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Japan International Cooperation Agency (JICA)
Ministry of Housing and Urban Affairs
National Earthquake Engineering Research Center

# A Study of Seismic Microzoning of the Wilaya of Algiers in the People's Democratic Republic of Algeria

## Final Report

## Volume VI

## TECHNICAL GUIDELINE FOR SEISMIC MICROZONING

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# 1. Definition of Seismic Microzoning

Although it has been more than 40 years since the term “Seismic Microzoning” was initially used, its definition still remains somewhat unclear. Seismic Microzoning surveys conducted in earthquake-prone countries around the world have produced many drawings referred to as Seismic Microzoning Maps, but the contents of each differ significantly. For example, a survey drawing may be called a Seismic Microzoning Map when it is a map of the distribution of expected earthquake ground motion values on the bedrock as derived by analysing fault distribution in and around the survey area, historical earthquake catalogues, and so on. In another case, it could also be based on geological maps without assuming any specific earthquake model, the relative risk of earthquake motion, liquefaction potential, and slope failure potential, e.g., according to the rank of A to E.

Apart from the above exclusive coverage of natural disasters, there are also cases where structures susceptible to earthquake damage are assessed to quantitatively evaluate damage at the time of earthquake occurrence and to indicate their distribution on a drawing. These could include houses, schools, hospitals, and other buildings; bridges/roads, railways, piers, and other civil engineering structures; water pipes, city gas pipes, electric power lines, and other lifeline facilities. The same term, Seismic Microzoning, thus has a wide variety of definitions.

Seismic Microzoning conducted to date can be classified from a number of viewpoints.

a) It can be classified by coverage as follows:

a-1) Exclusive coverage of natural hazards

- Earthquake motion
- Liquefaction potential
- Slope failure potential
- Tsunami height

a-2) Coverage of damage to structures caused by natural hazards in addition to the natural hazard itself

- Buildings (including houses, hospitals, schools, and public offices)
- Civil engineering structures (including roads, bridges, flyovers, railways, piers, airports, and embankments)
- Lifelines (including water supply, sewage, city gas, electric power, and telephone)

a-3) Further coverage of secondary damage caused to the above structures

- Human damage (including people who died, were injured, or lost their own houses)
- Fires after earthquakes (including the number of occurrences of fires and buildings lost due to fire)

a-4) Coverage in terms of economic/social influences other than physical damage

- Amount of direct damage due to earthquakes
- Amount of damage due to slowing economic activities (including effects of suspended commercial trade or disrupted factory production)
- People who need to be cared for mentally

b) Quantitative/qualitative evaluation

b-1) Quantitative evaluation

A quantitative evaluation is an evaluation of the resistance to earthquake motion, damage to a building, and so on, determined as an absolute amount. For earthquake motion, earthquake intensity is calculated and for building damage the number of damaged buildings is determined. In some final Seismic Zoning Maps, however, such indications may be made not in absolute terms but according to the better understood “rank”.

b-2) Qualitative evaluation

In qualitative evaluation, the relative risk of damage rather than absolute amount is evaluated. For example, based on the subsurface layer, the potential for earthquake motion is ranked, e.g., A for very soft soil, C for bedrock, etc. This is an evaluation where the overall distribution of a rank and comparison between points are meaningful.

c) Objective

c-1) Zoning for Seismic Code and other standards

The Seismic Code and earthquake insurance geographical division system divide the entire country into several geographical regions and specifies the level of earthquake risk and the zone factor to intensify the input value of earthquake motion for each. In Seismic Microzoning conducted to determine such a level or value, it is considered more important to evaluate a wide region with a unified measure and clearly define the relative differences in susceptibility to earthquakes rather than to calculate their quantities. This means not establishing deterministic earthquake scenarios assuming a specific epicentre but rather to develop a probabilistic model, which statistically deals with influences of any different seismic sources. The general practice is the evaluation of natural hazards which affect the degree of damage to a building.

c-2) Zoning for earthquake disaster prevention plans/measures

When developing effective earthquake disaster prevention plans and disaster management programs, it is important to know in advance what will happen to the regions in question when earthquakes occur. In the case of wind and flood damages, although they are other types of natural hazards, damages and where they are likely to occur can be empirically predicted since they may occur regularly, for example almost annually. Contrary to this, earthquakes causing serious damage occur irregularly. In Algiers, for instance, its eastern region suffered damage due to the Boumerdes earthquake in 2003 but more disastrous earthquakes impacting on the ALGER CENTER have dated back to 1716. Also, experiences due to more recent suffering have receded in people’s minds as some 10 to 20 years have passed since such events. As earthquakes therefore occur at much longer intervals than the human lifespan, it is difficult to adapt experiences of such disasters in developing effective countermeasures. Hence, Seismic Microzoning is conducted to realistically determine what will happen to cities if major earthquakes were to occur.

An earthquake is a natural phenomenon that has a substantial influence on the wider area. Its impacts are not simply confined to hazards due to earthquake motion and land liquefaction, but also to physical damage to buildings. Therefore, it generally causes disruptions to lifelines, impacting on the social life and economic activities, and results in a rapid expansion of suffering throughout society. The level of damage depends not only on natural conditions, including soil conditions, the status of buildings, roads, and other artificial structures, but also the emergent response activity to the disaster. Seismic Microzoning, therefore, often analyzes the physical distribution of earthquake motion, the extent of damage, etc. as a basis to develop a time-lapse earthquake disaster scenario extending over the minutes, hours, days, and weeks that have elapsed after the earthquake. In such cases, Seismic Microzoning is a prior comprehensive simulation of damage due to the occurrence of earthquakes.

In the light of the above classification, the project of “A Study of Seismic Microzoning of the Wilaya of Algiers in the People’s Democratic Republic of Algeria” can be classified as;

a)-3: Coverage of natural hazards, damage to structures and secondary damages,
b)-1: Quantitative evaluation, and
c)-2: Zoning for earthquake disaster prevention plans/measures.

