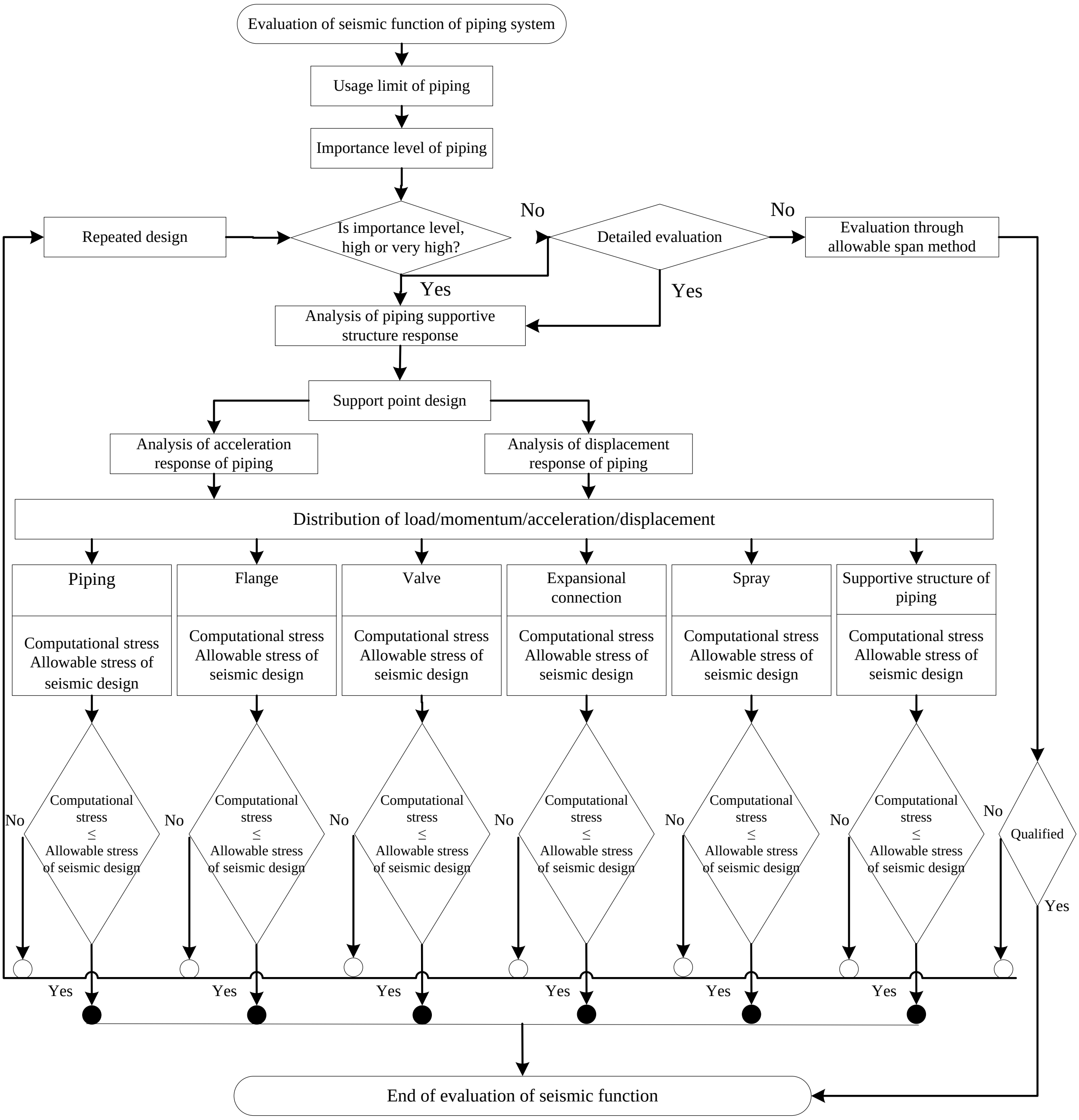


Figure 5-Standard structure of seismic design for piping system

3-Analysis of structure response for piping support

Figure 6 shows the analysis of structure response for piping support

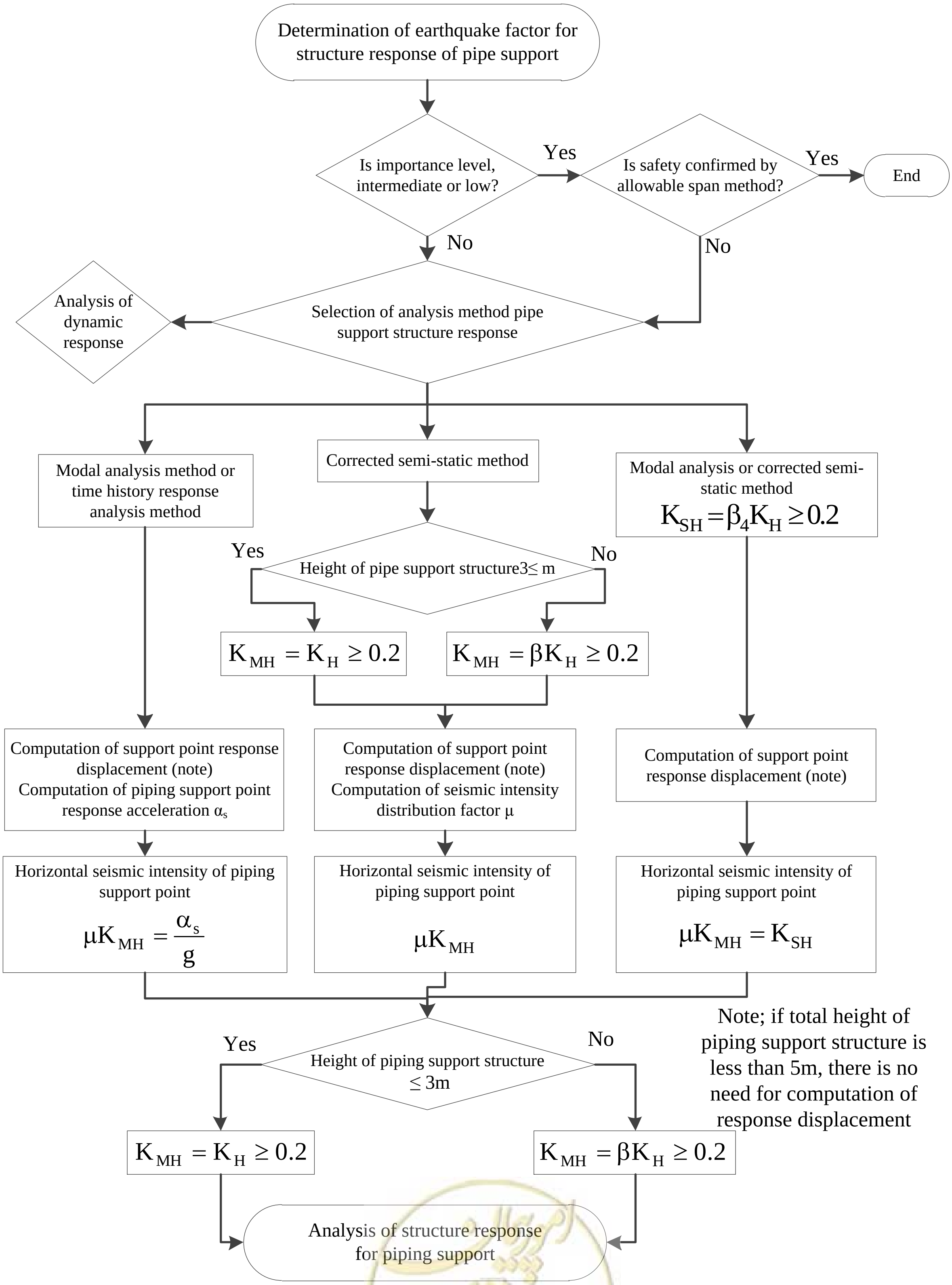


Figure 6-Steps of designing piping support structure response

1-Importance level of piping support structure

In analysis of piping support structure response, importance level is computed on the basis of importance level of related piping, acceleration and displacement of response in the piping support point. Evaluation of seismic function is performed according to maximum importance level among importance level of equipment and piping related to piping support structure.

2-Amplification factor of horizontal response through semi-static method

Table 4 presents amplification factor of horizontal response β_4 of piping support structure designed through semi-static method based on height H from the ground level.

Table 4-Amplification factor of horizontal response β_4

H(m)	β_4
$H(m) \leq 16$	2.0
$16 < H(m) < 35$	$1.04 + 0.06H$
$H(m) \geq 35$	3.14

3-Amplification factor of horizontal response through semi-corrected method

Amplification factor of horizontal response of piping support structure designed through semi-corrected method is obtained from multiplication of amplification factor of standard response in correction factor. Amplification factor of standard response is based on the natural period and type of ground in the location of piping support structure and correction factor on the basis of decay factor of piping support structure[1].

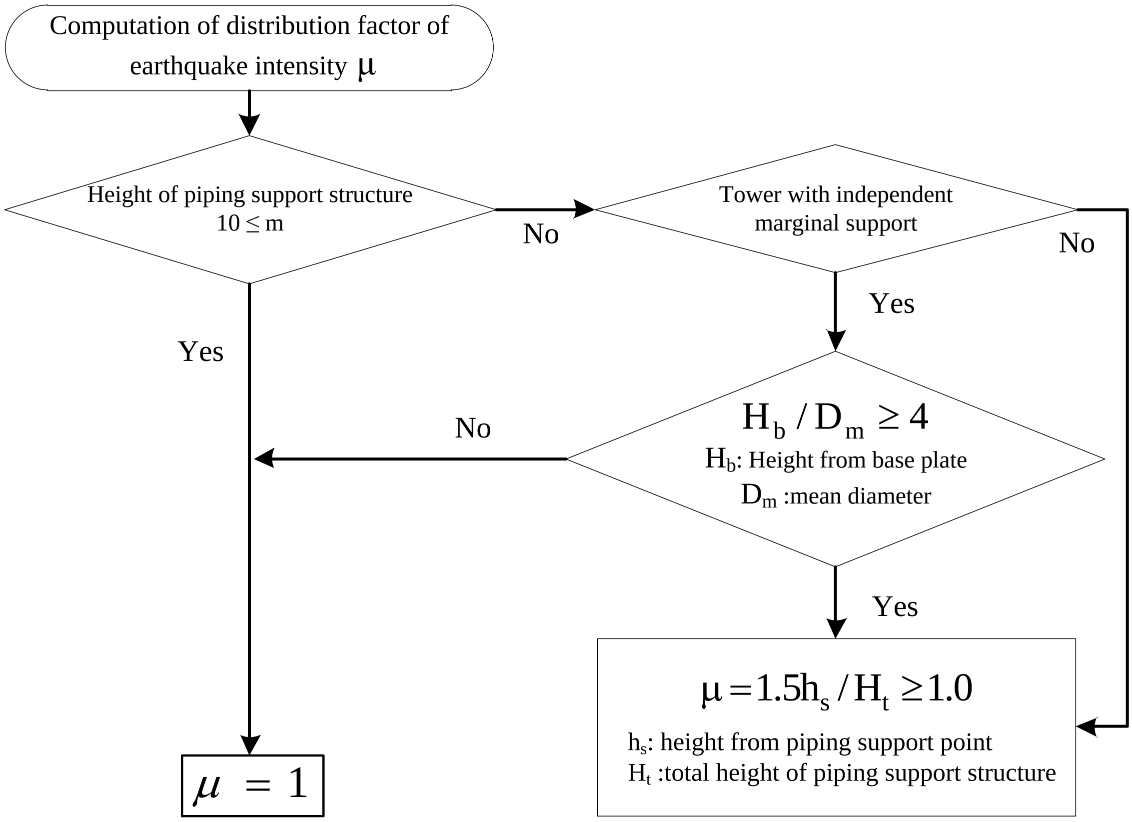
4-Amplification factor of vertical response

Amplification factor of vertical response in the tower with independent marginal support is to be 1.5 and in other piping support structures is to be 2.

5-Distribution factor of seismic intensity

When analysis of piping support structure response id performed via corrected-semi static method, distribution factor of seismic intensity μ is computed on the basis of type and height of piping support structure, as is shown in the figure 7.

1 - these are given in the guideline for loading and seismic analysis of lifelines



For support piping in spherical container shell, because all of the shell is displaced.
h_s:height of shell center (mm)
H_t: height of upper crest of spherical shell (mm)
When μ<1.0. then μ=1

Figure 7-seismic intensity distribution in the case that piping support structure is analysed via corrected earthquake factor method

4-Analytical model and corrected seismic force for designing piping system

Analytical model of piping system for response analyse of acceleration and displacement is developed according to following rules:

1- Analytical model of piping support structure is developed on the basis of evolution step of seismic function of towers, containers and framed structures.
2- In analytical model of piping, direct pipe is considered as beam element and curved pipe is considered as curved beam element.
3- In total, analytical modeling of piping is performed between fixed points. However, from mechanical point of view, modeling is not confined to these points.
4- Solidity of piping beam element is computed by reduction of allowable value of corrosion from nominal dimensions.
5- Allowable value of corrosion is considered for calculation of weight.
6- Allowable value of corrosion is considered for calculation of piping tension.

5-calculation of piping tension

1-load composition

Piping load compositions in table (5) is used for evolution of seismic function.

Table 5-load composition in seismic designing of piping

Load type / Tension type	Earthquake force		Stimulant weight	Fluid pressure
	Relative displacement	**Inertia force**		
		○	○	○
	○ Supportive structure	○		

2-flexibility factor and stress intensification factor

Flexibility factor and stress intensification factor for calculation of longitudinal stress of piping and alternative stress are obtained from table (6). However, if data be available, modeling is not confined to these points.

3-combination of stresses of earthquake in different direction

In evaluation of commutative stress, the most unfavorable direction of earthquake is used. If determination of this direction is difficult, two horizontal directions are applied independently. For combination of horizontal and vertical direction of earthquake, stress in two horizontal directions and two vertical directions are computed and momentum and axial force are obtained from sum of absolute value.

4-relative displacement

When reliance points of piping are located in various supports, relative displacement values between various reliance points must be computed for vibration in different directions.

5-External force for evaluation of flange and spray of equipment

Sum of absolute value of axial force and momentum of above-explained components of external force are considered in evaluation of flange and spray of equipment

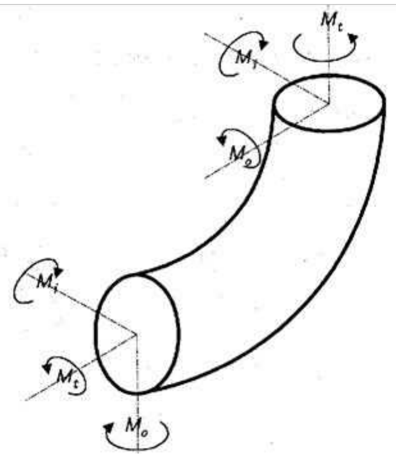
6-Specifications of stress calculation

Longitudinal elastic module in operational temperature is used for calculation of stress. Value of elastic module has been given in the clause of 3-4 in the text. Suitable value of the Poisson ratio is 0.3.

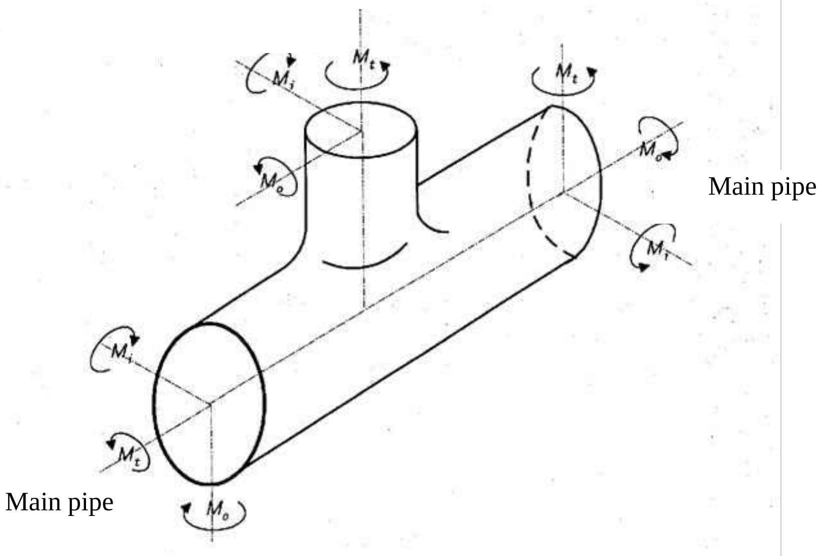
7-Seismic intensity distribution in piping height

in the case of changing horizontally corrected seismic intensity in reliance point of piping in height, linear distribution is convenient.

Nearly mean horizontally corrected seismic intensity is considered in design of two reliance points. If its distribution is biased to one direction, seismic intensity distribution requires more accuracy.



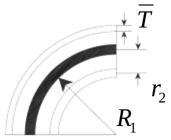
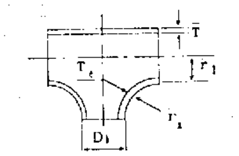
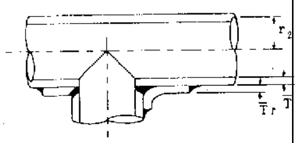
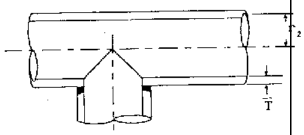
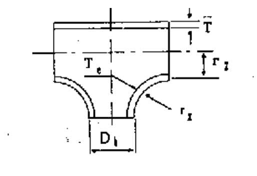
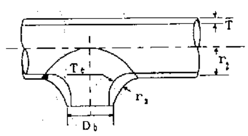
Appied momentum to curved pipe

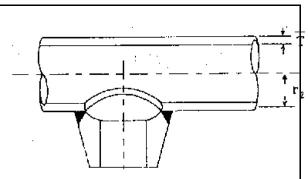


Appied momentum to junction pipe

Figure 8-Definition of momentum

Table 6-Flexibility factor and factor of stress intensification

Connection type	**flexibility factor** k	**factor of stress intensification (3)(2)**		**Characteristic value of flexibility**	**Simplified design**
		Inter-planar i_i	**Extra-planar** i_o		
Welded elbow or bending (2) (4) (5) (6)(7) of pipe	$\frac{1.65}{h}$	$\frac{0.9}{h^{2/3}}$	$\frac{0.75}{h^{2/3}}$	$\frac{\overline{T}R_1}{r_2^{\,2}}$	
ASMEB16 and 9. Welded T shape of pipe $r_x \geq 1/8D_b$ $T_e \geq 1.5\overline{T}$ (2)(4) (6)(11)(13)	1	$\frac{3}{4}i_o + \frac{1}{4}$	$\frac{0.9}{h^{2/3}}$	$3.1\frac{\overline{T}}{r_2}$	
Reinforced shape with T pipe, sheet or saddle (2)(4) (8)(12)(13)	1	$\frac{3}{4}i_o + \frac{1}{4}$	$\frac{0.9}{h^{2/3}}$	$\frac{\left(\overline{T}+1/2T_r\right)^{2.5}}{\overline{T}^{1.5}r_2}$	
Non-reinforced T-shape of pipe (2)(4) (12)(13)	1	$\frac{3}{4}i_o + \frac{1}{4}$	$\frac{0.9}{h^{2/3}}$	$\frac{\overline{T}}{r_2}$	
Welded t-shape of exit pipe $r_x \geq 0.05D_b$ $T_e < 1.5\overline{T}$ (2)(4) (13)	1	$\frac{3}{4}i_o + \frac{1}{4}$	$\frac{0.9}{h^{2/3}}$	$\left(1+\frac{r_x}{r_2}\right)\frac{\overline{T}}{r_2}$	
Junction welded connection $r_x \geq 1/8D_b$ $T_e \geq 1.5T$ (2)(4) (11)(13)	1	$\frac{3}{4}i_o + \frac{1}{4}$	$\frac{0.9}{h^{2/3}}$	$3.1\frac{\overline{T}}{r_2}$	

Welded junction connection (reinforced integrated type) (2)(4) (9)(12)	1	$\frac{0.9}{h^{2/3}}$	$\frac{0.9}{h^{2/3}}$	$3.3\frac{\bar{T}}{r_2}$	

Values in the parenthesis points to these notes:

Table 7- Flexibility factor and factor of stress intensification

Connection type	factor of stress intensification i	Flexibility factor *k*
Lip to lip welding connection type, reducer or end flange of pipe or Lip to lip welding connection type	1.0	1
Pipe flange or two sided weld	1.2	1
Connection or angle weld or pipe or ended flange or bean welding	Note 14	1
Overall flange or connection (JPI-7S-15 or ASMEB16.9 stub end)	1.6	1
Torsion connection or pipe or flange	2.3	1
Undulated direct pipe or curve, note 10	2.5	5

Note:

1-When there is not available data, tables 6 and 7 are used. This table is valid for $D/T \geq 100$

2- Flexibility factor k, mentioned in these tables is used for elbow. Flexibility factor k and factor of stress intensification i is considered equal or more than 1 and in the case of bolt, considered to be equal to 1. Both factors are applied in the bending of the pipe for effective length of curvature (indicated as more bolded line in the figure) and in the T-shaped pipe are applied in the intersection point.

3-For both factors of i_i and i_0, relation $\frac{0.9}{h^{2/3}}$ can be used conservatively.

4- Curvature characteristic value of h is computed for before-mentioned relation and values of k and i can be obtained directly, where:

$\bar{T}$	for elbow, nominal thickness of pipe connection and for T-shaped pipe, nominal thickness of installed pipe (mm)
T_e	angle section thickness of T-shaped pipe (mm)
$\bar{T}_r$	plate or saddle thickness (mm)
r_2	mean radius of installed pipe (mm)
R_1	curvature radius of welded elbow of pipe bending (mm)
R_x	curvature radius on the surface exterior from junction limit on the face that include mother pipe axis and extruded pipe (mm)
D_b	external diameter of extruded pipe (mm)

5-when flange is installed in part of the pipe or in the two ends of it, characteristic value of curvature h is computed and factor value of C_1 is obtained directly (using figure 9) and values of i and h of the table are corrected using this factor.