# 2. Data Collection

## 2-1 Scale of Base Map and Grid Size

The suitable base map scale for a seismic microzoning study varies depending on the extent of the study area. For the assessment of large areas, the use of small-scale maps gives little benefit in spite of its cost and difficulty. On the other hand, hazard analysis of small areas requires sufficiently detailed maps.

Square grids are often used as the unit of analysis. Grid size differs depending on the extent of the study area. It is not useful to use needlessly small grids since the use of small size grids requires a greater amount of detailed data.

Table 2-1 shows examples of appropriate base map scales and grid sizes for seismic microzoning studies. For example, the area of the seismic microzoning study of Algiers is around 230 $km^2$ and the 250 m sq. grid was adopted.

Table 2-1 Desirable Base Map Scale and Grid Size for Study Area

| Study Area | Scale of Base Map | Grid Size |
|---|---|---|
| 100 - 400 $km^2$ | 1/25,000 | 250m sq. |
| 400 - 1,600 $km^2$ | 1/50,000 | 500m sq. |
| 1,600 - 6,400 $km^2$ | 1/100,000 | 1km sq. |
| 6,400 - 25,600 $km^2$ | 1/200,000 | 2km sq. |

If the necessary data will be collected in printed maps, the scale of those maps should be smaller than the scale in the above table.

## 2-2 Necessary Information

A variety of information and data are required for a seismic microzoning study. The contents and the precision of the information and data are different according to the kind of hazard/damage to be studied and the methodologies adopted. Necessary information is roughly classified as follows:

a) Basic Information

- Administrative Boundaries
- Land Use
- Population/Building Census
- Earthquake Hazard/Damage Record, etc.

b) Seismological Information

- Active Faults Map
- Seismotectonic Map
- Historical/Instrumental Earthquake Catalogue
- Strong Motion Record, etc.

c) Geological Information

- Topography Map
- Geological Map
- Bedrock Distribution, etc.

d) Geotechnical Information

- Boring Log
- PS Logging
- Soil Property
- Groundwater Levels, etc.

e) Building Information

- Building Inventory
- Building Distribution
- Seismic Code and Standards, etc.

f) Infrastructure Information

- Bridge Inventory
- Road Distribution Map
- Water Pipeline Distribution Map and Inventory
- Sewage Pipeline Distribution Map and Inventory
- Gas Pipeline Distribution Map and Inventory
- Electric Power Line Distribution Map and Inventory
- Telephone Line Distribution Map and Inventory, etc.

# 3. Hazard Analysis

Hazard analysis has various methods as well as approaches. In the 1970s when Hazard Analysis became a practice in Japan and California, evaluation was undertaken only under empirical rules, using limited data. A variety of approaches have since been developed and used on the strength of various accumulated data, developed analytical approaches, advanced computers, the introduction of GIS, and so on. Even in these regions, however, the new sophisticated approaches are not always used. This is because more advanced analytical approaches require more data and the results become more sensitive to the accuracy of the data. Applying advanced analytical approaches in cases where sufficient data cannot be obtained or where such data are inaccurate introduces problems and it is therefore possible to make serious mistakes. As a result, an advanced analytical approach should not be applied recklessly but a proper approach should be selected in accordance with the available data.

Under these circumstances, there is a large variety of hazard analysis methods that can be chosen from according to objective area, condition of data, and purpose etc. In this guideline, the methods were roughly classified to several groups in order to select the most suitable method for this study area.

## 3-1 Scenario Earthquake

How to assume a scenario earthquake differs completely from one region to another. The selection of the method to establish the scenario earthquake is strongly affected by the seismotectonic and seismic conditions in addition to the availability of necessary data. For example, even if a precise database of historical earthquake catalogues is available, it is unrealistic to establish the scenario earthquake based on the historical earthquakes if only minor damage has been experienced at the site. In many cases, several procedures are used together.

**[Method-1] Based on the historical damaging earthquakes**

This method depends on the characteristic earthquake model. The characteristic earthquake model suggests the maximum earthquakes that occur at a particular fault have almost the same magnitude and the same time interval. This method assumes the re-occurrence of an earthquake that affected the study area in the past or repeatedly occurred in the past and is also expected to occur in the future.

**[Method-2] Based on an active fault**

An active fault is a fault that became active or that has shown repeated movement over the last 1 to 2 million years and is likely to have another earthquake sometime in the future. The active faults are, naturally, suited for scenario earthquakes. In general, there are many faults on geological maps but most of them are not “active”. The identification of whether the fault is active or not is the key point of this method.

**[Method-3] Based on the seismotectonic and seismological information**

Especially in offshore areas, it is difficult to find faults and identify whether they are active or not. In a seismologically active area, e.g. plate boundary, the epicentral distribution indicates the location. In recent years, ocean bottom surveys have been conducted in several areas and many important faults were found, however it is still difficult to identify the activity.

In these cases, the crustal movement by GPS observation is the most important information to estimate the activity of the fault. The seismotectonic model is helpful to establish the scenario earthquake.

The necessary data in each method, which is outlined above, and the grade of necessity is shown in Table 3-1.

Table 3-1 Necessary Data and their Availability - Scenario Earthquake -

| Data | Necessity in each Method | | | Availability in Algiers | |
|---|---|---|---|---|---|
| | Method-1 | Method-2 | Method-3 | | |
| | Based on the Historical Damaging Earthquakes | Based on the Active Fault | Based on the Seismotechtonic and Seismological Information | Quality | Quantity |
| Earthquake Hazard Record | ◎ | △ | △ | △ | △ |
| | | | | | |
| HIstorical Earthquake Catalogue | ◎ | ○ | ○ | ○ | ○ |
| | | | | | |
| Instrumental Earthquake Catalogue | △ | △ | ◎ | ○ | ◎ |
| | | | | | |
| Location/Length of Active Faults | △ | ◎ | | ○ | ◎ |
| Recurrence Interval/Last Event of Active Fault | △ | ○ | | × | × |
| | | | | | |
| Crustal Movement Record (GPS) | | △ | ◎ | ◎ | ○ |
| | | | | | |

◎ : The data in Study Area is indispensable
○ : Necessary but can be estimated or substituted by the data in other area
△ : Desirable

◎ : OK
○ : Almost OK, but additional information is recommended
△ : Exist, but not enough
× : Not exist or scattered

**[In Algiers]**

The availability of the data is shown in Table 3-1.