6-Thickness of pipe connection with lip to lip welding is significantly greater than the thickness of installed pipe. If this thickness isn’t considered, high amount of error may be occurred.
7-Pressure affects very highly on i and k values of elbow and pipe bending with high diameter and thin wall. Correction of the table values is performed through following relations. In two following relations, E is longitudinal elastic module (MPa).

k is divided on $1+6\left(\frac{P}{E}\right)\left(\frac{r_2}{\bar{T}}\right)^{7/3}\left(\frac{R_1}{r_2}\right)^{1/3}$.

i is divided on $1+3.25\left(\frac{P}{E}\right)\left(\frac{r_2}{\bar{T}}\right)^{5/2}\left(\frac{R_1}{r_2}\right)^{2/3}$.

8-For $\bar{T}_r > 1.5\bar{T}$, h value is equal to $h = 4\bar{T}/r_2$
9-a pressure equal to pressure applied on direct pipe is applied on this connection.
10-Both factors are used for bent. Flexibility factor of bolt is assumed to be equal to 0.9.
11-When there is not sufficient data and bent is not in the suitable limit of diameter and thickness, Characteristic value of bent h is equal to $\bar{T}/r_2$.
12-For junction connection with various diameters, where ratio of external diameter of junction pipe to main pipe is in the limit $0.5 < d/D < 1.0$, extra-planar stress intensification factor (SIF) that can be extracted from figure 9, may not be convenient. That indicates that glossy welding reduces SIF, so suitable SIF must be chosen.
13-In stress intensification factor, it is assumed that minimum diameter of the body is twice the diameter of main pipe. It is required that certain considerations must be adopted for input narrow pipe.
14-Maximum up to 2.1 or lesser value of $2.1\bar{T}/C_x$ is considered but this value must be higher than 1.3. Here, C_x indicate welding base of angle. Lower values are assumed for base length.

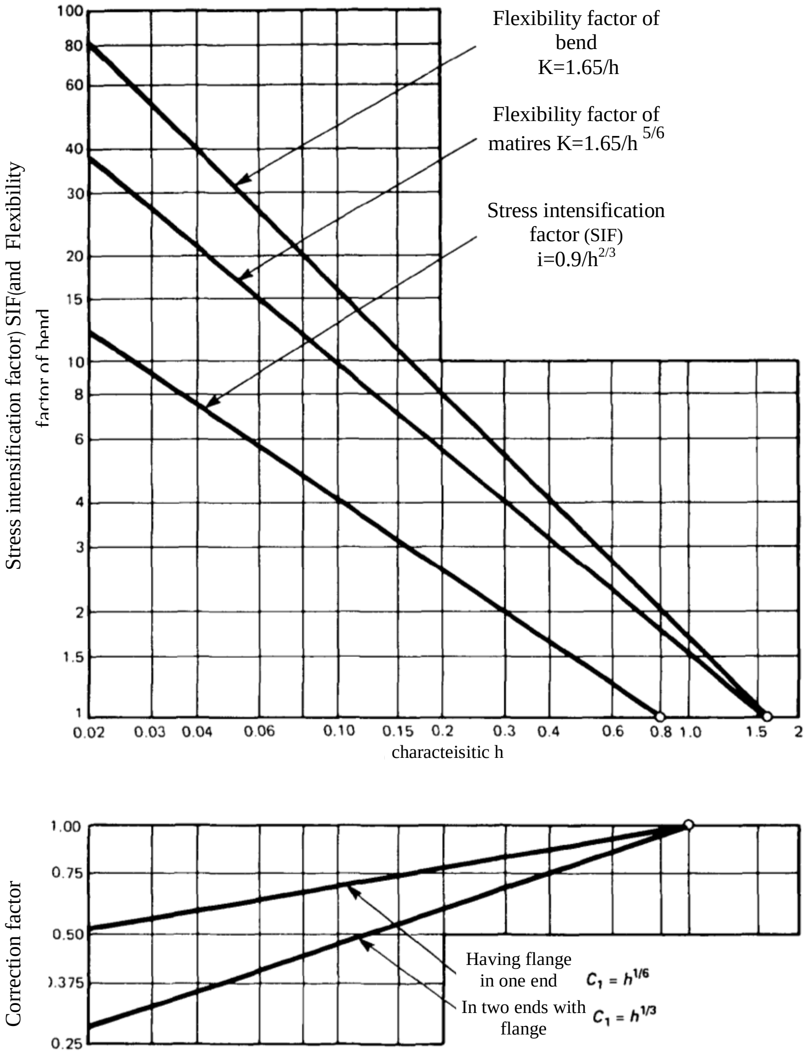


Figure 9-flexibility factor and stress intensification factor (SIF)

6-Allowable stress of piping seismic design

Table (8) presents allowable stress of piping seismic design based on stress type.

Table 8- allowable stress of piping seismic design

Stress type	Allowable stress of seismic design
Longitudinal stress	S
Alternative stress limit	$2S_y$

Where

S allowable stress for seismic design of compressive material (N/mm^2)

S_y yield strength or yield equivalent strength using 0.2% strain of material

Table 9- allowable stress of piping seismic design based on material type

Material type	S
A)material of aluminum alloy and steel material with 9% Nickel for low temperature lower than room temperature	$S = \min\{0.6S_u , 0.9S_y\}$
B) Austenitic stainless steel material and steel material with high alloy of Nickel, used in temperatures higher than room temperature	$S = \min\{0.6S_{u0} , 0.6S_u , 0.9S_{y0} , S_y\}$
C)material beyond that of a) and b)	$S = \min\{0.6S_{u0} , 0.6S_u , 0.9S_{y0} , S_y\}$

Where

S_u and S_{uo} Tensile strength in design temperature and normal temperature material which its value is four times of allowable tensile stress

S_y and S_{yo} yield strength or yield equivalent strength using0.2% strain of material

7-Step of seismic function evaluation of flengic connection

1-Steps of seismic function evaluation

Figure 10 shows Step of seismic function evaluation of flengic connection.

2-Allowable stress of seismic design

Allowable stress of seismic design is determined based on stress type through table (10).

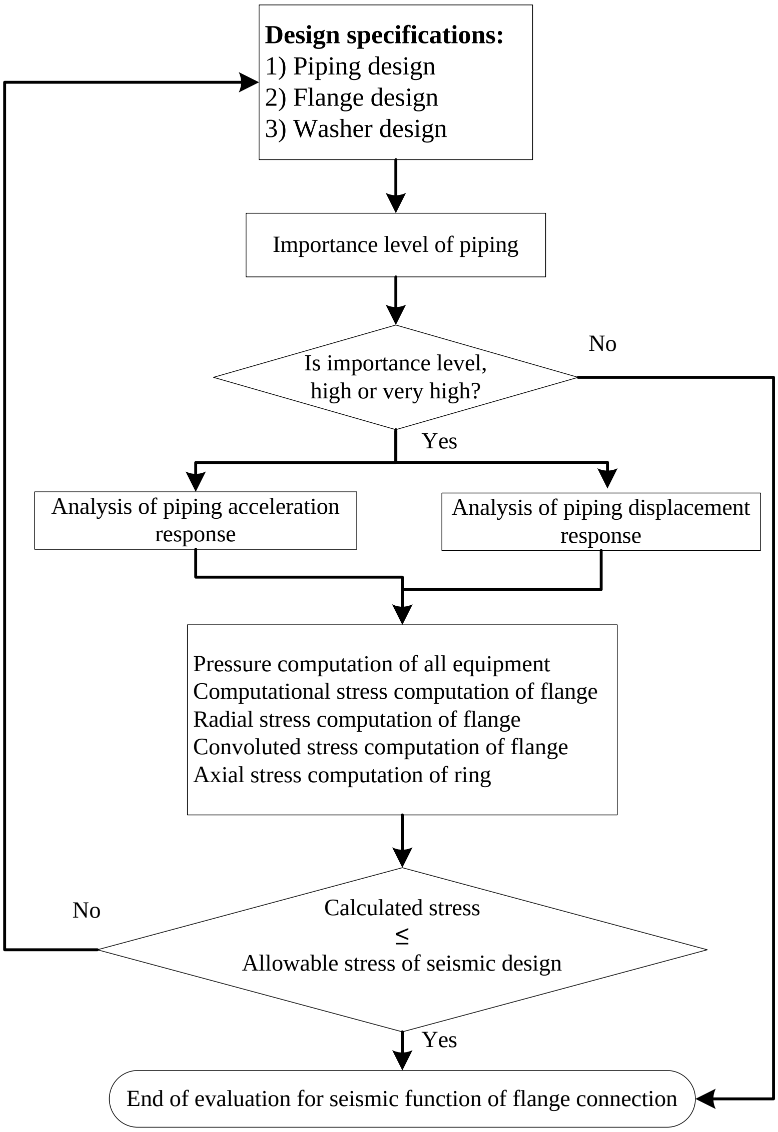


Figure 10-Steps of evaluation for seismic function of flange connection

Table 10-Allowable stress of seismic design of flange connection

Stress type	**Allowable stress of seismic design**
Radial stress of flange	S
Convoluted stress of flange	S
Axial stress of ring	$2S_y$

S and S_y are explained in section 3-4.

3-Stress calculation parameters

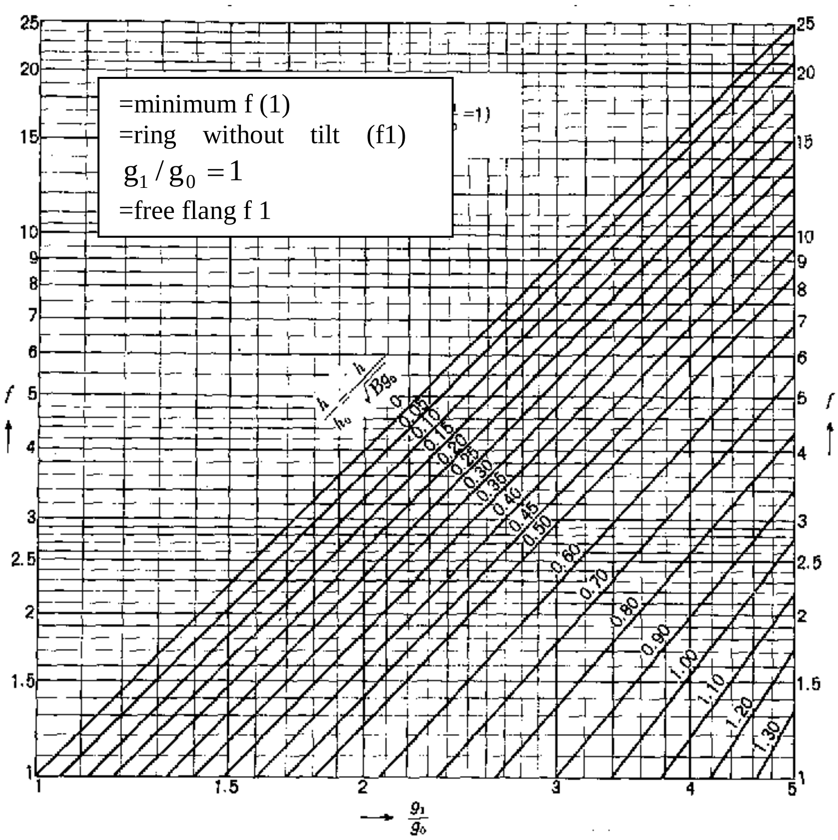


Figure 11-value of parameter f

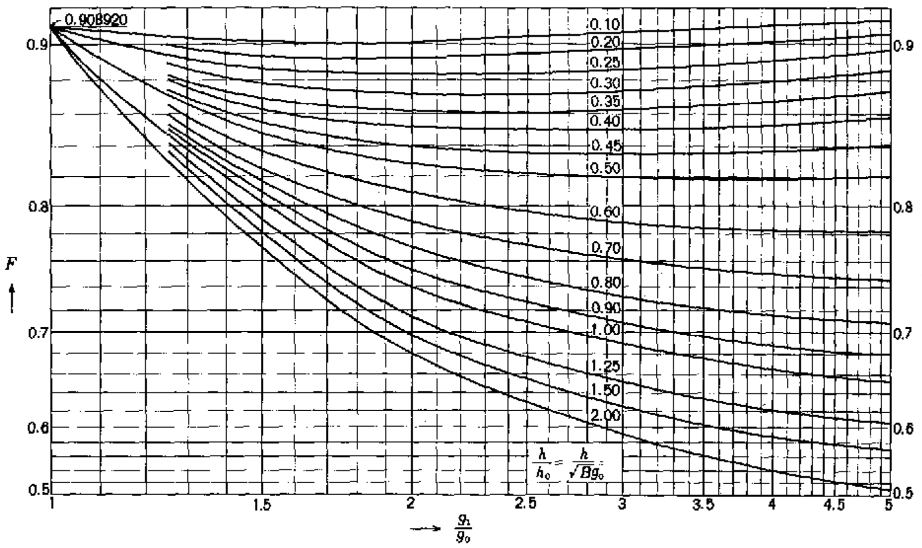


Figure 12-value of parameter F

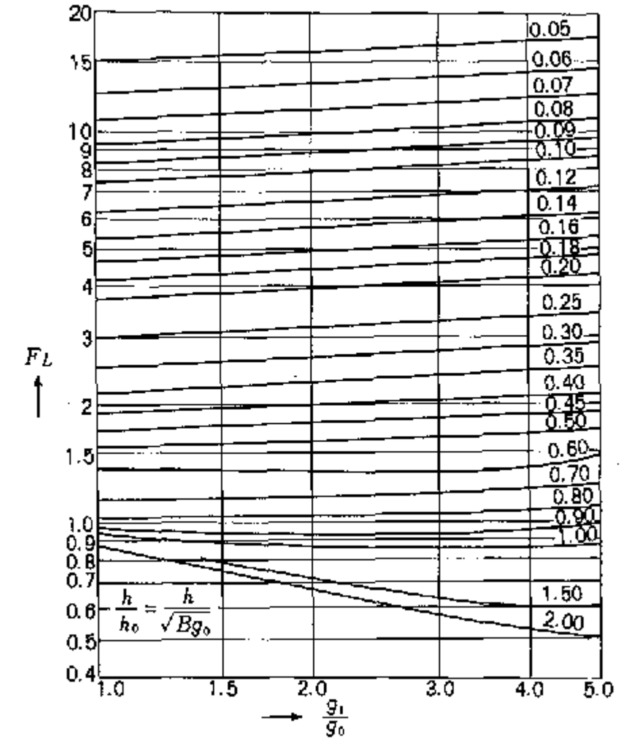


Figure 13-Parameter value of F_L

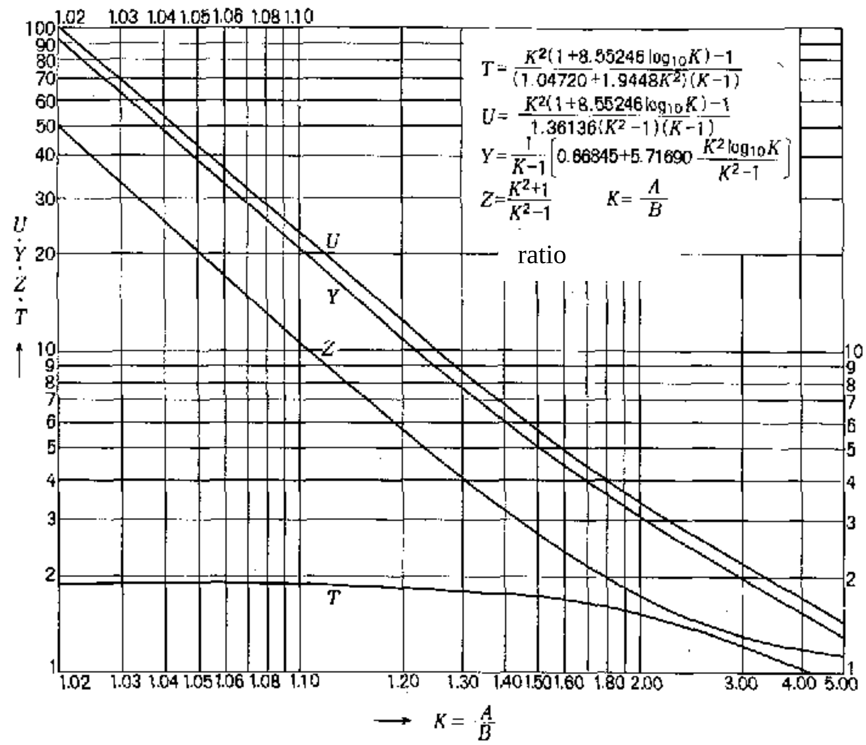


Figure 14-Values of parameters U, Z, Z and T

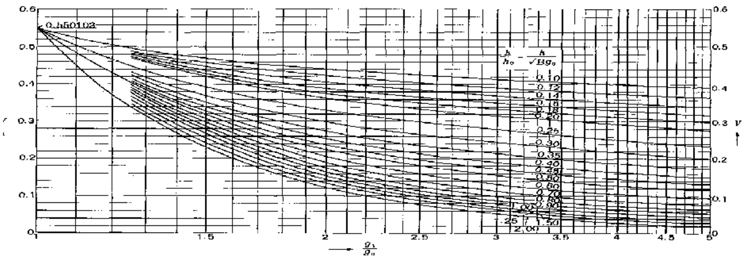


Figure 15-Parameter value of V

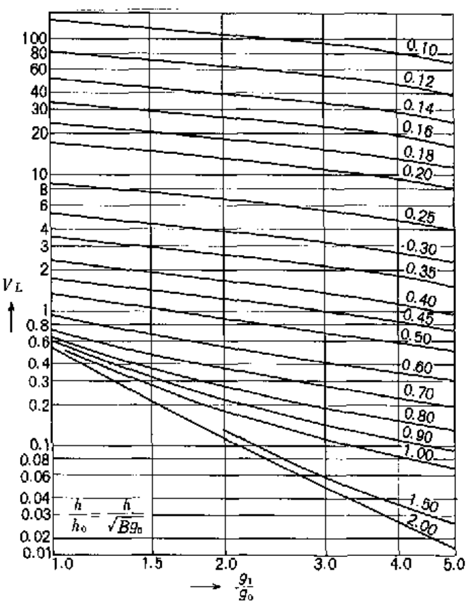


Figure 16-Parameter value of V_L

8-Seismic evaluation of valve

Because of high natural frequency of common values such as handy valves, resonance does not occur in them due to earthquake acceleration. In earthquakes with natural frequency, high inertia force is produced in actuator of high weight and relatively far gravity center from piping valve. So seismic function is evaluated by stress calculation in the weakest part between the main body of valve and weighting parts in the outside of the piping center for intertie force resulted from earthquake such as a valve with natural frequency of lower than 20 Hz. In strength is secured, valve cutoff function will be safe.

1-Steps of seismic function evaluation

Figure 17 shows steps of seismic function evaluation.

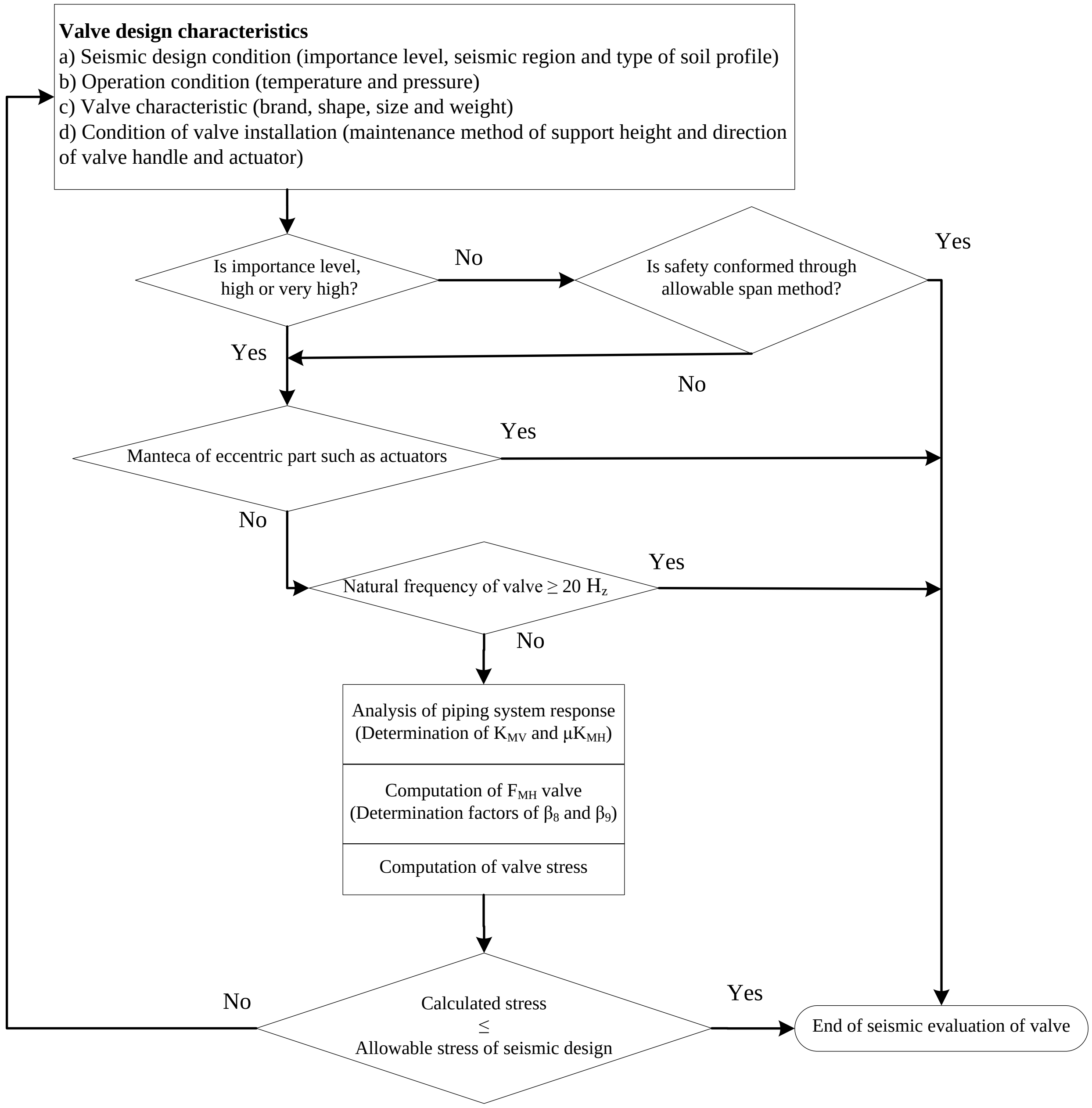


Figure 17-steps of evaluation of valve seismic function

2-Allowable stress of seismic design

Table 11 presents allowable stress of seismic design based on valve type.

Table 11- Allowable stress of valve seismic design

Valve type	Allowable stress of seismic design
Closing valve in time of earthquake	0.5S
Other valves	S

In this table, S is the value that is given in the section 4-3.

9-Method of seismic evaluation for expansional connection

When expansional connection is used for seismic function improvement of piping system, it should be noted that a suitable type of connection is installed in the suitable position and support is in the suitable state. For seismic evaluation for expansional connection, amplitude of maximum axial stress must be lower than total amplitude of allowable stress corresponding to 500 times of numbers of design alternatives.

1-Steps of seismic function evaluation

Figure 18 shows steps of seismic function evaluation for expansional connection

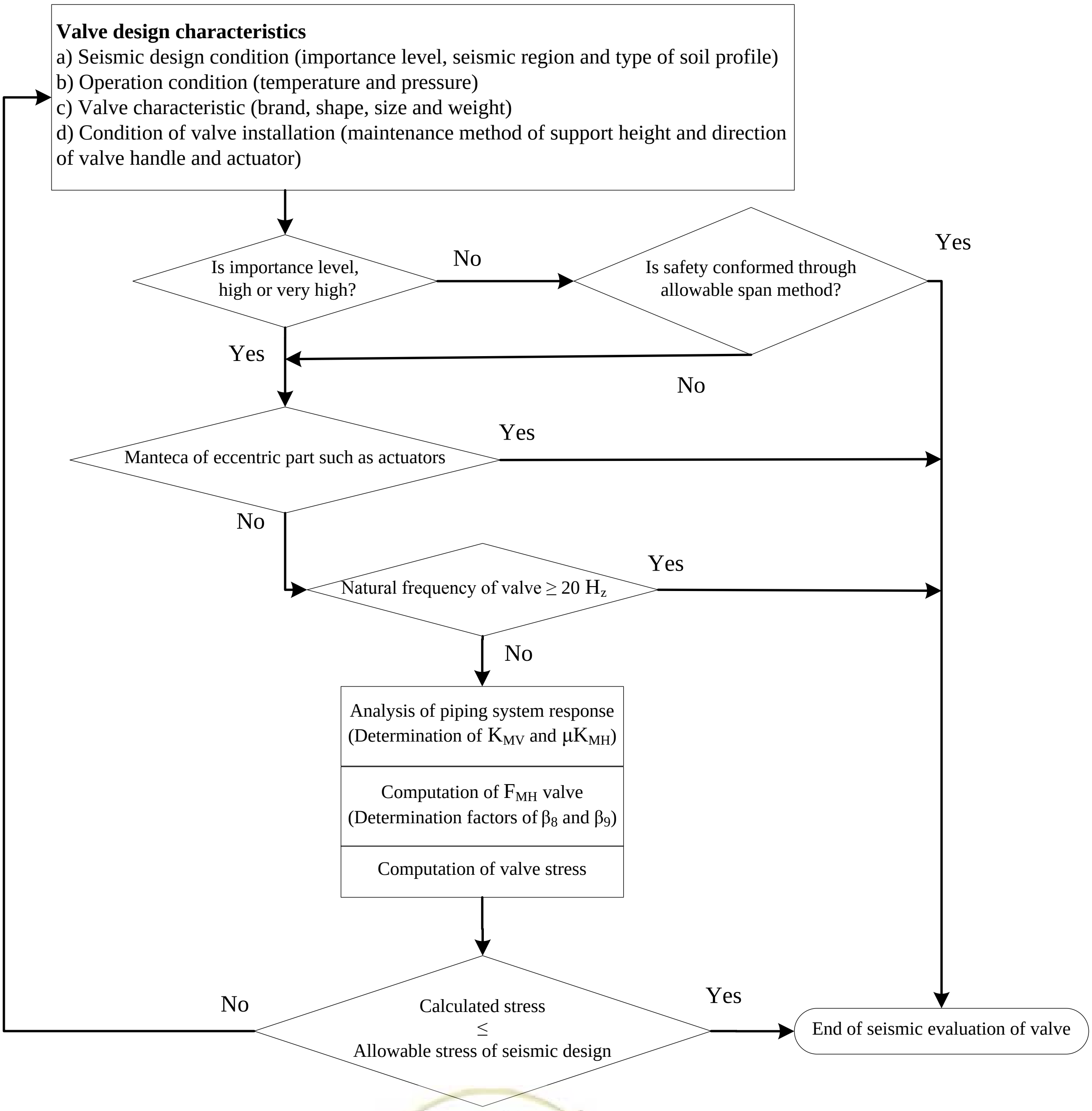


Figure 18- steps of seismic evaluation for expansional connection

2-Allowable stress of seismic design

Allowable stress of seismic design for produced axial stress in expansional connection accordion is equal to following value, because its value is twice of allowable stress range corresponding to 500 times alternation.

3-Allowable stress of seismic design for produced axial stress in expansional connection folds of solid steel, low alloy steel, ferritic stainless steel and high extensionable steel is as following:

a) $S_a = 2 \times 724 = 1448 MPa$, if least extensional stress is equal or less than $551.6 MPa$.

b) $S_a = 2 \times 724 = 1448 MPa$, if least extensional stress is varied between 792.9 and $896.3 MPa$.

c) If least extensional strength is varied between 551.6 and 792.9 MPa, its value is calculated through interpolation method from values of clause a) and b).

4-Allowable stress of seismic design for axial stress of expansional connection accordion manufactured from stainless steel, nickel alloy (Ni-Cr-Fe alloy and Ci-Ni-Fe alloy) and Cu-Ni alloy is $S_a = 2 \times 1020 = 2040 MPa$.

10-Method of seismic evaluation of spray in towers and containers

1-Steps of evaluation of spray seismic function

1-1-Evaluation of seismic function of spray in towers and containers in done in such a way that primary local stress and difference between minimum value and maximum sum of primary stress and secondary stress must be lower than allowable stress of seismic design respectively.

1-2-Stress intensity of spray in towers and containers connected to the piping system with high level of importance is computed according to horizontal seismic force of piping design, vertical seismic force of design and load related to displacement of pipe support point.

1-3- Evaluation of seismic function of spray in towers and containers indicated in figure 19 is performed through a simple technique on the basis of the Bairard method. Detailed analysis with finite element method includes similar steps.

1-4-Strength investigation of pressurized container is performed through a simple technique (the Bairard method) on the basis of thin shell theory or detailed analysis such as the finite element method (FEM) indicated in the table 12.

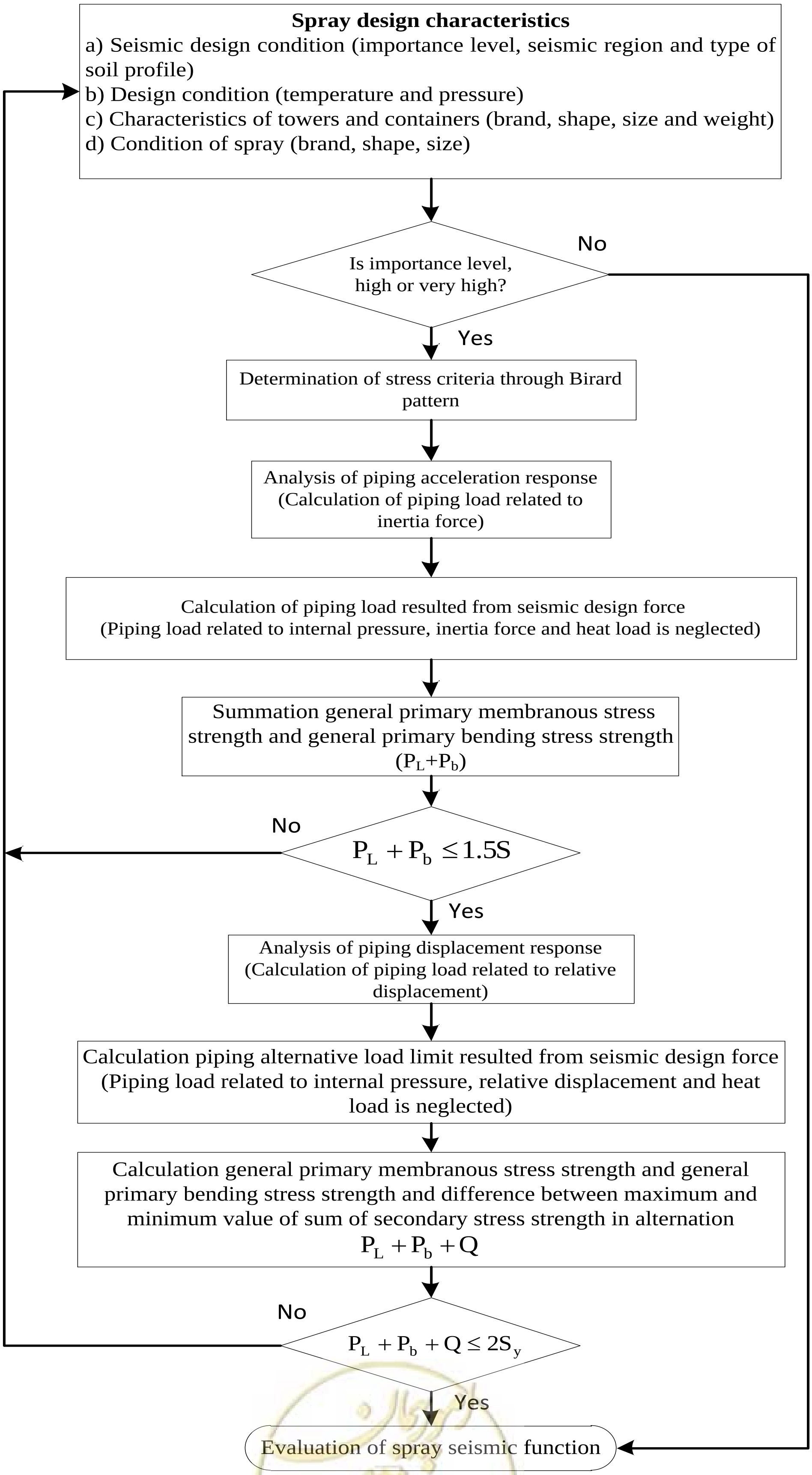


Figure 19-Steps of evaluation of spray seismic function of towers and containers

Table 12-Calculation method of spray of towers and containers

Spray	Analytical standard and technique
Pressure container	(1)- WRC107-1979 (weld research association)/"local stresses in cylindrical and spherical shell due to external loading" (according to thin wall shell theory, the Bairard method) (2)- WRC297-1987/"local stresses in cylindrical shells due to external loading on sprays) (a simple technique on the basis of finite element method (FEM)) (3)-Analysis using FEM

2-Allowable stress intensity for seismic design

Table 13 presents allowable stress intensity for spray seismic design of towers and containers according to stress type.

Table 13-Allowable stress intensity for spray seismic design of towers and containers

Allowable stress intensity of seismic design	Type of stress intensity
S	general primary membranous stress intensity
1.5S	primary local membranous stress intensity and primary moment-resisting stress intensity
$2S_y$	primary local membranous stress intensity, primary moment-resisting stress intensity and difference between maximum and minimum sum of secondary stress strength in one cycle

Where S and S_y indicate value mentioned in section 4-3.

11-Seismic function evaluation of pipe support

1-Seismic function of pipe support

Figure 20 shows steps of seismic function evaluation of pipe support that support piping system.

1-1-function of pipe support (support function) is to keeping fix the piping. All earthquake effects is exerted on piping through support and earthquake is exerted to pipe support points. Earthquake effects on piping can be reduced through suitable array of support.

1-2-Piping support drawing

Piping support drawing is prepared for characterization of connection path design to towers and containers. In piping support drawing, piping and support fixing are characterized and displacement allowance and free heat displacement of piping support structure due to earthquake, external force, dead weight and weight load is determined.

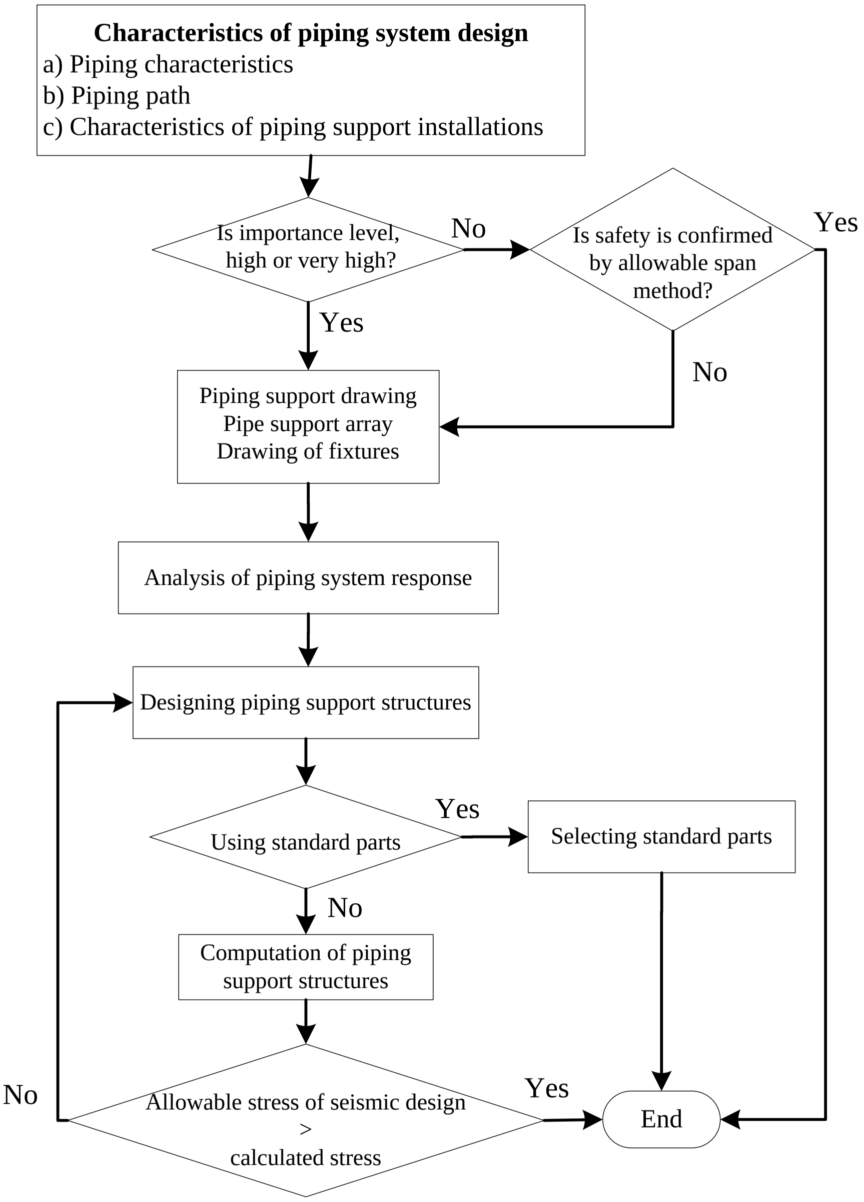


Figure 20-Steps of seismic function evaluation of pipe support

1-3-Analysis of piping system response

Loading condition of pipe support is obtained from response analysis based on design condition of related pipe support provided confirmation of seismic function of components such as piping, flangic junctions and valves after analysis of piping system response.

1-4-Support structural drawing

Analysis of piping system response is performed on the basis of related support map and shape and dimension of pipe support.

1-5-Evaluation of support seismic function

For evaluation of support seismic function, assumed calculative stress in section (4) must be lower than allowable stress of seismic design.

1-6-Simplified evaluation

Evaluation of seismic function may be neglected if the importance level of piping is low and the design is performed by means of allowable span method.

1-7-Standard support

If a pipe support is used that its earthquake-resistance has already been confirmed and pipe support loading condition is lower than standard load limit of pipe support, evaluation of seismic function may be neglected according to before-mentioned clause 4.

2-support function and type

Pipe support is contributed in haltering pipe displacement in pipe support point or fix point defined in the table (14).

Table 14-Function and detailed definitions of pipe support

Classification		Smaller classification		**Detailed definition**
name	function	name	function	
bracer	Piping load in earthquake or piping displacement harnessed due to heat deformation	brace	Displacement and rotation is braced in three direction	Converted to foxed support point
		guide	Piping displacement is braces in the direction perpendicular on brace axis	Support point of piping is positioned in a direction where piping displacement is braced
		u-shaped bolt or band	Displacement is braced in two directions perpendicular on piping axis	Support point of piping is positioned in two directions perpendicular on piping axis but when is bind installed to 4B piping or lower, three directions can be considered.
		Axial stopper	Displacement is braced in the direction of piping axis	Support point of piping is positioned in the direction of pipe axis
		Three-axial stopper	Displacement is braced in three direction of piping	Support point of piping is positioned in three direction of pipe
Vibration-resistant machine	Prevention from vibration and piping displacement	Liquid separator apparatus	Slow displacement is allowable but quick displacement is braced	Support point of piping is positioned in the braced direction
		separator apparatus of mechanical cable type	Slow displacement is allowable but quick displacement is braced	
		spring separator apparatus	Displacement is reduced by spring	Function as a spring support point, however support point in allowable span method is considered.
seat	Dead weight of piping is tolerated	seat	Dead weight of piping from lower face is tolerated and displacement of beneath direction is braced.	When displacement isn't occur due to weight, heat and seismic load, pipe support axis is considered in the vertical direction.
		Solid joint	Dead weight of piping from upper face is tolerated and displacement of beneath direction is braced.	
		Variable joint	Rebound is tolerated by springs	When displacement isn't occur due to weight, heat and seismic load, pipe support axis is considered in the vertical direction.
		Constant joint	Dead weight of piping from upper face is tolerated and displacement of beneath direction is braced.	Converted into spring support but in the allowable span method, pipe support is considered in the vertical direction,

In this definition, the purpose of pipe support point is support point related to acceleration response of piping system in earthquake. Loadings such as liquefaction and load due to heat deformation in support points such as separators may not be considered in some circumstances.

3-Pipe support array

Pipe support is consisted of welded part, appurtenance seat and pipe support structure.

3-1-welded part: pivot, saddle, heat insulation material of metallic insertion, horizontal juncture, seat, etc, installed or directly welded to piping

3-2- Appurtenance seat: metallic joints such as U-shaped bolt, pipe pin and insulator, installed for maintenance or bracing pipe to frame structure, pipe rack, adjuncts and traverse (under head), etc.

3-3-Piping support structure: beam or independent single structure installed on frame structure, pipe rack, towers, containers, etc. Piping is fixed to support structure through appurtenance sets and welded components.

4-Loading conditions

Support calculations are done using exerted load from piping through table 15.

Table 15-conditions of piping loading

Load type	Non-compressive part	Compressive pipe
Dead weight of piping	○	○
Load due to heat stress in piping	○	
Inertia force of piping in earthquake	○	○
Load due to relative displacements in support structure in earthquake	○	○

5-Calculated stress

According to structure type, pipe support stress may be calculated on the basis of analytic and standard method indicated in the table 16 in the mentioned components in the following:

5-1-pillar

5-2-beam

5-3-brace

5-4-foundation bolt

5-5-important components special to seismic design

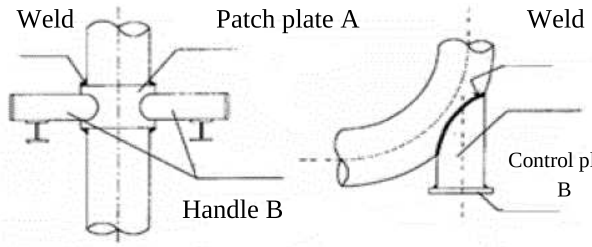
5-6-allowable stress of support seismic design

According to type of compressive section material and material of support structure, various allowable stresses are applied for seismic design. Minimum value between allowable stresses of seismic design of compressive section and material of support structure is used as an allowable stress for material of support structure that directly welded to compressive section material. Welded or installed support structure material must be selected among material listed in the left column of the table 16 based on metal type of compressive section.

Table 16-Calculated stresses of pipe support

	Section	Analytical and standard technique
Pressure-resistant material	Welding parts (circle and rectangle)	WRC-107 (1979), ASMECASE-391, CASE-392-3, and FEM analysis
	Welding parts (planar material)	ASMECASEN-318-5 and FEM analysis
	Welding parts (saddle)	FEM analysis
Non-compressive material	Structural part of Welding part	Design standard of metallic structure
	Welding parts (saddle)	FEM analysis
	Installation metallic martial	Design standard of metallic structure and FEM analysis
	Support martial	Design standard of metallic structure and FEM analysis

Compressive section materials are materials that stress is produced in them due to primary internal pressure and non-pressure Martials are different from materials of compressive section.



A: Part which is needed to study allowable stress for seismic design of pressurized material while manufactured of support structure.

B: part for which only allowable stress of seismic design for support structure is applied.

Figure 21-material of compressive section and material of support structure

-allowable stress of seismic design for materials of non-compressive section of support

Allowable stress of seismic design for materials of non-compressive section of support is indicated in the clause 3-4.

-allowable stress of seismic design for support pressure-resistant materials

Allowable stress of seismic design for support pressure-resistant materials is explained with detail in the clause 3-4.