In Algiers, its eastern region suffered damage due to the Boumerdes earthquake in 2003 but more disastrous earthquakes impacting on the ALGER CENTER have dated back to 1716. The 1716 earthquake is too old and there is not enough information to establish a scenario earthquake based on this earthquake.

Therefore, the scenario earthquake in Algiers was decided based on the active fault [Method-2]. Figure 3-1 shows the location and inferred surface traces of active faults in and around Algiers. Blida Fault, Sahel Fault and Thenia Fault are inland faults, so the locations were studied by geologists. Khair al Din Fault and Zemmouri Fault were found by recent ocean bottom surveys. The activity of these faults was studied based on the seismotectonic modelling [Method-3] (Figure 3-2).

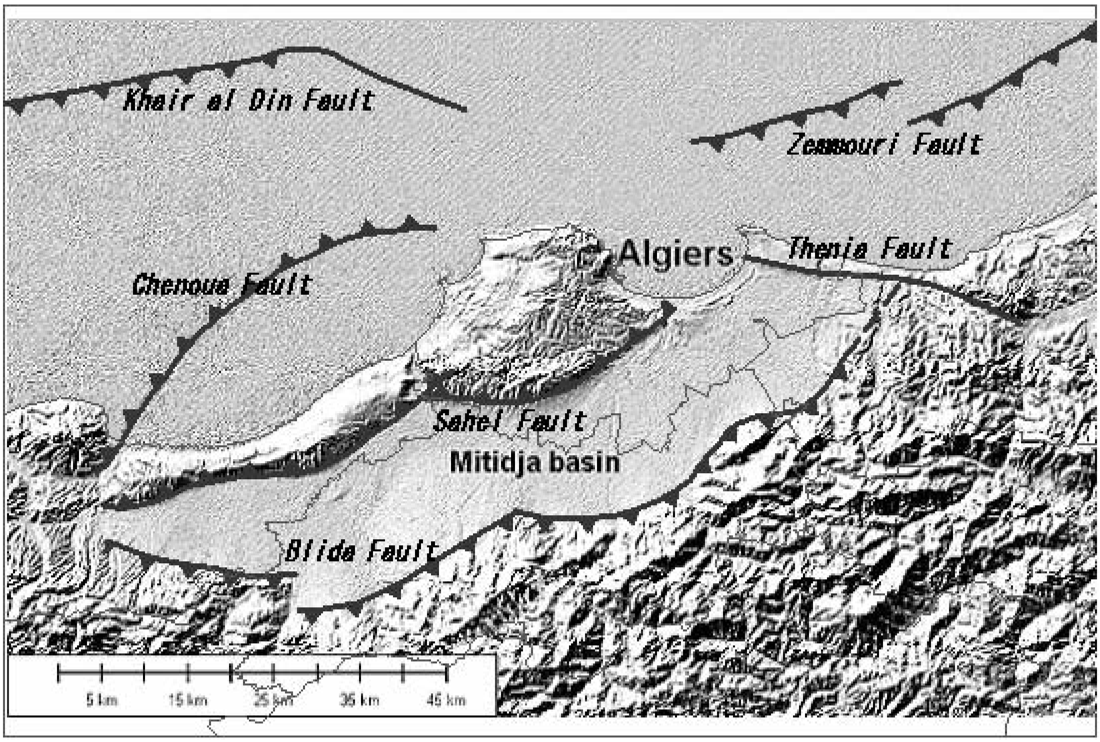


Figure 3-1 Location and Infrared Surface Traces of Faults
(Background Image: SRTM DEM)

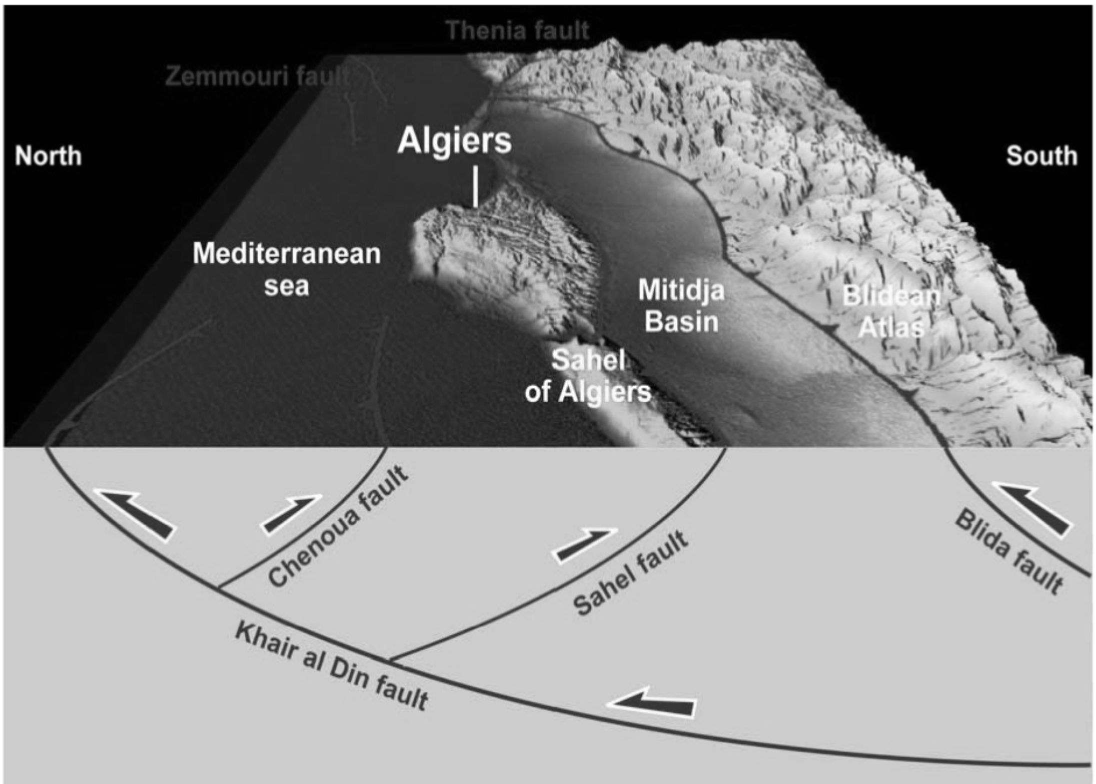


Figure 3-2 Block Diagram Showing 3D Geometry of the Proposed Seismotectonic Model