6-Overlooking evaluation of seismic function

If support part of pipe and in same loading condition, resulted tensions is lesser than allowable stress of seismic design, calculations related to separate evolution of seismic design of various parts can be overlooked.

12-Flowchart of piping seismic design through ductile method

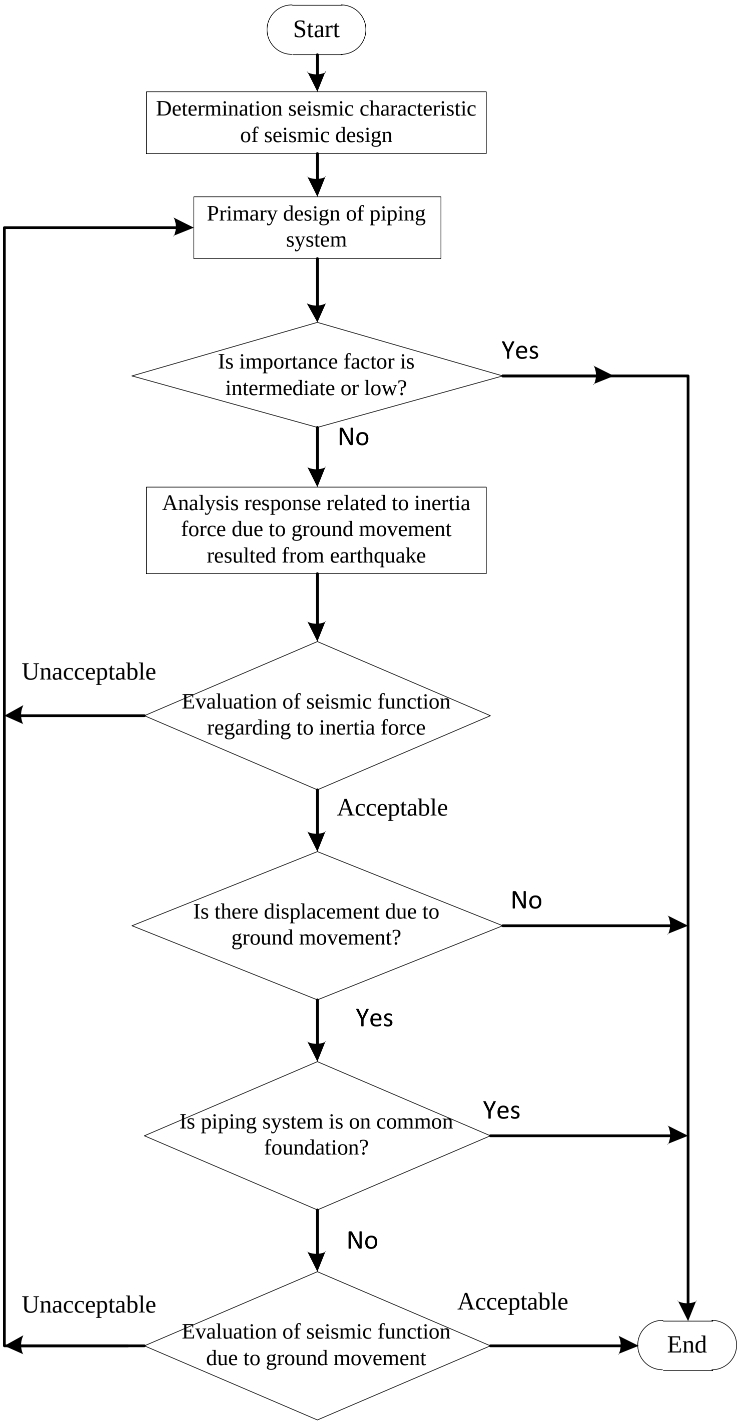


Figure 22- Flowchart of piping seismic design through ductile method

13-Evaluation of pipe bending in big deformations

According to good flexibility of steel pipes and their high deformability, these pipes don't quickly reach the rupture point even if the deformation in these pipes exceeds the range of yield stress. When these pipes undergo big deformations due to earthquake and other phenomena, it is essential to know their style of deformation after yield point and range of rupture point.

In piping system, embowed pipes give high flexibility to structural system with regard of structural characteristics. So, proper understanding of big deformations of embowed pipes as principal components of this system is essential for seismic strength evaluation of piping systems including pipes and their supports.

If pipe bent undergoes moment resisting momentum M, angle change θ (which is called bend angle, afterward) is obtained from equation (12) based on beam theory. In this relation, spheroidal (distortion) effect is considered.

$$\theta_B = k_e 90 \frac{R_1}{EI} M \tag{12}$$

Where

K_e	Flexibility factor in elastic deformations (K_e=1.65/h_d)
h_d	characteristic value of moment-resisting deformation (h_d= $\bar{T} R_1 / r_2^2$)
$\bar{T}$	Pipe thickness (mm)
E	Longitudinal elastic module (N/mm^2)
r_2	mean radius of pipe (mm)
R_1	bent radius of pipe (mm)
I	inertia moment of section (mm^4)
θ_B	bent angle (degree)

Equation (12) helps significantly in quantities understanding big deformations of bent pipe with consideration of plastic deformation. This relation is converted to relation (13).

$$\theta_B = k_p 90 \frac{R_1}{EI} M \tag{13}$$

Where K_p is flexibility factor in plastic deformation. Approximate value of flexibility factor in in-plane bending mode, in-plane expansion and ex-plane bending are given in following correspondingly, using results from non-linear analysis of pipe with 90 degree bent by means of finite element method.

Approximate relation for in-plane bending mode:

$$k_p = \left[(1.25h_d + 0.33)\theta_B(90/\alpha_p) - 0.48h_d + 0.4\right]\frac{S_o}{S_y} k_e \geqq k_e \tag{14}$$

Approximate relation for in-plane expansion mode:

$$k_p = \left[(1.28h_d + 0.03)\theta_B(90/\alpha_p) - 0.66h_d + 0.75\right]\frac{S_o}{S_y} k_e \geq k_e \tag{15}$$

Approximate relation for ex-plane bending mode:

$$k_p = \left[(1.1h_d + 0.24)\theta_B(90/\alpha_p) + 0.15h_d + 0.19\right]\frac{S_O}{S_y} k_e \geq k_e \tag{16}$$

Approximate relation formean bending, in-plane expansion and ex-plane bending:

$$k_p = \left[0.37 + \left(h_d + 0.25\right)\theta_B\left(90/\alpha_p\right)\right]\frac{S_o}{S_y}k_e \geq k_e \qquad (17)$$

Where

α_p	Pipe bent angle (degree)
S_y	yield strength or equivalent strength of yield using 0.2% strain of material (N/mm^2)
S_0	215 N/mm^2

With consideration of maximum equivalent plastic strain in embowed pipe, approximate relation of various charactristic value of pipe with 90 degree bent is obtained using characteristic parametric study of big deformations of in-plane bending (bending and expansion) and ex-plane bending using finite-element method.

$$\theta_B = 29.1\frac{\left(\varepsilon_{eq}{}^{p}\right)^{0.829}}{h_d{}^{0.456}} \qquad (18)$$

Plastic strain $\varepsilon_{eq}{}^{p}$ is equivalent plastic strain and obtained from relation (19).

$$\varepsilon_{eq}{}^{p} = \sqrt{\frac{2}{3}}\sqrt{\left(\varepsilon_x{}^{p}\right)^2 + \left(\varepsilon_y{}^{p}\right)^2 + \left(\varepsilon_z{}^{p}\right)^2 + 2\left(\varepsilon_{zx}{}^{p}\right)^2 + 2\left(\varepsilon_{xy}{}^{p}\right)^2} \qquad (19)$$

Where

$\varepsilon_{eq}{}^{p}$	Equivalent plastic strain
$\varepsilon_x{}^{p}$ ،$\varepsilon_y{}^{p}$ ،$\varepsilon_{xy}{}^{p}$ ،$\varepsilon_{zx}{}^{p}$ ،$\varepsilon_{yz}{}^{p}$ و $\varepsilon_z{}^{p}$	Components of plastic strain

14-Flowchart of inertia force and response displacement

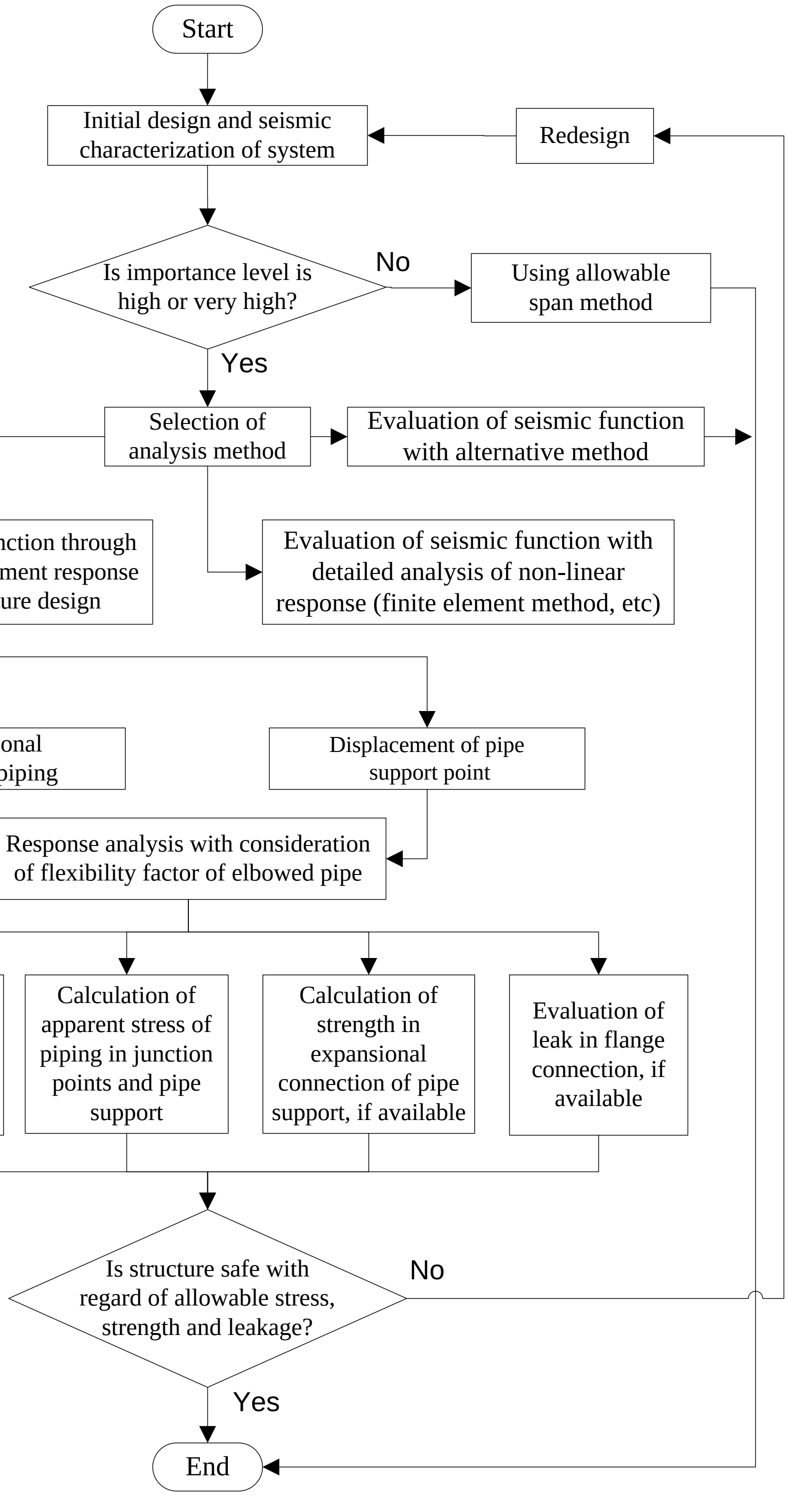


Figure 23-Flowchart of inertia force and response displacement

14-1-Method of derivation response displacement of pipe support point through corrected semi-static approach

1-Response displacement of pipe support point of containers and towers

In clause 3, the calculation method of piping design seismic intensity in pipe support points through semi-static approach is presented. Response displacement of pipe support point of containers and towers is calculated from relation (20) as following:

$$\delta_x = \begin{cases} (\mu_p + 1)\dfrac{K_y}{K_{MH}}\delta_{xMH} & \mu_p < 0 \\ \delta_{xMH} & \mu_p = 0 \end{cases} \tag{20}$$

Where

δ_x The value of horizontal displacement of pipe support point in earthquake (mm)

μ_p ductility factor of pipe support structure (towers and containers) by corresponding value with rupture mode in which μ_p has its maximum value (see relation 4-1, chapter 4).

K_{MH} Corrective horizontal seismic factor of the design, related to pipe support structure (containers and towers)

δ_{xMH} Horizontal displacement of response (mm) in pipe support structure in containers and towers related to corrected semi-static approach of the design K_{MH}. Response displacement is calculated through one of three methods including corrected semi-static method, modal analysis or analysis of time history response with replacement of pipe support structure (containers and towers) with suitable vibration system model.

2-Displacement of pipe support point in framed structures

Calculation method of response displacement in pipe support point of framed structures through semi-static method is presented together with an example of a framed structure by assumption of shear deformation model. Response displacement and displacement of intermediate stories related to i^{th} story is calculated from relation (21). The value of i is varied between 1 and 4.

$$\Delta Y_i = s_i \Delta X_i \qquad Y_i = Y_{i-1} + \Delta Y_i \tag{21}$$

Where

Y_i Response displacement in story i, Y_0=0 (mm)

ΔY_i Relative displacement of story in i^{th} story, $\Delta Y_i = Y_i - Y_{i-1}$ (mm)

s_i ratio of displacement in i^{th} story that is calculated from relation (22) but its value is equal or more than 1.

$$s_i = 1 + \mu_{pi} = 1 + \frac{1}{4C}\left\{\left(\frac{K_{MHi-1}}{K_{MHi}}\right)^2 - 1\right\} \tag{22}$$

K_{MH1} is calculated from relation (23):

$$K_{MH1} = \min(K_{MH(i-1)}, K_{yi}) \text{ but } K_{MH0} = K_{MH} \tag{23}$$

Where

K_{yi} is yield seismic factor in i^{th} story which its value is given by relation (24):

$$K_{yi} = \frac{Q_1}{\mu W_i + \ldots\ldots + \mu_4 W_4} \tag{24}$$

K_{MH} is corrective horizontal seismic factor for pipe support structure design (frame) that is calculated from relation (25).

$$K_{MH} = \beta_5 K_H \tag{25}$$

Where

β_5 Horizontal response magnification factor

K_H Horizontal seismic factor related to design seismic force in the ground surface

ΔX_i i^{th} relative in terms of K_{MH1} (mm) that its value is given by relation (26):

$$\Delta X_i = K_{MHi} \frac{\mu_1 W_i + \ldots\ldots + \mu_4 W_4}{k_i} \tag{26}$$

Where

W_i Level load in i^{th} story (kN)

K_i springy of i^{th} intermediate story (kN/mm)

Q_1 yield strength (kN)

μ_i distribution factor of seismic intensity in i^{th} story (see relation 10-16, chapter 10)

μ_{pi} ductility factor in i^{th} story (that is calculated from relation 20)

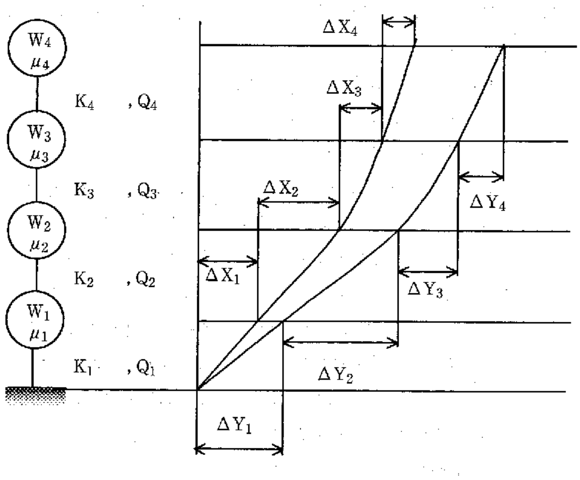


Figure 24-Relative displacement of i^{th} story

14-2-equivalent linear analysis, detailed analysis and response magnification factor

1-Equivalent linear analysis

1-1-Outline

It is appear rationally to use evaluation by equivalent linear model for the purpose of easing design for designing piping system in plastic mode. Equivalent linear model is used in analysis of embowed pipe using flexibility factor (k_p) in plastic region. This factor (k_p) is obtained by modification of flexibility factor (k_e) in elastic region using flexibility characteristic value and angular displacement. Although according to analytical results, flexibility factors of in-plane bending, in-plane expansion and ex-planar bending have different values, but same correction factors are used for them. With consideration of making no difficulty in the accuracy of the problem, lower frequency of small torsional angle than to allowable ultimate ductility factor and duplication of positive and negative deformation, these correction factors are assumed to have the same value.

1-2-Placticity factor of embowed pipe

In pipes of 90 degree bent, mean flexibility factor is determined with consideration of small angular deformations and frequent positive and negative shifts of deformation. Flexibility factor, k_p, is obtained by referring to relation (15) for in-plane bending, in-plane expansion and ex-plane bending (in this relation, θ_D must be used instead of θ_B). However, k_p is equal to 1.0 with assumption of torsion. Moreover, in the pipes with bent of 45 or 60 degree, etc. angle of allowable displacement is derived through interpolation.

1-3-Equivalent linear analysis procedure

Method of Equivalent linear analysis is applied according to following procedure.

Behavior of embowed part of pipe is assumed as its equivalent ductile factor and then, whole system is analyzed. In this manner, flexibility factor for bending angle due to resultant of loads composition is considered.

Convergence of calculation is done until flexibility factor obtained from calculations, in terms of bend angle is conformed to assumed value. In these calculations, ductility factor is convergence criterion and calculation is considered as convergent when its error is about 5%. Value of angular displacement of bent is obtained as mean least square of relative displacement of angle in three directions between junction of two points of bent and direct pipe.

Calculations are confirmed when angular displacement of bent obtained from analytical result of whole system is lower than allowable angular displacement.

Except for bent part, evaluation of calculation in other parts is done by stress as it shouldn't be in plastic range.

1-4-Condition of equivalent linear analysis method

Following assumptions are considered in the case of equivalent linear analysis method:

Non-linear displacement and characteristic of frictional load of supports and cracks (looseness), etc are not considered.

Inertia force and response displacement are applied in one direction.

According to operational load composition and seismic load, the worst direction in the composition is considered.

Table 17 presents composition of loads.

Table 17- Composition of operational load and seismic load

Load type / Stress type	Seismic load		Operational pressure	Fluid pressure
	Relative displacement	Inertia force		
Longitudinal stress σ_1		○	○	○
Range of alternative stress σ_E	○ (support structure)	○		

2-Detailed analysis

In detailed analysis by means of finite element, etc. there is a method called hybrid (composite) method, in which embowed pipe, T-shaped pipe and their equivalents are modeled using shell element or spatial element and direct pipe as beam element. In embowed pipe where non-linear behavior is indicated using section flatting, analysis is performed with consideration of non-linear behavior of materials and geometry (big deformations). However, suitable value obtained from material tension test must be used for yield strength and rate of non-linear cold hardening of materials.

Elasto-plastic analysis of pipe using beam model by means of finite element method can be applied for analysis of non-linear element of bent pipe (elbow). Although vibration of both system of support and piping must be analyzed but for facilitation of work, acceleration and displacement obtained from response analysis in support point may be applied as a seismic force to piping.

Equivalent plastic strain is obtained from relation 27:

$$\varepsilon_{eq}{}^{p} = \sqrt{\frac{2}{3}}\sqrt{\left(\varepsilon_x{}^{p}\right)^2 + \left(\varepsilon_y{}^{p}\right)^2 + \left(\varepsilon_z{}^{p}\right)^2 + 2\left(\varepsilon_{yz}{}^{p}\right)^2 + 2\left(\varepsilon_{zx}{}^{p}\right)^2 + 2\left(\varepsilon_{xy}{}^{p}\right)^2} \quad (27)$$

Where

$\varepsilon_x{}^{p}$, $\varepsilon_y{}^{p}$, $\varepsilon_z{}^{p}$, $\varepsilon_{yz}{}^{p}$, $\varepsilon_{zx}{}^{p}$ and $\varepsilon_{xy}{}^{p}$ components of plastic strain

3-Response factor of magnification

In earthquakes of high intensity, effect of energy wastage in elasto-plastic behavior of embowed pipes is predictable. In addition, the effect of decrease in non-linear response of friction or support gap, etc. is high. Reduction effect of displacement response in additional decay effect due to high hazard level earthquakes is predictable because of existence of about 3-5 mm gap in pipe support in the direction perpendicular to support of high pressure gas piping system. This effect especially is significant in cases where support structure exerts high response due to flexibility and has high flexibility as a result of piping shape and support type.

In evaluation of piping system with equivalent decay ratio method that uses spectrum of frictional system response, reduction in response is appeared due to effect of frictional decay effect.

For example, according to an experimental calculation, magnification response factor in frictional system with one degree of freedom (decay factor of 2%) in equivalent frictional decay factor is about 1.5 based on type of the ground using a value resulted from division of frictional force on input acceleration.

In seismic response analysis of piping system, magnification response factor is obtained as following through using equivalent linear beam model:

1-Although magnification factor of horizontal response in pipe support structure is assumed to be 2, but pipe supports that have sliding surfaces and energy wastage effect exists in them is considered to be 1.5.

2-In all cases, magnification response factor in vertical direction is equal to be 2.

3- Magnification response factor is obtained with exchange of absorbed energy as a result of non-linear behavior with a suitable equivalent decay constant.

14-3-Evaluation of rupture modes of bent pipe, junction pipe and direct pipes and details of undulatory deformation of pipe

1-Evaluation of rupture modes of bent pipe

Bent pipe is evaluated using allowable angular deformation. Allowable angular deformation is obtained under conditions in which allowable plastic strain of bent is 2%. This bent is calculated through parametric study of charactristic bent's big deformations in two modes of in-plan (bend and expansion) and ex-plan bend in compare with various characteristic value of embowed pipe using finite element method.

$$\theta_{aL2} = 29.1\frac{\varepsilon_{paL2}^{\;0.829}}{h_d^{\;0.456}} \tag{28}$$

Where ε_{paL2} is equivalent plastic strain and θ_{aL2} is allowable angular displacement. This relation is for pipe with bent of 90 degree and for pipes with bent of 30 and 45 degree, allowable angular displacement is computed through interpolation.

As a result, torsion angle of bent pipe with consideration of maximum plastic strain and according to allowable ultimate plastic deformation regarding acceleration response is equal to half of range, or equal to 2% such that in half amplitude of 2%, $\varepsilon_{paL2} = 0.02$ in equivalent plastic strain. So, torsion angle is computed as relation (29).

$$\theta_a = \frac{1.14}{h_d^{\;0.46}} \tag{29}$$

Where

θ_a is allowable angle of bent pipe (in terms of degree).

Table 18 shows relation between characteristic values of bent deformation and allowable angle of high radius elbow of 90 degree. In this regard, allowable plastic strain is assumed to be 2%.

2-Evaluation of rupture modes for junction pipes and direct pipes

Since in response analysis, junction pipe and direct pipe are modeled as linear beam element and existence of destabilizing phenomena such as buckling due to high strain is unacceptable so as an alternative, same evaluation is used for safety.

3- Wavy shape deformation

Wavy shape deformation is progressive non-elastic deformation. Non-elastic strain changes per each period frequently. Constant-shape wavy deformation occurs when net non-elastic strain due to alternative known load is constant in next cycle.

There is probability for increase of progressive non-elastic deformation in the apparatus and piping. This increase is due to addition of frequent variation of mechanical secondary stress, heat secondary stress or both of them to a place where first stress resulted from internal pressure and dead weight is higher than a certain limit.

For example, in embowed pipe, increase in the amount of progressive non-elastic deformation occurs a result of overlapping of primary stress with internal pressure and a mechanical alternative secondary stress, such that tensile stress of pipe wall is produced in the sounding of the pipe because of internal pressure and excess applied alternative load through seismic loading. So it is essential that primary and secondary stress in that unit be remained in the allowable range to prevent excess plastic deformation or progressive deformation. In equivalent linear method in embowed pipe, as explained in section 1, reception criterion is the value of allowable displacement that is obtained by assumption of allowable ultimate plastic deformation equal to be 2% in the half of amplitude in equivalent plastic strain.

Table 18-allowable angle in high radius elbow of 90 degree

Allowable angle (degree)	Charactristic value of bending displacement	Wall thickness (mm)	External diameter (mm)	Nominal diameter	
				(B)	(A)
1.69	0.420	3.7	48.6	1－1/2	40
1.79	0.371	3.9	60.5	2	50
1.75	0.392	5.2	76.3	2－1/2	65
1.82	0.360	5.5	89.1	3	80
1.89	0.331	5.7	101.6	3－1/2	90
1.94	0.312	6.0	114.3	4	100
2.03	0.283	6.6	139.8	5	125
2.11	0.260	7.1	165.2	6	150
2.23	0.231	8.2	216.3	8	200
2.32	0.213	9.3	267.4	10	250
2.39	0.198	10.3	318.5	12	300
2.39	0.200	11.1	355.6	14	350
2.39	0.200	12.7	406.4	16	400
2.39	0.200	14.3	457.2	18	450
2.45	0.189	15.1	508.0	20	500
2.50	0.181	15.9	558.8	22	550
2.49	0.183	17.5	609.6	24	600
2.49	0.182	18.9	660.4	26	650

14-4-Details of design procedure for flange connection and required contact pressure of washer

1-Procedure of seismic function evaluation

Figure 26 shows procedure of seismic function evaluation in flange connection. In fact, required seismic function in flange connection is equal to insulation degree of connection for passage of high pressure gas due to effect of existence load in piping.

Leak from washer is evaluated using washer factor, which is obtained by conversion of axial tension force F and bending moment M due to seismic load to the equivalent pressure in the washer plate.

1-1-Piping axial force and bending moment in flange connection is calculated from analysis of acceleration response and piping displacement.

1-2-Leakage evaluation is done in such a way that required connection pressure of washer must be lower than connection pressure of washer in the first step of bolt tightening. Required connection pressure of washer is equal to sum of applied pressure on washer due to internal pressure of pipe and pressure equal to axial force and calculated bending moment in piping.

2- Required connection pressure of washer in simple evaluation of leakage

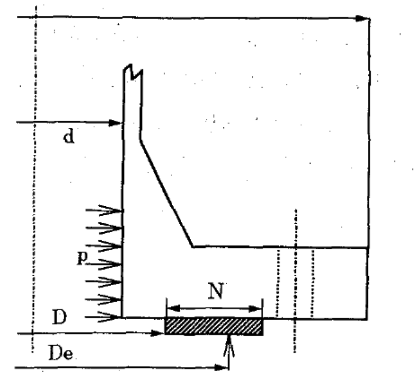


Figure 25-Edge of seat plate of washer

Equal internal pressure resulted from axial force F and bending moment M due to seismic load in plate of washer is given as relation 30.

$$p_e = \frac{4F_g}{\pi D_e^{\,2}} + \frac{16M}{\pi D_e^{\,3}} \qquad (30)$$

Where

F_g axial force due to seismic load (N)

M bending moment due to seismic load (N.mm)

D_e mean diameter in connection face of washer (mm), $D_e=D_{1G}+2(N_g-b_g)$

D_{1G} Internal diameter of washer (mm)

N_g washer width (mm)

b_g effective width of washer (mm)

So required connection pressure of washer by given is $_{eq}$Prelation 31.

$$p_{eq} = mp + \alpha p_e \leqq \sigma_a \qquad (31)$$

In this relation, m is washer factor and α is correction factor of leakage pressure due to load resulted from equal internal pressure (0.75m).

In the connection of flange with bolt in pipe, joint implementation isn't possible. In this case, converted stress due to primary tightening for each bolt as conventionally used before is known. As a result, connection pressure of washer due to primary tightening of bolt is computed simply on the basis of bolt number, cross section of all bolts and washer size with least standard diameter.

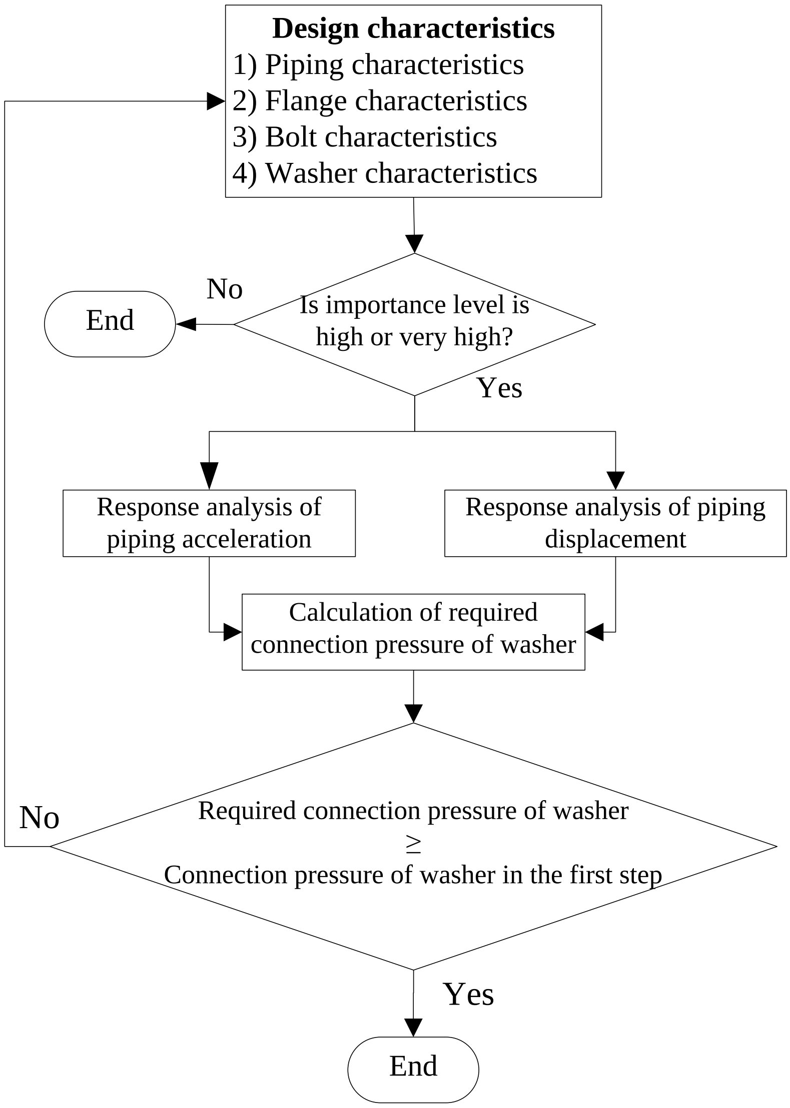


Figure 26-Evaluation of seismic function of the flange connection

14-5-Details of expansional connection evaluation

1-calculation of total value of axial stress

Total value of axial stress in expansional connection is given by a method similar to the allowable stress method

2-Evaluation

Figure 27 shows the flowchart for evaluation of seismic function in the expansional connection. In piping system involving expansional connection, for evaluation of seismic function in the expansional connection, relative displacement in both ends of connection must be lower than allowable displacement in 50 times vibration. Especially, total value of maximum axial stress produced in accordion part of expansional connection due to relative displacement of support structure in both ends must be lower than total value of allowable stress corresponding to 50 times allowable cycle for relevant connection materials. In this manner, it is possible to estimate axial relative displacement and relative displacement in the direction perpendicular to the axis, separately. Evaluation is done in such a way that maximum total axial stress in accordion must be lower than twice of value of allowable stress corresponding to 50 times allowable cycle obtained from figures of Japan standard Institute B8281 "analysis of stress and rupture of compressive container" (1993) according to accordion materials.

a- Allowable stress, S_a in seismic design of axial stress produced in accordion part of expansional connection made from hard steel (carbonized), low alloy steel, ferritic stainless steel and high tension strength steel is equal to following values:

- When minimum tensile strength is equal or lower than 551.6 MPa, $S_a = 2 \times 1896 = 3792\,\mathrm{MPa}$
- When minimum tensile strength is in the range of 792.9-896.3 MPa, $S_a = 2 \times 1586 = 3172\,\mathrm{MPa}$
- When minimum tensile strength is in the range of 551.6-792.9 MPa, it is obtained appropriately from value of a) and b).

b- Allowable stress used in seismic design produced in expansional connection accordion made from austenitic stainless steel, nickel alloy (Ni-Cr-Fe and Ni-Fe-Cr alloy) and Ni-Cu alloy is equal to be $S_a = 2 \times 2379 = 4758\,\mathrm{MPa}$.

3-Expansional connections for purposes other than earthquake movement

In piping systems that expansioanl connections other than designed connection for design's seismic movement are available, pipe support must be designed in such a way that displacement of expansional connection is not exceeded from tolerance limit due to seismic movement of the design or support of expansional connection must have adequate strength to maintain function of displacement bracing component (bracing through bolt rack, rack plate, adjustment ring, etc) and contain applied reaction computed through analysis of pipe system response due to ground movement.

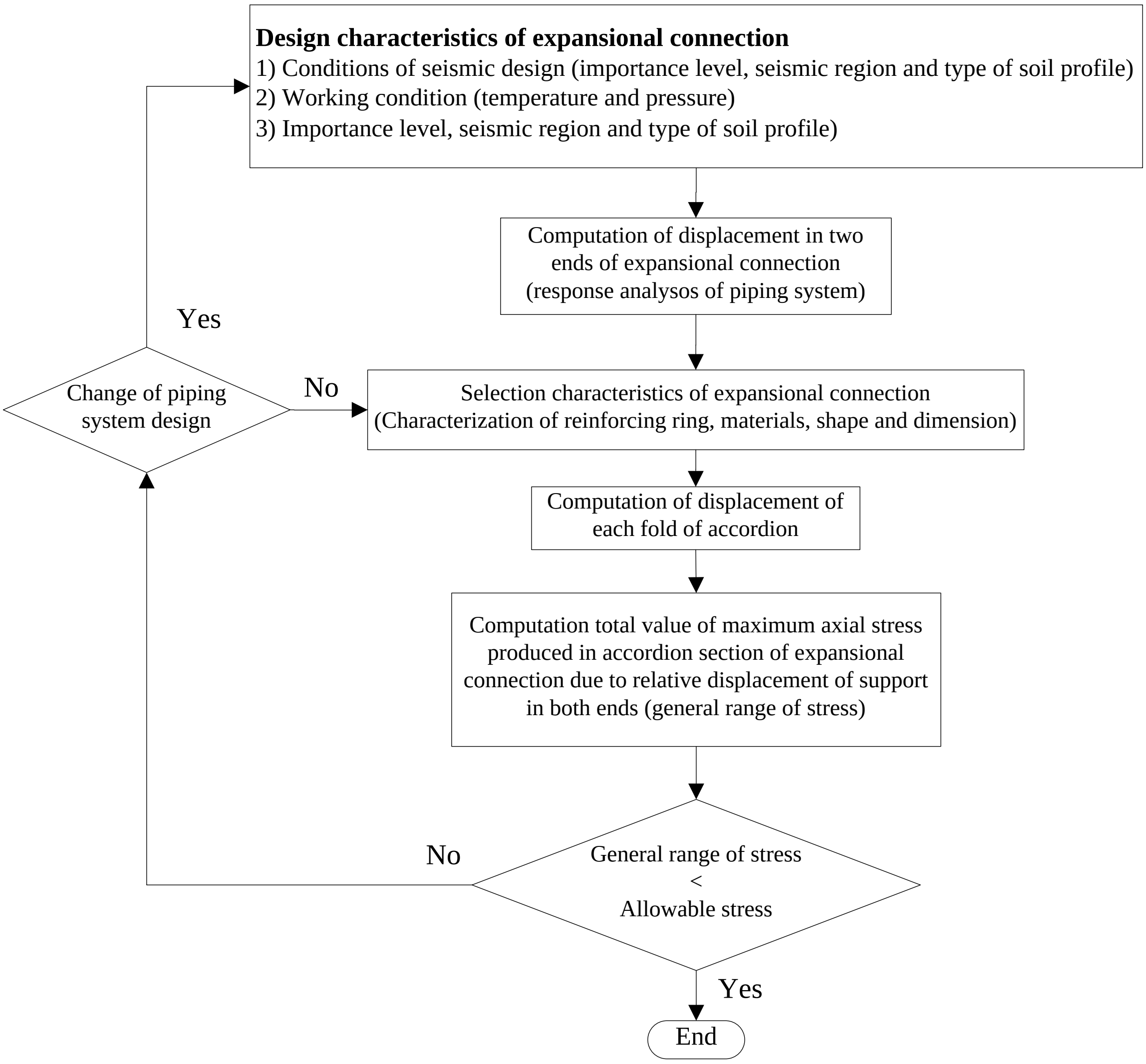


Figure 27-flowchart of seismic function evaluation of expansional connection

14-6-Details of spray evaluation of towers and containers

Figure 27 shows flowchart of seismic function evaluation of expansional connection for towers and containers with simplified method based on the Bairard method. Stress intensity for sprays of towers and containers in piping system is computed on the basis of load related on horizontal seismic force of the design, vertical seismic force of the design and displacement of pipe support point.

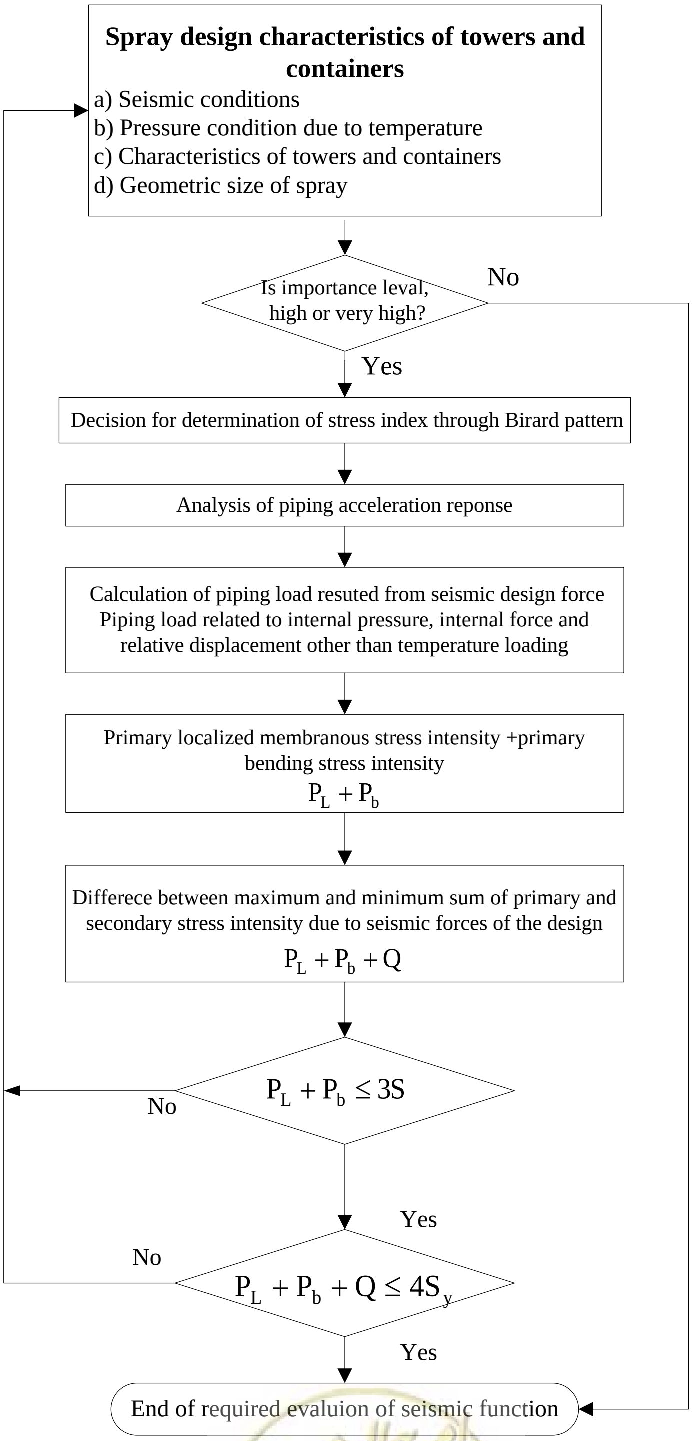


Figure 28- Flowchart of required seismic function evaluation in sprays of towers and containers due to inertia force

Table 19 presents allowable stress intensity in seismic design of towers and containers

Table 19-Allowable stress intensity in seismic design of sprays of towers and containers

Allowable stress intensity in seismic design	Stress type	Number
3S	Primary localized membranous stress intensity+primary bending stress ($P_L + P_b$)	1
$4S_y$	Difference between maximum and minimum sum of primary and secondary stress intensity due to seismic movement of design in one cycle $(P_L + P_b + Q)$	2

Computation method of spray stress of towers and containers is a simplified method (Bairard method) based on thin shell theory in the field of investigation of compressive container or detailed analysis by means of the finite element method (FEM), etc.

Procedure of detailed analysis using finite element method, etc. is similar to the simplified procedure.

14-7-Evaluation of required seismic function in pipe support

1- Evaluation procedure of required seismic function in pipe support

Figure 29 shows the evaluation procedure of required seismic function in pipe support (pipe support structure, appurtenance seat and weld metal components).

1-1-Pipe support function (support function) is to confine piping displacement or fixing it. All effects of earthquake on piping are applied on its support. In earthquake, ground movement (acceleration and displacement) and pipe support displacement due to liquefaction and ground displacement (that are called input seismic conditions thereafter) are applied on pipe support as an input in its support point. With adequate adjustment of pipe support, effect of earthquake movement on piping and its support structure (or piping system) is reduced.

1-2-Conditions of support loading and analytical response model of piping system

In evaluation f seismic function, analytical response model of piping system is adjusted for each of input conditions based on pipe support function.

In addition to pipe support function, piping support function (function of constraining deformation and enduring load due to seismic force), piping bracing, control seismic input of piping and freeing pipe support function due to ground movement are presented.

1-3-Allowable conditions of seismic force of the design in pipe support point

Response analysis of piping system inertia force is performed using acceleration and displacement of pipe support point due to earthquake movement that is explained in clause 14 of Appendix.

Produced force in support point that is obtained from response analysis is varied according to inertia force loading conditions of pipe support. Response analysis of ground movement in piping system is performed just like as analysis of pipe support point displacement. Produced displacement in support point that is obtained from response analysis is varied according to ground movement loading conditions in pipe support.

1-4-Evaluation of required seismic function

If function evaluation of allowable conditions is not acceptable, design change is performed and the mentioned procedure is duplicated until seismic function is conformed to changed characteristics.