## 3-2 Bedrock Motion

### [Method-1] Empirical Attenuation Function (PGA, PGV, Sa)

In general, the earthquake ground motion is larger if the magnitude of the earthquake is larger or the epicentral distance is smaller. The empirical relation between the earthquake motion, the epicentral distance and the magnitude, the so-called attenuation formula, was developed from observed records and proposed by many researchers. The attenuation formula for PGV and response spectra as well as PGA is proposed. The bedrock motion can be estimated using the attenuation formula developed from the observed record at the bedrock or estimated bedrock motion from surface observation records. The earthquake motion is affected by source characteristics and path effects as well as the magnitude and distance. Therefore, the result derived by the attenuation formula is affected by the characteristics of the data that were used in its development. Hence, a formula derived from the observed data in and around the study area is preferable. To define the original attenuation formula for the study area, many data covering a wide range of magnitude and epicentral distance were used from stations where the ground condition is well known. The existing attenuation formula can be used within the coverage of magnitude and distance of the data in its development.

### [Method-2] Empirical Green's Function Method

Observed earthquake motion can be modelled by the convolution of slip distribution in time and space domains at the fault surface and the response of materials in propagation pass for unit slip (Green's function). The idea of the empirical Green's function is to use an observed small event for Green's function instead of a theoretical one to calculate a large event. The advantage of the empirical Green's function is that a small event contains propagation-path effects and local site effects if the propagation-path of the small event is the same as that of a large event. Many researchers have studied the empirical Green's function method. The method by Irikura (1986) is one of the most famous and practically applied. To estimate the bedrock motion, observed records at the rock outcrop site or de-convoluted waveforms from the surface observations will be used. If the waveforms of the adequate small event that occurred in the expected source zone of the target scenario earthquake source zone are available, this method is suitable to simulate a large event at the observation site.

**[Method-3] Stochastic Green's Function Method**

The largest shortcoming of the empirical Green's function method is that it needs the waveforms of the small event that occurred in the source zone of the target earthquake. On the other hand, the stochastic Green's function method uses stochastically derived small events from the theoretical source model of dynamic features. They include asperity distribution with stress drop and source spectra. This method was advocated by Dr. Irikura (for example, Kamae et al. (1991)) and adopted in the seismic microzoning project within Japan by the Cabinet Office of Japan.

The necessary data in each method, which is outlined above, and the grade of necessity is shown in Table 3-2.

Table 3-2 Necessary Data and their Availability - Bedrock Motion -

| Data | Necessity in each Method | | | Availability in Algiers | |
|---|---|---|---|---|---|
| | Method-1 | Method-2 | Method-3 | | |
| | Empirical Attenuation Function (PGA,PGV,Sa) | Empirical Green's Function Method | Stochastic Green's Function Method | Quality | Quantity |
| Earthquake Hazard Record | △ | △ | △ | △ | △ |
| Historical Earthquake Catalogue | ◎ | ◎ | ◎ | ○ | ○ |
| Intensity Distribution of Historical Earthquakes | △ | △ | △ | △ | △ |
| Instrumental Earthquake Catalogue | △ | △ | △ | ○ | ◎ |
| Micro Earthquake Catalogue | △ | △ | △ | × | × |
| Strong Motion Records (List,Wave Form) | | ◎ | | ○ | △ |
| Ground Condition of the Strong Motion Observatory | | ○ | | × | × |
| Location/Length of Active Faults | ◎ | ◎ | ◎ | ○ | ◎ |
| Recurrence Interval/Last Event of Active Faults | △ | △ | △ | × | × |
| Crustal Movement Record (GPS) | △ | △ | △ | ◎ | ○ |
| Attenuation Function for Algeria | ○ | | | △ | △ |
| Fault Length - Magnitude Relation in Algeria | ○ | | | × | × |
| Static Source Model (L,W,dip,Vr) | ○ | ◎ | | △ | △ |
| Dynamic Source Model (Asperity, Stress Drop) | | | ◎ | × | × |

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△ : Desirable

◎ : OK
○ : Almost OK, but additional information is recommended
△ : Exist, but not enough
× : Not exist or scattered

**[In Algiers]**

The availability of the data is shown in Table 3-2.

In Algiers, [Method-1] is adopted based on the data availability. The following three attenuation formulas were selected after considering Algerian records:

- Laouami et al. (2005)
- Ambraseys et al. (2005)
- Berge-Thierry et al. (2003)

The applicability of these three attenuation relationships to the Algiers area was evaluated through comparison with the strong motion records observed in the 2003 Boumerdes earthquake. The PGA values of horizontal components are plotted in Figure 3-3 according to the ground condition. The lines in the upper graphs in Figure 3-3 are the formulae by Laoumi et al. (2005) and Berge-Thierry et al. (2003), while that of Ambraseys et al. (2005) is in the lower graphs. The formula by Ambraseys et al. (2005) provides better estimates than the other two; therefore we have decided to use the method of Ambraseys et al. (2005) for bedrock motion calculation in Algiers.