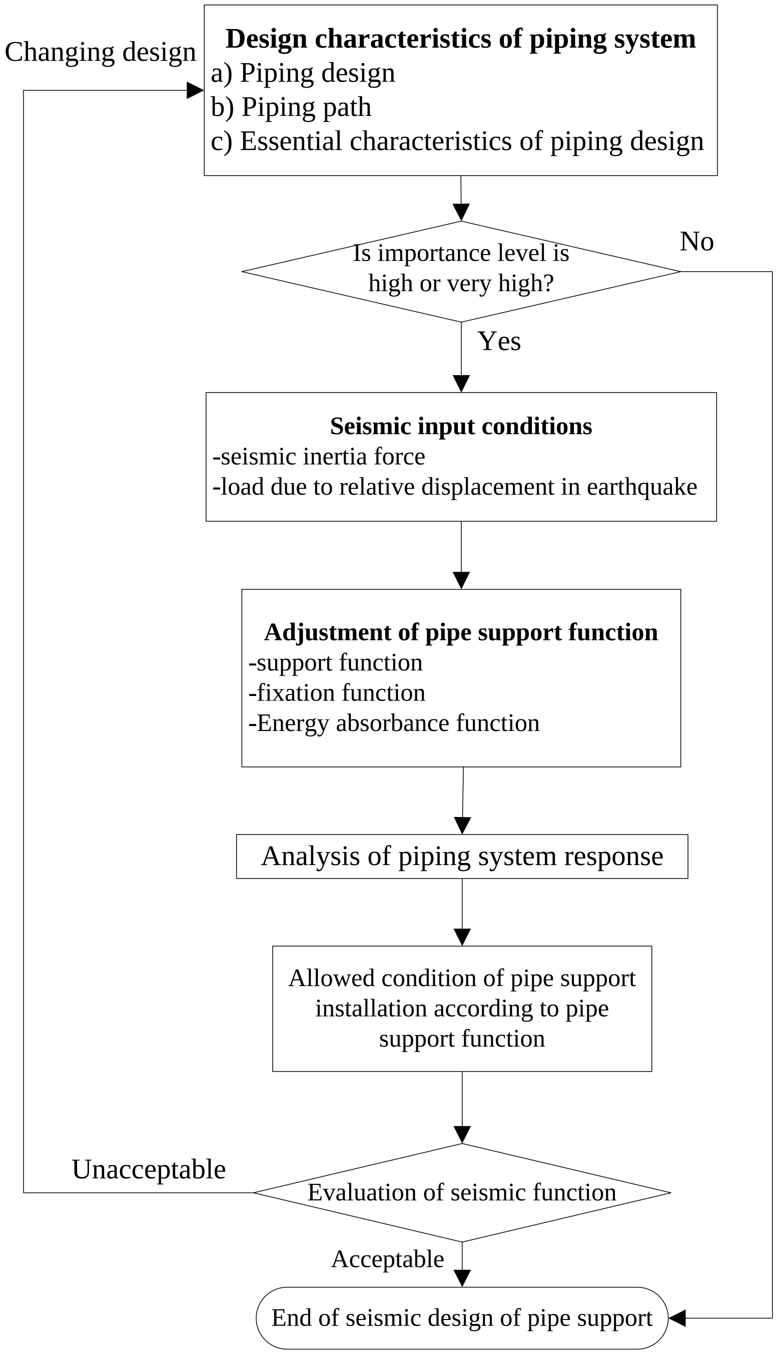


Figure 29-evaluation required seismic function in pipe support

2-Function of pipe support and allowable conditions

In the following, pipe support function (pipe support structure, appurtenance seat and weld metal components) is presented for maintaining seismic function of piping against leakage of high pressure gas. Maintenance of intended function is necessary during and after occurrence of earthquake.

2-1- Support function

In fact, tolerance of piping reaction or bracing its displacement in earthquake due to response acceleration of ground movement or displacement due to ground liquefaction is the actual function of support. Although plastic deformation is allowable in the case of one-pipe support that has relevant function, support load must not be lower than loads that leading to rupture or (destruction load) buckling. In this manner, for appurtenance seat that doesn't have the capability of absorbing deformation after plastic deformation step such as hinges or insulators and has composite function between components, stress produced from seismic load must be lower than yield stress.

2-2-Fixation function

In fact, dynamic indolence of piping from two sides of pipe support or prevention of reciprocal deleterious effect is a function. In a supports involving related function, separation of analytical model of piping response can be considered. In fixed support of pipe (that is called fixed support of pipe or brace, thereafter) formed stress in pipe support due to reaction or moment due to deformation and rotation in three direction must be lower than yield stress.

2-3-Function of energy absorbance

Absorbance of earthquake energy through installation of seismic separators (and whatnot) in pipe support and control seismic input due to earthquake movement in piping is accounted a function. Pipe support must have following qualifications to fulfill this function:

Response displacement must be lower than allowable displacement

Plastic displacement of energy absorbent must be lower than deformability of energy absorbent.

According to energy absorption in pipe support, it is conceived that function confirmation is necessary.

3-Loading condition

Table 20 present evaluations on the basis of applied loads on piping.

Table 20-Loading condition of piping

Load type	Non-compressive components	Compressive components
Load due to piping weight (pressure and weight)	○	○
Load due to heat stress of piping	○	
Inertia force in piping due to earthquake	○	○
Load due to relative displacement in support structure due to earthquake	○	○

4-Allowable condition of pipe support

Allowable condition of pipe support is considered according to load characteristics and their deformations. Figure 30 shows specifications of load and deformation. In this figure, dashed part indicates difference in data. Yield load, limit state load and releasing load are defined as follow:

Yield load

Yield load is equal to value of design's yield load of, F_{yd} or lower than values indicted in figure 30.

Limit state load

Limit state load is equal to minimum rupture load, F_n (minimum load that leads to destruction) or than values indicted in figure 30.

Releasing load

Releasing load is equal to maximum rupture load, F_x (maximum load that leads to destruction) or than values indicted in figure 30.

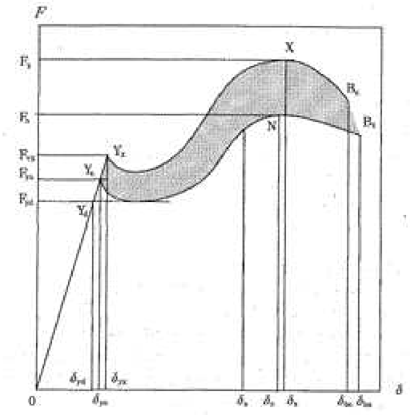


Figure 30-Bold diagram and deformation of piping support structure

F_{yx} Maximum yield load

δ_{yx} Maximum yield displacement

Y_x Maximum yield strength

F_{yn} Minimum yield load

δ_{yn} Minimum yield displacement

Y_n Minimum yield strength

F_{yd} Yield load of design

δ_{yd} Yield displacement of design

Y_d Yield strength of design

δ_a Allowable displacement

F_x Maximum rupture capacity (maximum load of destruction)

X Location of rupture maximum capacity

F_n Minimum rupture capacity (minimum load of destruction)

N_r Location of minimum rupture load

δ_{bx} Maximum rupture displacement

B_x Location of maximum rupture displacement

δ_{bn} Minimum rupture displacement

B_n Location of minimum rupture displacement

5-Calculation method of allowable condition of pipe support structure

5-1-Yeild load of pipe support

Yield load of used materials are described in clause 3-4 of material.

5-2-Limit state load of pipe support structure: limit state moment

Limit state moment is as follow:

a)When moment affects around strong axis of H-shaped section and main axis of can-shaped section:

$$\text{If } \frac{N}{N_r} \leqq \frac{A_w}{2A} \text{ then ، } M_{Pc} = M_P \tag{32}$$

$$\text{if } \frac{N}{N_r} > \frac{A_w}{2A} \text{ then ، } M_{Pc} = 1.14\left(1 - \frac{N}{N_y}\right) M_p \tag{33}$$

b) When moment affects around weak axis of H-shaped section

$$\text{If } \frac{N}{N_r} \leqq \frac{A_w}{A} \text{ then } M_{Pc} = M \tag{34}$$

$$\text{If } \frac{N}{N_r} > \frac{A_w}{A} \text{ then } M_{Pc} = 1.14\left\{1 - \left(\frac{N - N_{wY}}{N_Y - N_{wY}}\right)^2\right\} M_p \tag{35}$$

In this case, $N_{wY} = A_w \sigma_Y$

c) Plastic moment due to joint effect of bending moment around string axis M_x, bending moment around weak axis My and axial force in these positions $M_{P_{CX}}$ and $M_{P_{CY}}$ are obtained correspondingly from clauses a) and b).

$$\left(\frac{M_x}{M_{Pcx}}\right)^2 + \frac{M_y}{M_{Pey}} = 1 \tag{36}$$

d) Design relation of components

Compressive axial force N and maximum bending moment M_1 must be fulfill in the following relation.

$$\frac{N}{N_{er}} + \frac{C_M M_1}{\left(1 - \frac{N}{N_E}\right) M_{er}} \leqq 1.0 \tag{37}$$

$$\frac{M_1}{M_{cr}} \leqq 1.0 \tag{38}$$

N	Axial pressure (N)
N_E	Euler buckling strength in bending face (N)
N_{er}	minimum strength value against buckling (N)
M_{pc}	bending plastic moments due to compressive force (N.mm)
M_1	absolute value of greater moment affect in both ends of pillar (N.mm)
M_2	absolute value of smaller moment affect in both ends of pillar. When component undergoes simple deflexion, moment is positive and when component undergoes double deflexion, moment is considered negative (N.mm).

M_{cr} strength of lateral buckling when there isn't any compressive axial force (N.mm). when moment is existed around weak axis of can-shaped section, H-shaped section and steel pipe, it is assumed that $M_{cr} = M_P$.

C_M Factor related to distribution of bending moment when bending moment affect around strong axis.

$$C_M = 0.6 + 0.4\frac{M_2}{M_1} \geqq 0.4 \tag{39}$$

$$C_M = 1 - 0.5\left(1 - \frac{M_2}{M_1}\right)\sqrt{\frac{N}{N_E}} \geqq 0.25 \tag{40}$$

6-Calculation method of allowable condition for U-shaped bolt

6-1-Yeild load of U-shaped bolt is as following:

$$F_{L1z} = \frac{\pi}{4} d_b{}^2 \sigma_{by} \tag{41}$$

$$F_{L1y} = 2\frac{\pi}{4} d_b{}^2 \sigma_{by} \tag{42}$$

6-2-Limit state load value of U-shaped bolt is as following:

$$F_{L2z} = 0.7\frac{\pi}{4} d_b{}^2 \sigma_{bBu} \tag{43}$$

$$F_{L2y} = 1.4\frac{\pi}{4} d_b{}^2 \sigma_{bBu} \tag{44}$$

F_{L1z} :Yield load in the direction perpendicular to the pipe axis of U-shaped bolt (N)

F_{L1y} :Yield load in the direction perpendicular to U-shaped bolt (N)

F_{L1y} :Yield load in the direction perpendicular to U-shaped bolt (N)

F_{L2z} :Limit state load in the direction perpendicular to the pipe axis of U-shaped bolt (N)

F_{L2y} :Limit state load in the direction perpendicular to U-shaped bolt (N)

d_b of Dia eterU-shaped bolt (mm)

σ_{bBu} :Rapture stress of U-shaped bolt (N/mm^2)

In the next table, an example of computed load from above-mentioned clauses is given. In these calculations, material type of U-shaped bolt is assumed to be SS400.

Table 21-Allowable load of U-shaped bolt

Limit state load		Yield load		db	External diameter of pipe	Nominal diameter of pipe
FL2 y	FL2z	FL1 y	FL1z			
kN	kN	kN	kN	mm	mm	B
44	22	38	19	10	48.6	1-1.2
44	22	36	19	10	60.5	2
44	22	38	19	10	76.3	1-1.2
63	32	55	28	12	89.1	3
63	32	55	28	12	101.6	3-1.2
63	32	55	28	12	114.3	4
113	56	99	49	16	139.8	5
113	56	99	49	16	165.2	6
113	56	99	49	16	190.7	7
176	88	148	74	20	216.3	8
176	88	148	74	20	241.8	9
176	88	148	74	20	267.4	10
253	127	213	106	24	318.5	12
253	127	213	106	24	355.6	14
253	127	213	106	24	406.1	16
253	127	213	106	24	457.2	18
253	127	213	106	24	508.0	20
253	127	213	106	24	558.8	22
253	127	213	106	24	609.6	24

7-Calculation method of allowable condition of other appurtenance seat

7-1-yeild load

Yield load of material is explained in clause 4-3-material

7-2-Limit state load

Limit state load is considered to be 1.5 times of yield load.

8-Calculation method of allowable condition for joints with weld

8-1-yield load

Yield load of used material is explained in clause 4-3-material

7-2-Limit state load

Limit state load is 1.5 times of yield load.

15-Flexibility of piping system and seismic design procedure due to ground liquefaction

1-Providing flexibility of piping system

Providing flexibility of piping system of piping system is depends on methods of clauses 1-1 to 1-3 or a combinative method.

1-1-Piping ring method

In this method, relative displacement among support points is absorbed through insertion of piping ring among related support points. Relative displacement is braced in three directions with formation of piping ring.

1-2-Method of flexible pipe

In this method, relative displacement among support points is absorbed through insertion of flexible pipes among related support points.

1-3-Method of pipe free support

In this method, high flexibility exists in system and relative displacement is absorbed using supports with releasing function of displacements due to ground displacement in foundation. With movement of piping foundation (settlement and horizontal displacement of ground due to liquefaction), high displacement is apparent in piping system that may damage other surrounding structures such as adjacent small pipes (pipe belongs topping system) in the intersection of these structures with system. So it is necessary to be noticed.

2-Procedure of seismic function evaluation of piping system to displacement of foundation due to ground displacement

Figure 31 shows procedure of seismic function evaluation of piping system to displacement of foundation due to ground displacement

2-1-This evaluation is performed in the state of liquefaction

2-2-If piping foundation together with first support of pipe is constructed on the same foundation after seismic safety valve, seismic function evaluation of relevant piping system due displacement resulted from ground movement in foundation is acceptable.

2-3-When piping system cannot be constructed on one common foundation, it is necessary to confirm flexibility of piping system and impossibility of leakage of high pressure gas from piping due to relative displacement of foundation according to proposed function or results of investigations

3-Response analysis and range of seismic function evaluation due to ground movement

3-1-Piping system on common foundation

In order to secure seismic strength of high importance piping system due to ground movement, this system is constructed on common foundation and first support point after seismic safety valve is fixed using welding (or any other method). On the other word, related pipe support must be fixed to prevent leakage due to deformation of weak pipe support as a result of loading from ground movement.

In the example of figure 33, to secure seismic function of hatched section (a), influence of hatched section (b) due to ground movement is analysed and evaluation of seismic function is performed for fixed support of pipe and foundation of hatched section (c) according to fixation function of that part due to ground movement. It is preferred that seismic function of piping system due to ground movement in the analysed section to be confirmed.

3-2-Piping on non-common foundations

Piping system must have adequate flexibility against ground movement. According to flexibility evaluation of piping system in example of figure 34, in which evaluation of hatched section (a) is performed under influence of hatched section (b) due to ground movement and seismic function of hatched section (c) is evaluated, it is preferred to perform similar evaluation of piping system in analyzed extent. However, analysis of hatched section (b) is depends on the analytic model that simulate excellently deformation mode of piping, piping support structure and its foundation as a result of ground movement.

3-3- No-fixing support on common foundation

If piping system is to be building on common foundation but first support of pipe next to seismic safety valve is not fixed due to its position on common foundation, evaluation of seismic function due to ground movement is performed according to procedure in clause 2.

4-Evaluation of seismic function of piping system with free support as a result of ground movement

In piping system with free support, it is required that seismic function of system to be confirmed both against maximum reaction of free support and effect of displacement due to ground movement.

4-1-In this case for seismic function evaluation of piping design, procedure shown in picture 32 is used instead of following range shown by shading in figure 33.

-evaluation of seismic function of pipe support against maximum reaction

a) Maximum reaction (that is called releasing reaction, thereafter) is calculated when support is free.

b) Relative displacement (that is called releasing displacement, thereafter) is obtained using analysis and with consideration strength of free support against releasing reaction.

c) At first, axial force, bending moment, shear force and support reaction resulted from released displacement are computed and then, seismic function of system is evaluated. In this case, evaluation procedure of primary force may be applied.

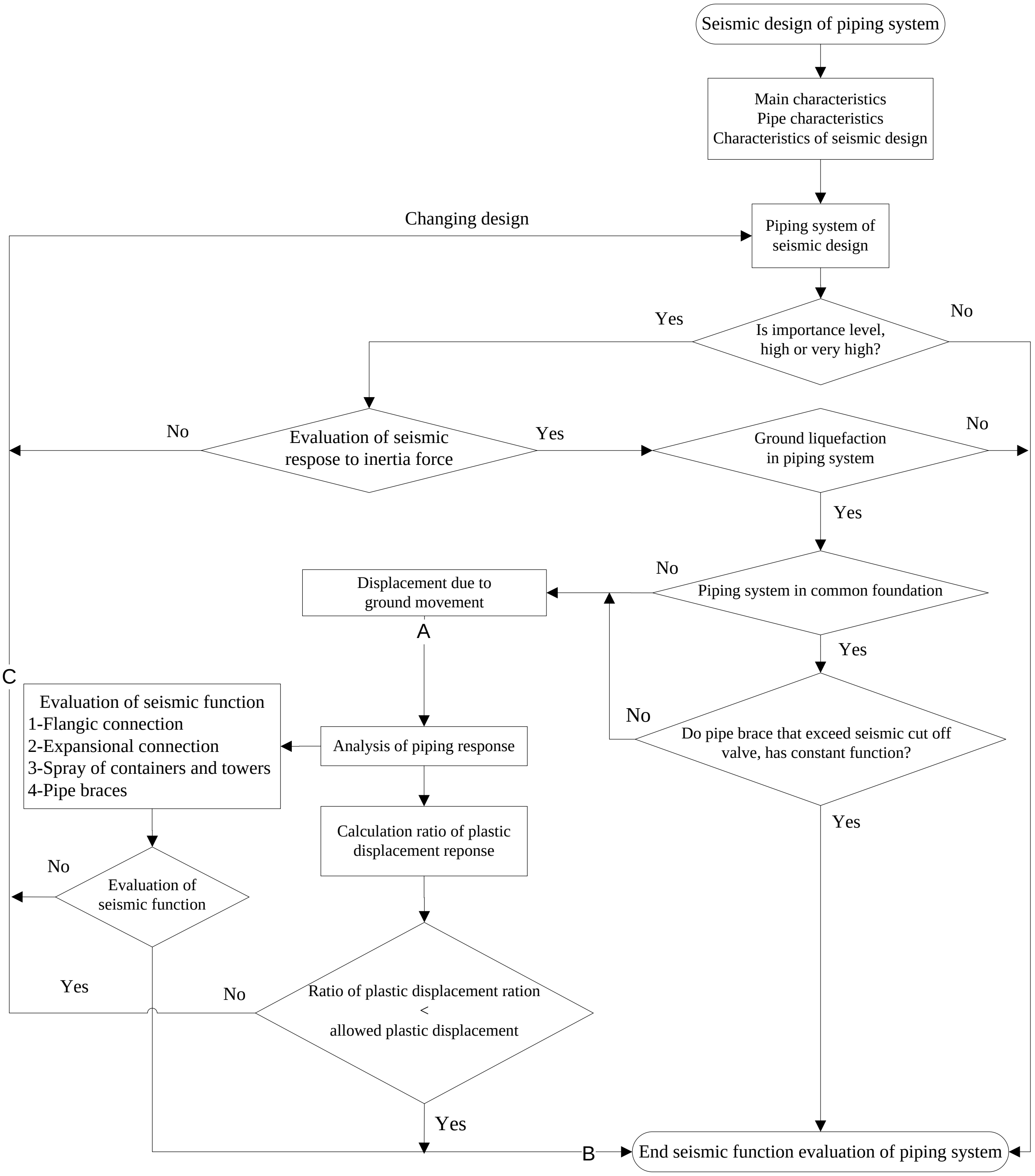


Figure 31-Steps of seismic function evaluation of piping system for ground movement

e) If intended seismic design is not fulfilled, design procedure must be changed. When only design method of free support is changed (reduction of maximum reaction) it is not necessary to go back to first step of seismic function evaluation. Seismic function evaluation of piping system reaction against inertia force must be performed separately.

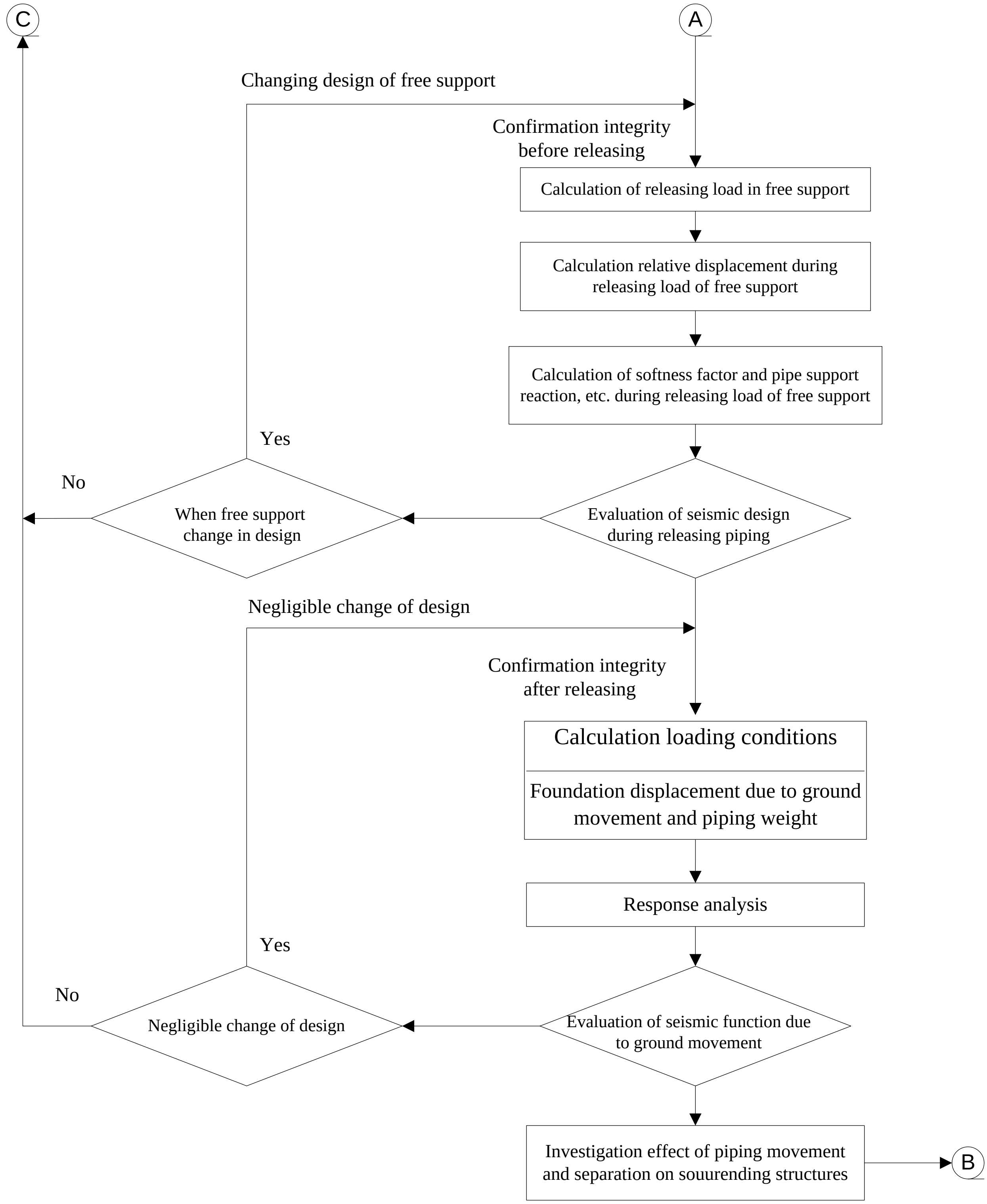


Figure 32-Flowchart of seismic function evaluation of piping system with free support due to ground movement

4-2-Confirmation seismic function of piping system against is a e ent ue to groun ove ent

a)In response of foundation displacement due to ground movement as a result of reaction and separation of piping system from free support and with assumption of missing support efficiency, pipe weight and load due to relative displacement in a point are composited and analyzed.

b) In evaluation of piping function against ground movement in common foundation, reaction and separation (and whatnot) of fixed support must be controlled. it is better that this control be performed in free support of piping.

c) If piping system is not built on common foundation, evaluation of piping system and piping support structure must be performed. In this manner, loading condition due to own weight of system must be evaluated as well as evaluation of displacement loading due to ground movement.

c) If seismic function is not fulfilled, design must be changed. If trivial changes imported in the design, evaluation is restarted from the first step.