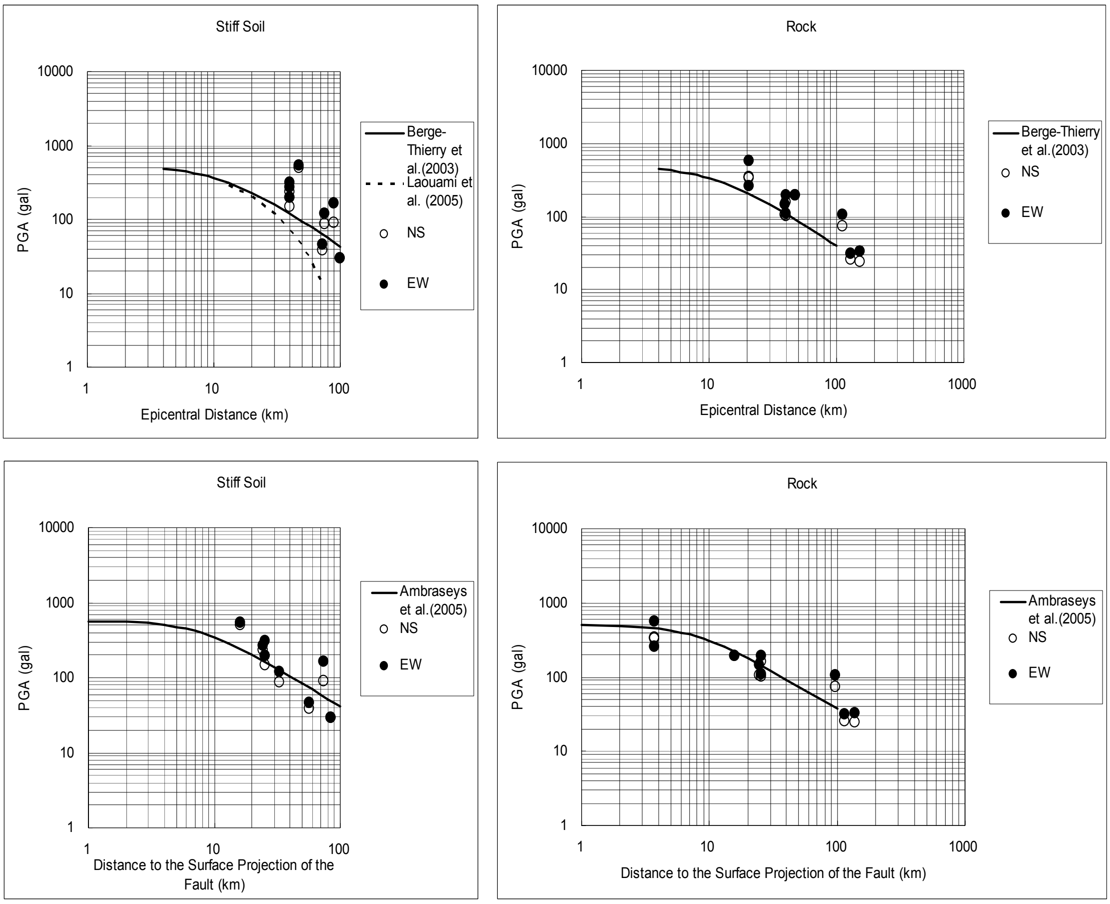


Figure 3-3 Comparison of observed PGA with attenuation formulas

## 3-3 Surface Amplification

### [Method-1] Based on Past Hazards

The most direct approach to evaluate surface amplification is to estimate the distribution of damage induced during past destructive earthquakes from the available data. The past earthquake damage shows that the earthquake motion in an area with thick sediment soil is larger than that in the rock area nearby, and that the value is not uniform even in a small area. The local site effects due to surface soil will be comparable during subsequent events if the ground condition is not drastically changed by large-scale development after the previous earthquake. The amplification due to surface soil can be evaluated by comparing the observed surface records or estimated intensity / PGA from damage due to past earthquakes based on observed records at the baserock or estimated values using the attenuation formula. In order to apply this approach for prediction, the past earthquakes should have been large enough to cover the target area.

### [Method-2] Estimation from Geology / Geomorphology

The surface geology / geomorphology have often been used to interpret the observed incremental intensity at each site. As far as the relationship between the geology and the surface amplification is concerned, Boucherdt & Gibbs (1976) and Midorikawa (1987) are well known. The relationship between geomorphological classification and amplification has been well studied in Japan and the proposed relationship by Matsuoka & Midorikawa (1994) or Fujimoto & Midorikawa (2003) are used. These relationships are analyzed based on the comparison of baserock motion and surface motion at the site where the ground condition is closely studied and a numerical simulation has been based on the ground model. However, these relationships are not unique world-wide and may differ from site to site.

### [Method-3] Estimation from Vs (S-wave velocity) of the Surface Layer

This method uses the S-wave velocity of the surface layer instead of the geology or geomorphology to evaluate surface soil amplification. Earthquake observations and numerical analysis have revealed that the average S-wave velocity from the surface to some depth is highly correlated with the surface soil amplification. Joyner & Fumal (1984), Borcherdt et al. (1991) and Midorikawa et al. (1994) have proposed the relationships between average S-wave velocity and amplification factor. These relationships are not unique world-wide and may differ site to site.

### [Method-4] Response Analysis (1D)

This method makes use of multiple reflection models for the propagation of the S-waves in a one-dimensional column. The ground is modeled as a series of horizontal layers and amplification in the frequency domain is calculated. Linear, equivalent linear or non-linear analysis can be used. The soil column from the bedrock to the surface, the S-wave velocity of each layer, density and the non-linear properties of the soil are necessary.

**[Method-5] Response Analysis (2D / 3D)**

The ground is modeled as 2D or 3D grid models. The finite element or finite difference methods are used for the numerical simulation of the wave propagation in the soil layer. A large amount of data is necessary compared to 1D analysis.

The necessary data for each method, which is outlined above, and the grade of necessity is shown in Table 3-3.

Table 3-3 Necessary Data and their Availability - Surface Amplification -

| Data | Necessity in each Method | | | | | Availability in Algiers | |
|---|---|---|---|---|---|---|---|
| | Method-1 | Method-2 | Method-3 | Method-4 | Method-5 | Quality | Quantity |
| | Based on the Past Hazard | Estimate from Geology/Geomorphology | Estimate from Vs of Upper Layer | Response Analysis (1D) | Response Analysis (2D/3D) | | |
| Intensity/PGA Distribution of Past Large Earthquakes | ◎ | ○ | ○ | △ | △ | △ | △ |
| | | | | | | | |
| Topographical Map | △ | ○ | ○ | ○ | ○ | ◎ | ○ |
| Geological Map | △ | ◎ | ○ | ○ | ○ | △ | ○ |
| Geomorphological Map | △ | ◎ | ○ | ○ | ○ | × | × |
| Subsurface Soil Map | △ | ◎ | ○ | ○ | ○ | △ | △ |
| Amplificatin Factor - Geology/Geomorphology/Soil Relation | | ○ | | | | × | × |
| | | | | | | | |
| Boring Log | | | ◎ | ◎ | ◎ | △ | ○ |
| Velocity Logging result | | | ◎ | ◎ | ◎ | × | × |
| Amplification Factor - Average Vs of Surface Layer Relation | | | ○ | | | × | × |
| | | | | | | | |
| Static Soil Property (N-value, Density etc.) | | | | ○ | ○ | △ | △ |
| Dynamic Soil Property | | | | ○ | ○ | × | × |
| | | | | | | | |
| Aerial Photograph | △ | △ | △ | △ | △ | ◎ | ◎ |
| | | | | | | | |
| Depth of Base Rock | | | ○ | ◎ | | △ | ○ |
| 2D/3D Structureof Base Rock | | | | | ◎ | × | × |
| Strong Motion Records (for response analysis) | | | | ○ | ○ | ○ | △ |
| | | | | | | | |

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× : Not exist or scattered

**[In Algiers]**

The data that was available before the Algiers seismic microzoning study began is shown in Table 3-3. This table shows that it was difficult to evaluate the surface amplification from existing data. Therefore, many boring surveys and PS loggings were planned and executed in the study to adopt the [Method-4] in Algiers (Figure 3-4).

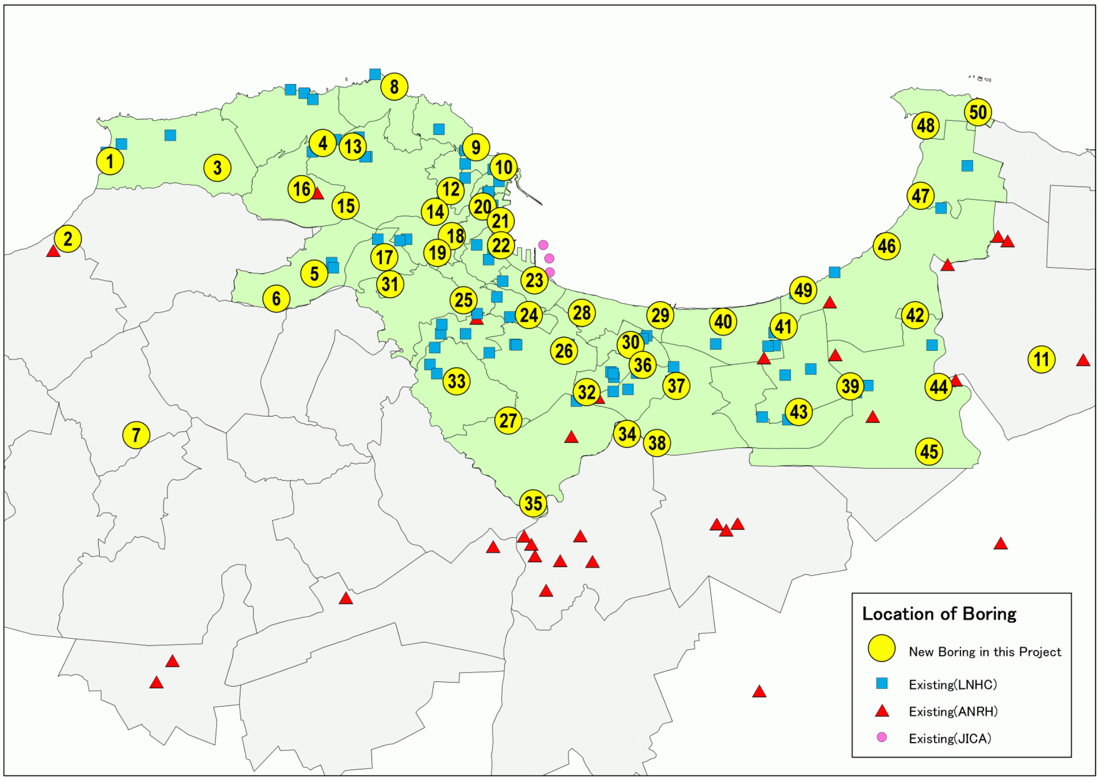



Figure 3-4 Boring Locations

To adopt the 1D response analysis, the following conditions were studied before beginning the numerical simulation.

1) Definition of engineering seismic bedrock and seismic motion on it

In Algiers, based on the existing geological map, existing borings, literature and compiled boring logs and PS loggings, fresh Plaisancian blue marl (p1-f) with Vs of 630 m/sec and fresh schist (mi-f) with Vs of 1030 m/sec were used for engineering seismic bedrock (Figure 3-5).

As the estimated bedrock motion is not defined assuming the adopted engineering seismic bedrock, the bedrock motion was converted to the value of engineering seismic bedrock using the empirical relation of Vs and amplification, e.g. by Midorikawa et al. (1994).