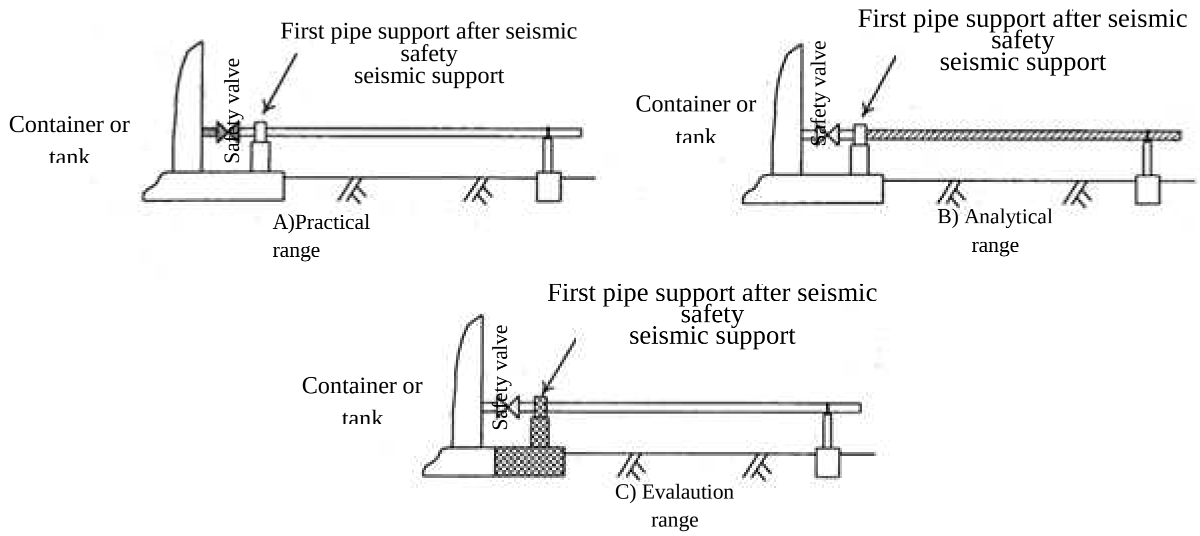


Figure 33-Piping system on common foundation

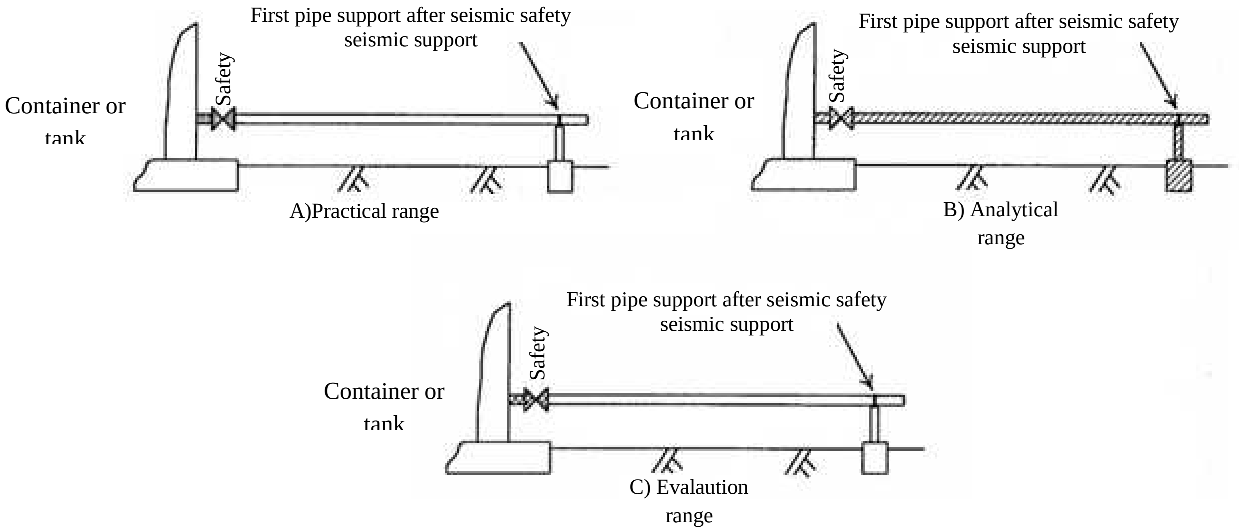


Figure 34-Piping system on non-common foundation

16-Displacement and relative displacement among foundation

1-Evalaution of foundation seismic function due to ground movement

Horizontal displacement, settlement and rotation (deviation) is occurred due to ground liquefaction, settlement and resultant horizontal displacement. Principally, piping support point must be built on common foundation but due to impossibility of whole system construction on a common foundation, piping foundation is constructed separately. So, evaluation of piping seismic function due to ground movement must be performed with attention to relative displacement between piping support and foundation.

2-Foundation displacement due to ground movement

Calculation of foundation displacement due to ground movement is as follow.

2-1-Settlement due to ground liquefaction and lateral extension

Settlement in extended foundation due to liquefaction and lateral expansion of the ground is calculated on the basis of presented procedure in the second phase. In adequate bearing capacity of stanchions, settlement due to liquefaction and lateral extension of the ground don’t occur.

2-2-Asymmetric settlement of foundation due to liquefaction and lateral extension of the ground

Asymmetric settlement of foundation due to liquefaction and lateral extension of the ground is calculated on the basis of presented procedure in the second phase. In adequate bearing capacity of piles, asymmetric settlement due to liquefaction and lateral extension of the ground don’t occur.

2-3-Lateral displacement due to lateral extension of the ground

Lateral displacement of extended foundation due to lateral extension of the ground is calculated on the basis of presented procedure in the second phase. Lateral displacement of piles due to lateral extension is calculated on the basis of response displacement in the second phase.

3-Relative displacement of foundation due to ground movement

Seismic function evaluation of piping system due to ground movement is performed through calculation of relative displacement among various supports of piping and displacement of original foundation.

In this manner, relation between horizontal relative displacement $\Delta_{12}(x)$ and vertical relative displacement among foundation of supports $\Delta_{12}(y)$ is as following:

Horizontal relative displacement:

$$\Delta_{12}(x)=(x_2+\theta_2H_2)-(x_1+\theta_1H_1) \tag{45}$$

Vertical relative displacement

$$\Delta_{12}(y)=y_2-y_1 \tag{46}$$

Parameters x_1, x_2, y_1, y_2, θ_1, θ_2, H_1 and H_2 are determined from following figure and table. Angle with clockwise rotation is considered positive.

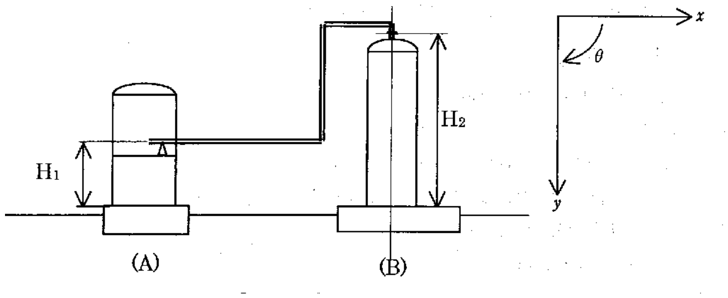


Figure 35-Pipe support structure

Table 22-foundation displacement of pipe support structure

Pipe support structure	(B)	(A)
Horizontal displacement (mm)	x_2	x_1
Settlement (mm)	y_2	y_1
Rotation angle (rad)	θ_2	θ_1

17-Flexibility factor and design procedure of embowed pipe

1- Flexibility factor of embowed pipe

In analysis of acceleration response, mean flexibility factor of in-plan bending, ex-plan bending and in-plan expansion are determined according to alternative negativeness or positiveness of displacement and smallness of torsion angle. However, value difference of related flexibility factor of in-plan bending, in-plan expansion and ex-plan bending cannot be neglected, because relative displacement is high and occur in one direction in response analysis of ground movement.

Although flexibility factor is obtained from relations 47 to 49 on the basis of deformation mode of embowed pipe but for simplicity of design and disregarding deformation mode, flexibility factor for in-plan bending can be obtained as safety margin from relation 47.

$$k_p = [(1.28h + 0.03)\theta_D(90/\alpha) - 0.66h + 0.75]\frac{S_0}{S_y}k_e \geqq k_e \qquad (47)$$

Where
α Angle of embowed pipe (degree unit)
θ_D angle variation of embowed pipe (degree unit)
k_e flexibility factor in elastic deformation
2-Analysis procedure
2-1-For displacement analysis of foundation due to liquefaction, relative displacement in the horizontal and vertical directions must be considered coincidently.
2-2- Flexibility factor of embowed pipe is obtained with consideration of corner angle and composition of relative displacement in the horizontal and vertical directions.
2-3-Composition of loads due to foundation displacement from ground displacement by a normal load is not considered. Confirmation of seismic function of system with consideration of piping weight (and whatnot) in essential when high weight is not imposed on pipe support or upward drive in vertical direction as a result of boiling phenomenon such as piping system with free support.

18-Details of allowable angle of embowed pipe

In evaluation of seismic function due to ground movement, allowable angle of embowed pipe is equal to torsion angle corresponding to plastic strain of 5%. Corner angle θ_{al2}, corresponding to equivalent plastic strain ε_{pal2} of embowed pipe is given by relation 48.

$$\theta_{aL2} = 29.1\frac{\varepsilon_{paL2}^{0.829}}{h^{0.456}} \qquad (48)$$

Where, ε_{pal2} is equivalent plastic strain and θ_{al2} is corner angle. This relation is for 90 degree pipe curve. For pipes with curve degree of 30 and 45, angular displacement is given through interpolation of corner angle value. In elbow of 90 degree with consideration of allowable ductility factor corresponding with plastic strain equivalent with 5%, allowable angle equivalent with ε_{pal2} 0.05 is obtained as relation 49.

$$\theta_a = \frac{2.43}{h^{0.46}} \qquad (49)$$

Table 23 present the relation between allowable angles (embowed pipe with right angle) and characteristic value of bending deformation in long arm elbow of 90 degree with nominal thickness of list 40.

Table 23-Allowable angle of long arm elbow of 90 degree (nominal thickness of list 40)

Allowable angle (degree)	Characteristic value of bending deformation	Wall thickness (mm)	External diameter (mm)	Nominal diameter (A)
3.62	0.420	3.7	48.6	40
3.83	0.371	3.9	60.5	50
3.74	0.392	5.2	76.3	65
3.89	0.360	5.5	89.1	80
4.04	0.331	5.7	101.6	90
4.15	0.312	6.0	114.3	100
4.34	0.283	6.6	139.8	123
4.52	0.260	7.1	165.2	150
4.77	0.231	8.2	216.3	200
4.95	0.213	9.3	267.4	250
5.11	0.198	10.3	318.5	300
5.10	0.200	11.1	355.6	350
5.10	0.200	12.7	4.6.4	400
5.10	0.200	14.3	457.2	450
5.22	0.189	15.1	508.0	500
5.34	0.181	15.9	558.8	550
5.31	0.183	17.5	609.6	600
5.32	0.182	18.9	660.4	650

19-Procedure of seismic function evaluation of flange connection due to ground movement

An important point that must be kept in mind regarding seismic strength of flange connection is that leakage due to applied loads in the connection of pipes must be prevented. In this case, leakage evaluation is performed through consideration of tensile axial force and bending moment formed due relative displacement between pipe support and piping support due to ground movement in the flange connection.

Evaluation of leakage is done with a method resemble to analyse of acceleration response in clause 5-3-6. Figure 36 shows flowchart of seismic function evaluation in flange connection.

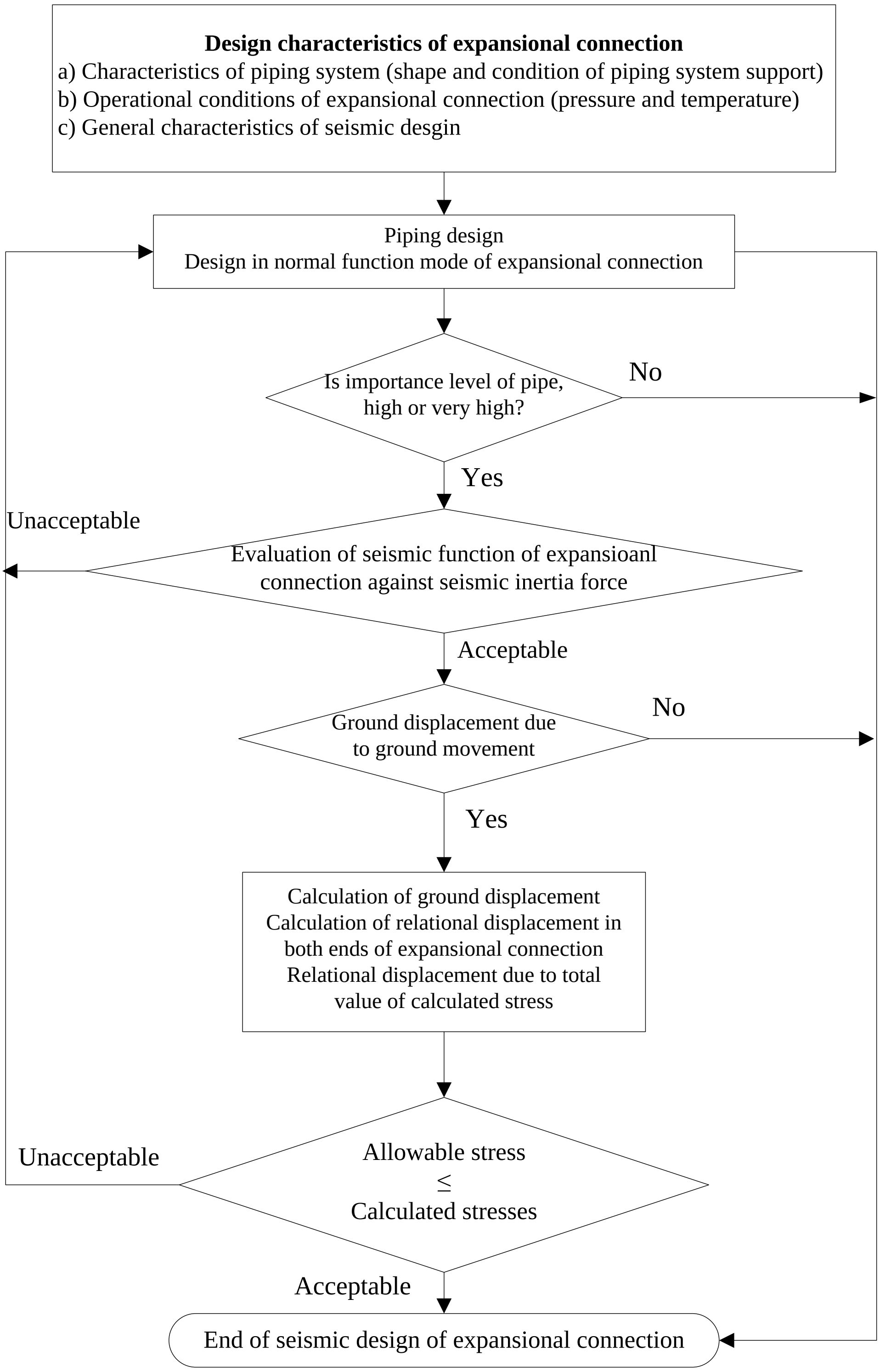


Figure 36-Procedure of seismic function evaluation of flange connection

20-Details of procedure of expansional connection evaluation due to ground movement

Relative displacement in both ends of expansional connection must be lower than allowable relative expansional displacement due to ten times vibration in expansional connection. In this case, inertia force and response displacement can be evaluated separately.

As well as direction in which relative displacement does not occur, it is necessary that connection must have adequate strength against reaction of calculated response.

1-Procsedure of seismic function evaluation of expansional connection due to ground movement

Procedure of seismic function evaluation of expansional connection due to ground movement is performed according to clauses 1-1 to 1-6 as shown in figure 37.

1-1- Seismic function evaluation of expansional connection with high importance due to ground movement is performed after confirmation of connection seismic function against seismic inertia force

1-2-Function of foundation against displacement due to ground movement is evaluated.

1-3-Displacement of foundation is calculated.

1-4-Analysis of piping system response is performed with concurrent consideration of horizontal and vertical displacement of foundation and relative displacement calculated in both ends of expansional connection.

1-5-Total value of produced stress in accordion part due to relative displacement between two ends of expansional connection is computed.

1-6-It must be confirmed that total value of computed stress is lower than allowable stress value.

2-Calculation method of total value of stress in expansional connection

Calculation method is as calculation method in section 8-3-5.

3-Calculation method of allowable stress value in expansional contact

Value of maximum axial stress produced in accordion part must be lower than twice of corresponding allowable stress with ten times replication.

3-1-Allowable stress S_a for seismic design of axial stress produced in accordion part of expansional contact made of hard steel (carbonized), low alloy steel, ferritic stainless steel and steel with high tensile strength is equal to following values:

a) When minimum tensile strength is equal or lower than 551.6 MPa, S_a=2×3999=7998 MPa

b) When minimum tensile strength is between 792.9-896.3 MPa, S_a=2×2896=5792 MPa

c) When minimum tensile strength is between 551.6-792.9 MPa, S_a is obtained from values in a) and b) appropriately.

3-2-Allowable stress in seismic design of produced axial stress in expansional connection accordion made from austenitic stainless steel, nickel alloy (Ni-Cr-Fe and Ni-Fe-Cr alloy) and nickel-cupper alloy is equal to S_a=2×4882=9746 MPa

4-Estimation of seismic function of expansional connection due to ground movement

For estimation of seismic function of expansional connection due to ground movement, total value of calculated stress in connection from clause 2 must be lower than value of calculated stress from clause 3. In this case, accumulative damages inflicted on connection due to loads of pervious earthquakes aren't considered.

As well, displacement bracing component in expansional connection in the direction that relative displacement isn’t absorbed must have sufficient strength (bracing through bolt rack, bracing through rack plate, adjustment ring, etc.) to maintain relevant function. This function includes endurance of calculated reaction from analysis of piping system response due to ground movement.
5-Designing expansional connection for purposes other than ground displacement
Pipe support in designing expansional connection for purposes other than ground displacement must be deigned in such a way that displacement exceeding tolerance limit due to seismic movement is not applied on expansional connection or support has adequate strength to maintain function of connection displacement bracing member. This function involves endurance calculated reaction from analysis of piping system response due to ground movement.