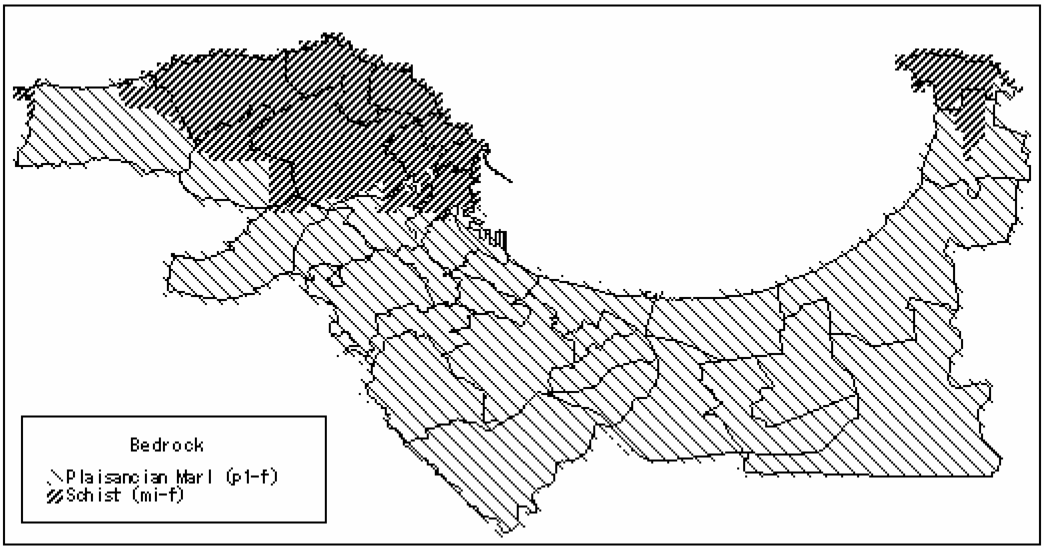



Figure 3-5 Distribution of two kinds of Engineering Seismic Bedrock

2) Non-linear property of soils

As there is no dynamic soil laboratory test to evaluate the non-linear dynamic property of soil in Algeria, the existing non-linear dynamic property of soil used in a seismic microzoning study of Tokyo Metropolitan Area, Japan, was applied after considering the similarities of the soil, S-wave velocity and N-value. Figure 3-6 shows the adopted non-linear properties of the soils.

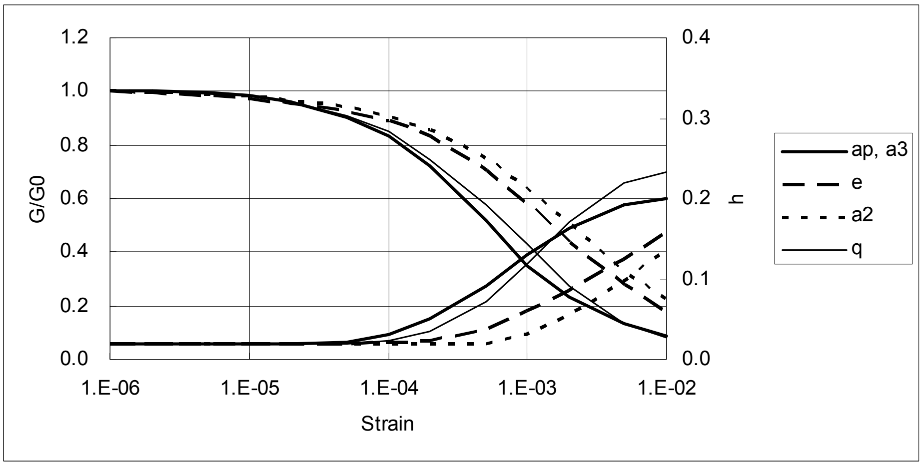



Figure 3-6 Non-linear Properties of Soils

3) Input seismic waves

In Algiers, the estimated bedrock waves during the 2003 Boumerdes earthquake were used as input seismic waves based on the following considerations. The magnitude of the Boumerdes earthquake, Mw=6.9, is comparable to the scenario earthquakes and the distance from the source area to the study area also does not differ much. Consequently, the frequency contents of the observed seismic waves in Algiers during the Boumerdes earthquake are suitable for the input motion of the response analysis.

Two horizontal components of two stations in Algiers were used; therefore, four wave forms were used in the analysis. The wave forms of the four input waves are shown in Figure 3-7. The averaged value of the four calculated PGA values, which correspond to the four input waves, was used as the final result.