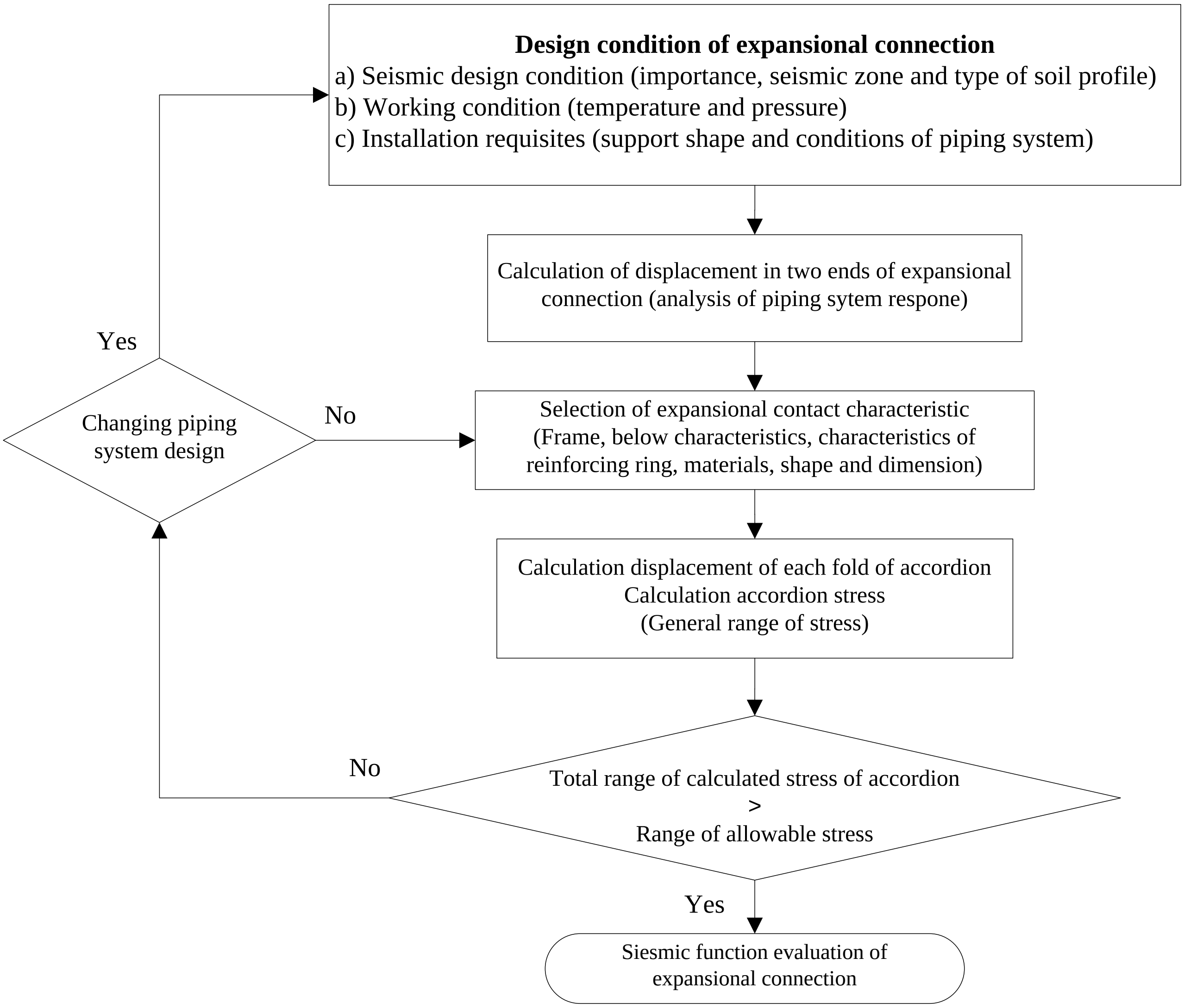


Figure 37-Flowchart of seismic function evaluation of expansional connection due to ground movement

21-Details of evaluation procedure of spray of containers and towers due to ground displacement

1-In seismic design, for displacement evaluation of pipe support point in spray of containers and towers due to ground displacement, sum of primary and secondary stress intensity resulted from related displacement must be lower than or equal to allowable stress intensity $4S_y$. In this case, separate evaluation with estimation of relevant inertia force is possible.

2-Evaluation spray of containers and towers using simplified method (the Bairard method) is performed on the basis of thin shell theory or analysis of finite element method (FEM) (and whatnot) and also according to section 5-3-9 in the subject of acceleration response analysis.

3-Figure 38, presents evaluation procedure of relative displacement of pipe support point in seismic function evaluation of spray of containers and towers due to ground displacement with simplified method based on the Bairard method. Also, in detailed analyses, evaluation is performed on the basis of similar method.

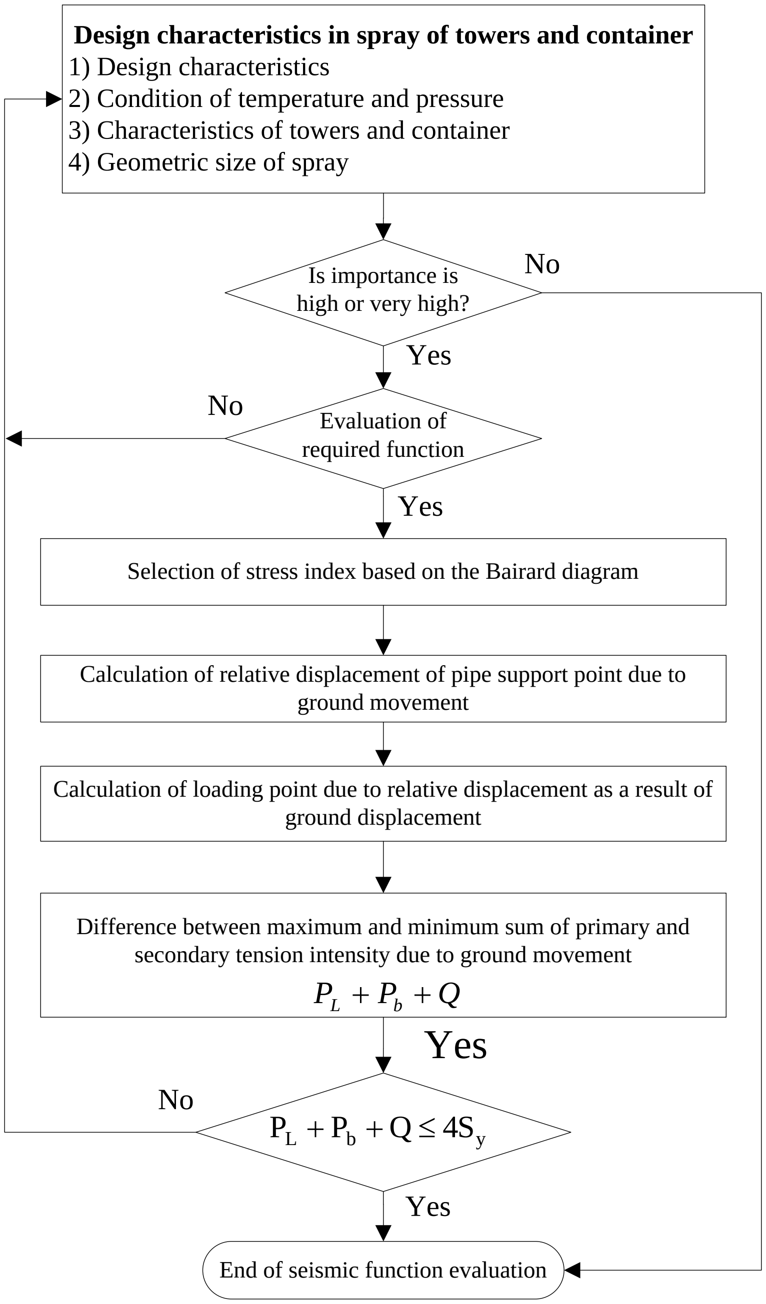


Figure 38- Seismic function evaluation of spray of towers due to ground movement

22-Seismic function evaluation of pipe support due to ground movement

1-Flowchart of procedure of seismic function evaluation of pipe support due to ground movement

Figure 39 shows pipe support evaluation flowchart due to ground movement.

1-1-Piping system evaluation due to ground movement is performed after confirmation acceptability of system evaluation against seismic inertia force.

1-2-Seismic function evaluation of piping system located on foundation due to ground movement is performed with determination of occurrence probability of liquefaction or ground displacement (ground movement)

1-3-If piping system together with seismic design equipment are founded on common foundation, it is required that fixation function of pipe support of equipment be performed in the nearest distance after seismic safety valve due to related ground displacement (ground movement).

1-4-If piping system together with seismic design equipment aren't founded on common foundation, it is required pipe support and its appurtenance have proper seismic function against relative ground displacement based on position of support point.

1-5- like evaluation of inertia force, seismic function of pipe support is evaluated through adjustment of allowable condition according to pipe support function.

2-Evaluationg seismic function of fixed pipe support

In the cases that ground displacement effects are predictable, piping system must be built on common (integrated) foundation.

In this case, first pipe support next to seismic safety valve in piping range is built on common foundation and support beyond system range is fixed.

On the other word, high deformation is occurring due to ground displacement in piping that is not located on common foundation. So it is required to be confirmed that piping located on common doesn't leak due to deformation resulted from deformation of pipe support that isn't located on common foundation. So with consideration the necessity for smallness of pipe fixed support deformation due to ground movement and endurance of reaction in three directions and bending moment surrounding three arises by piping, seismic function evaluation of pipe fixed support is done in such a way that produced stress due to reaction and moment resulted from ground displacement in piping must be lower than yield load.

In this case, piping reaction against ground displacement (and other things) is principally obtained from response analysis but it may be replaced by yield strength of total plastic moment of piping.

3-Seismic function evaluation of pipe reinforced support

Seismic function evaluation of pipe reinforced support against liquefaction due to ground displacement for non-occurrence of rupture is done in such a way that plastic deformation resulted from piping reaction in support must be lower than allowable plastic deformation.

4- Seismic function evaluation of free support

Free support is used to prevent from leakage in piping by reduction loading effects through separation of appurtenance seat of U-shaped bolt and removing support strength against ground movement, provided that support function for inertia force due to ground movement is maintained. However, this support must be in such conditions that have loading condition characteristics of pipe support (and other things).

5-Adjustment allowable conditions

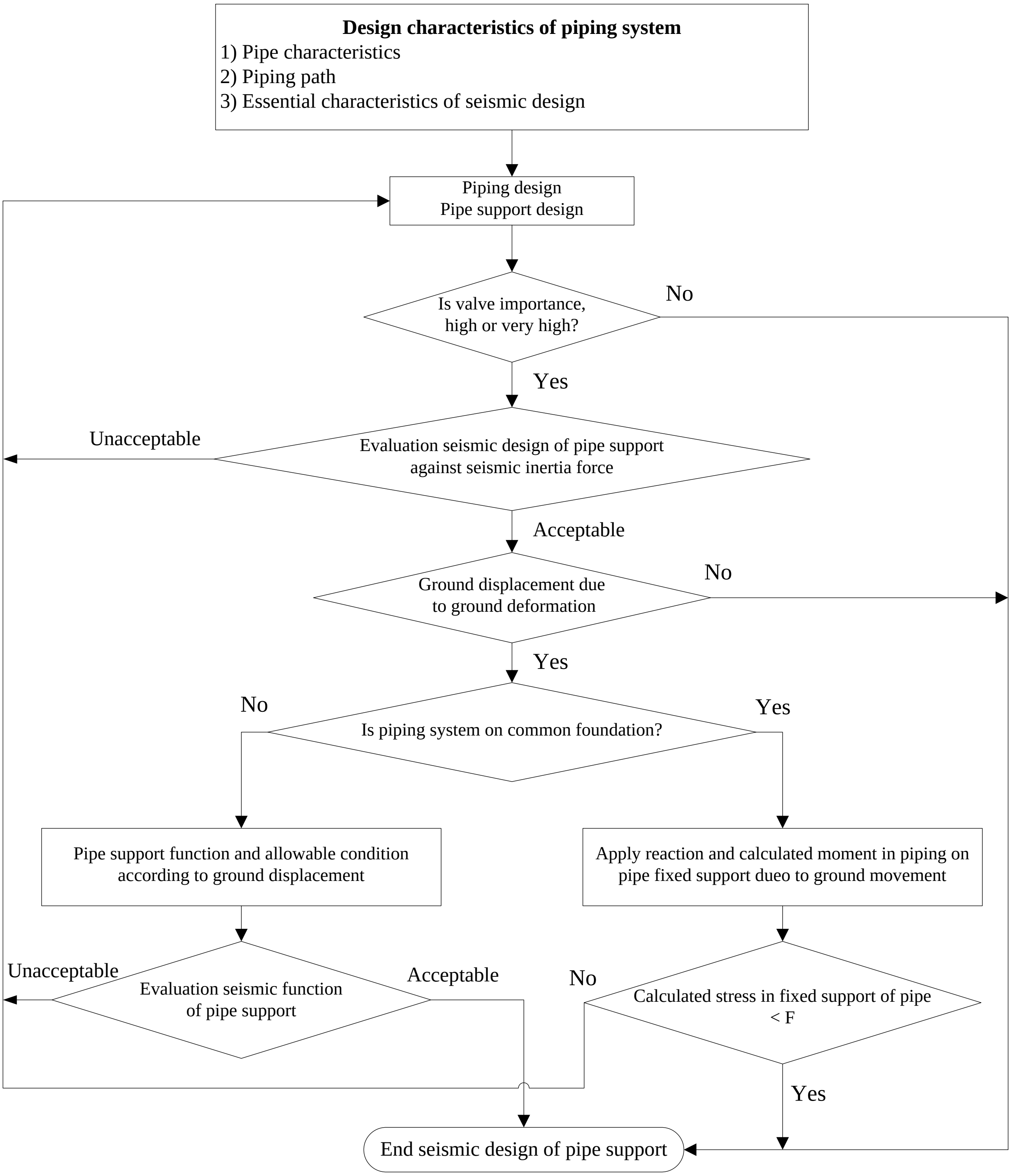


Figure 39-Flowchart of seismic design Evaluation of pipe support due to ground movement

23-Yeild strength evaluation of stud

Stud safety is evaluated in such a way that final moment capacity of stud M_u, according to relation 50, must be higher than multiplication of plastic moment of total section of M_{pc} pillar in safety factor α, also final shear strength of stud, Q_u, according to relation 50 must be higher than shear force in yield point, Q_{mu}.

$$M_u \geq \alpha \times M_{pc} \tag{50}$$

$$Q_u \geq Q_{mu} \tag{51}$$

Where

M_u final moment capacity of stud (N.m)

M_{pc} safety factor (SS400、SN400:α=1.3、SM490、SN490:α=1.2)

Q_u final shear strength of stud (N)

Q_{mu} shear force in yield point when plastic juncture is formed in the pillar (N).

When relations 45 and 46 are applied, plastic juncture is formed in the pillar and stud isn't destructed. D_s value used in determination of yield strength (relation 10-15) is determined by weight of stud's upper stories. If these relations aren't fulfilled, plastic juncture is formed in the stud and 0.05 is added to D_s of the first story during determination of yield strength.

For fulfilling stud function, it is required to consider these states

a) Preventation from decaying foundation concrete

b) Preventation from shear fracture of stud

c) Preventation from destruction of pillar floor

Following figure shows steps of safety evaluation of stud and its upper structures.

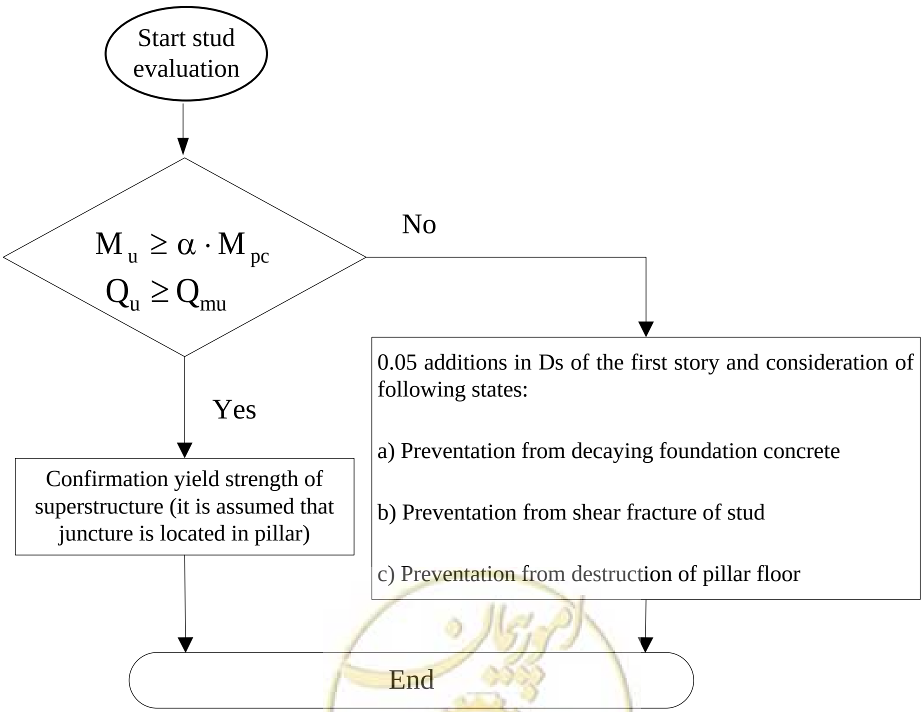


Figure 40-steps of seismic evaluation of stud and superstructure

1-Calculation of final moment capacity of stud M_u and final shear strength of stud, Q_u for three states in figure 41.

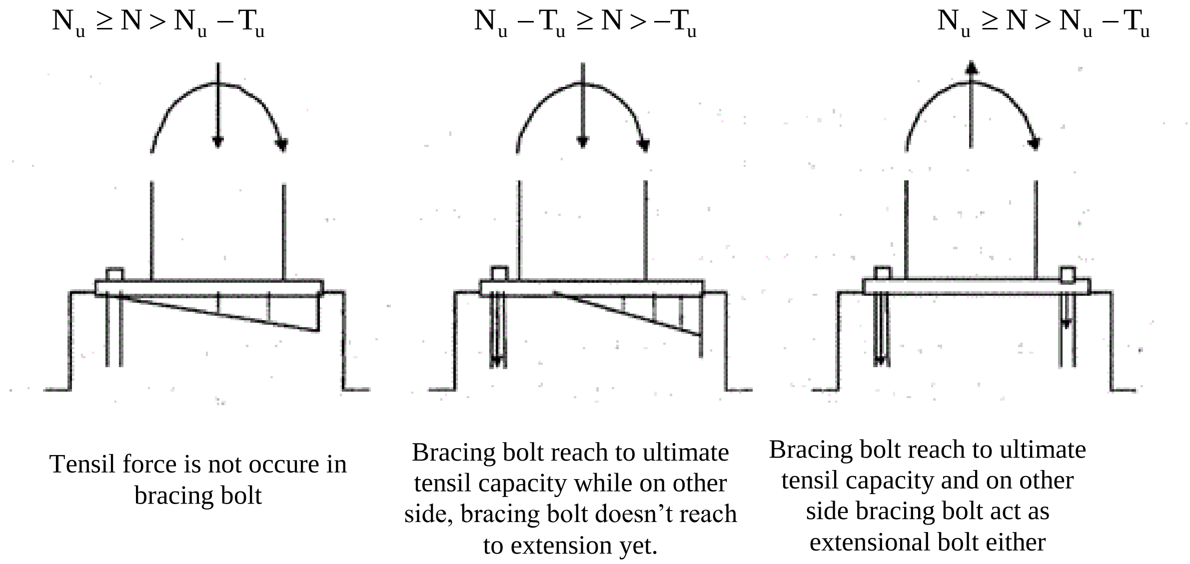


Figure 41–Stress in proximity of stud

Where

N	Axial tensile of pillar in yield point
N_u	Final compressive capacity of foundation concrete
T_u	Final tensile capacity of bracing bolt

1-1-final moment capacity M_u of stud

a) $N_u \geq N > N_u - T_u$

$$M_u = Ndt\left(\frac{N_u}{N} - 1\right)\ (N.m) \qquad (52)$$

Where

N_u	Final compressive capacity of foundation concrete N_u=0.85bDF$_c$ (N)
N	axial extension of pillar in yield point (N)
dt	distance between center of pillar cross section and center of gravity in extensional face of bracing bolt (mm)

(b) $N_u - T_u \geq N > -T_u$

$$M_u = T_u dt + \frac{(N + T_u)D_{bpl}}{2}\left(1 - \frac{N + T_u}{N_u}\right)\ (N.m) \qquad (53)$$

Where

D_{bpl}	length of pillar floor (mm)
T_u	final extensional capacity of bracing bolt (N) $T_u = n_t A_b F_{ab}$
b	width of pillar floor (mm)

F_c design strength of concrete (mm^2)
A_b cross section of bracing bar (mm^2)
F_{ab} strength of bracing bolt (N/mm^2)
n_t number of bolts

c) $-T_u \geq N > -2T_u$

$$M_u = (N + 2T_u)dt \quad (N.m) \tag{54}$$

1-2-Final shear force of stud Q_u

Final shear force of stud Q_u must be equal to maximum value of frictional strength Q_{fu} and final shear force of bracing bolt Q_{su}.

$$Q_u = \max(Q_{fu}, Q_{su}) \tag{55}$$

a) $N_u \geq N > N_u - T_u$

$$Q_{fu} = 0.5N_y \tag{56}$$

$$Q_{su} = 2S_u \tag{57}$$

Where

S_u final shear yield strength of bracing bolt

b) $N_u - T_u \geq N > -T_u$

$$Q_{fu} = 0.5(N + T_u)\ ,\ Q_{fu} \leq 0.5(N_u - T_u) \tag{58}$$

$$Q_{su} = S_u\left\{1 + \sqrt{1 - \left(\frac{T}{T_u}\right)^2}\right\} \tag{59}$$

$$T = N_u - T_u - N,\ \ T \leq T_u \tag{60}$$

c) $-T_u \geq N > -2T_u$

$$Q_{fu} = 0 \tag{61}$$

$$Q_{su} = S_u\sqrt{1 - \left(\frac{T}{T_u}\right)^2} \tag{62}$$

Where $T = -T_u - N$

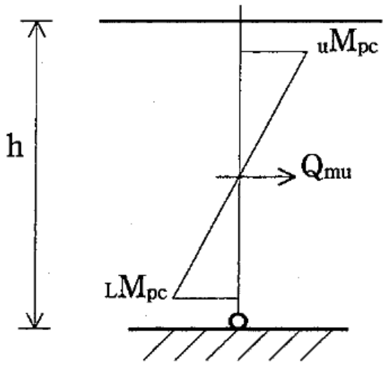
$$S_u = n_t A_b F_{ab} / \sqrt{3} \quad (N) \tag{63}$$

2-Plastic moment of whole pillar M_{pc}

Plastic moments of whole pillar are obtained with consideration of axial force.

3-Shear force of pillar in yield strength Q_{mu}

Shear force of pillar in yield point is obtained from horizontal yield point of first pillar of frame with consideration of axial force



$$Q_{mu} = \frac{_{u}M_{pc} + {}_{L}M_{pc}}{h}$$

Figure 42-Shear force of pillar in yield point

For creation of plastic deformation capacity in stud, it is needed that bracing bar have adequate capability for length variation. This capability is a function that threaded section isn't failed until yield of all parts of stalk. For this purpose, the ratio of yield to final strength of bracing bolt material must be equal or lesser than 0.7.