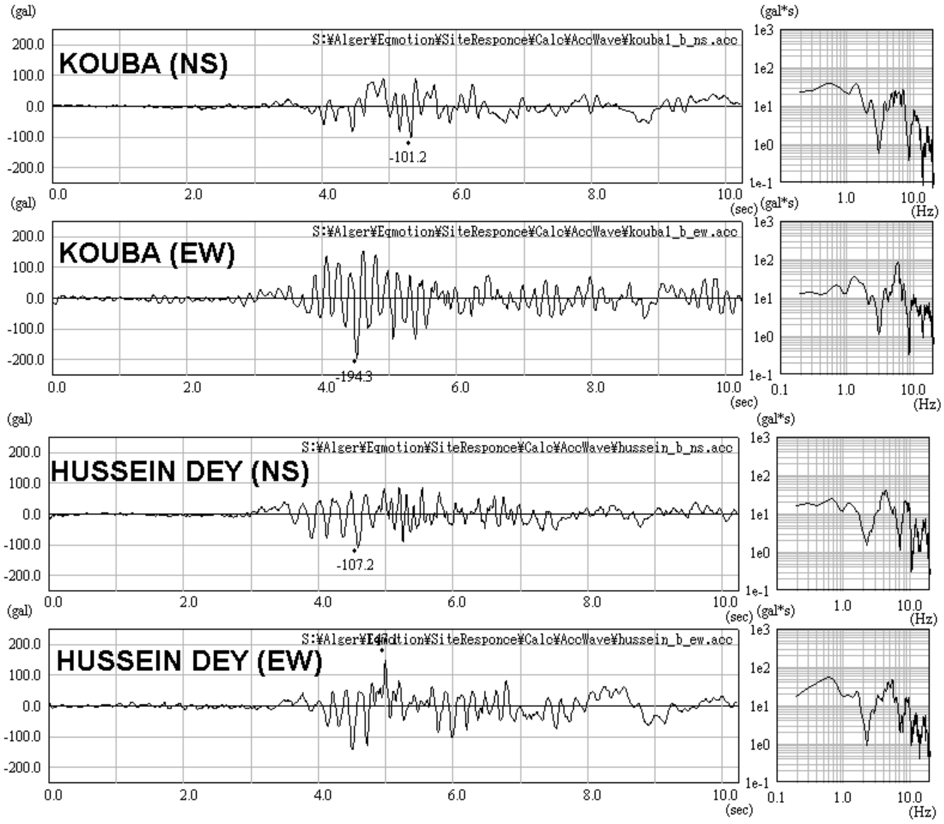



Figure 3-7 Input Waves Used for Response Analysis

## 3-4 Liquefaction Potential

### [Method-1] Based on Magnitude and Distance

Several researchers have analyzed the distribution of liquefaction during past earthquakes and have compared the distance from the epicenter to the farthest liquefied site with the earthquake magnitude. The results were compiled into the “epicentral distance to farthest liquefied site chart”. The charts by several researchers differ in terms of reflecting the difference in the sites and databases, though they are useful in delineating an approximate area to be studied for liquefaction potential. If the ground water level is low, the liquefaction potential can be judged as low.

Liquefaction is known to occur repeatedly at the same site. Therefore, the overlay of epicentral distance to the farthest liquefied site chart, ground water level map and liquefaction experience during past earthquakes is effective in estimating the liquefaction that would be produced by scenario earthquakes.

**[Method-2] Estimate by Geomorphological Criteria**

It is known that liquefaction has a higher correlation with the geomorphological unit, which reflects the sedimentary process of soils. For example, liquefaction potential of natural levees and abandoned river channels is high. If the correlation between the geomorphological unit and the liquefaction experience can be analyzed in the study area and the criteria for liquefaction estimation made, precise estimation will be possible. Interpretation of aerial photographs will also be considered.

**[Method-3] Numerical Analysis using a Geotechnical Ground Model**

The liquefaction resistance of soils susceptible to liquefaction is estimated and compared with the shear stress in the soil during the earthquake. If the shear stress is larger than the liquefaction resistance, the soil deposit is judged to liquefy. The liquefaction resistance is usually estimated from the SPT / CPT value. The methods by Seed & Idriss (1971) and the Japan Road Association (1980, 1991) are well known.

The necessary data in each method, which is outlined above, and the grade of necessity is shown in Table 3-4.

Table 3-4 Necessary Data and their Availability - Liquefaction Potential -

| Data | Necessity in each Method | | | Availability in Algiers | |
|---|---|---|---|---|---|
| | Method-1 | Method-2 | Method-3 | | |
| | Based on the Magnitude and Distance | Estimate by the Geomorphological Criteria | Numerical Analysis using Geotechnical Ground Model | Quality | Quantity |
| Maximum Liquefaction Distance - Magnitude Relation Chart | ○ | ○ | | × | × |
| Liquefaction History Map | ○ | ◎ | ○ | △ | △ |
| Topographical Map | | ○ | | ◎ | ○ |
| Geological Map | | ○ | | △ | ○ |
| Geomorphological Map | | ◎ | | × | × |
| Subsurface Soil Map | | ◎ | | △ | △ |
| Boring Log (with N-Value or CPT) | | | ◎ | △ | △ |
| Soil Test result (Density, Grain Contents, etc.) | | | ◎ | △ | △ |
| Aerial Photograph | | ○ | △ | ◎ | ◎ |
| Ground Water Level Map | ○ | ○ | ◎ | △ | △ |

◎ : The data in Study Area is indispensable
○ : Necessary but can be estimated or substituted by the data in other area
△ : Desirable

◎ : OK
○ : Almost OK, but additional information is recommended
△ : Exist, but not enough
× : Not exist or scattered

**[In Algiers]**

The data that was available before the Algiers seismic microzoning study began is shown in Table 3-4. This table shows that it was difficult to evaluate the liquefaction potential from existing data. Therefore, many boring surveys and PS loggings were planned and executed in the study to adopt the [Method-3] in Algiers.

As the ground water level is very important data to evaluate the liquefaction potential, it was measured at every boring point during the soil investigation. The observed groundwater levels are analysed along with the existing data (ex. Figure 3-8). The seasonal change of ground water can be estimated if enough data is available.

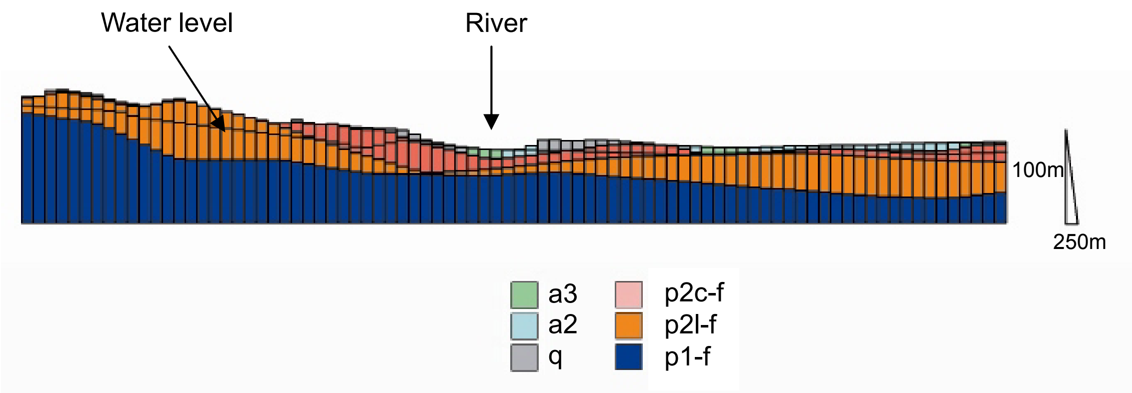



Figure 3-8 Example of water level section near OUED EL HARRACH

## 3-5 Slope Stability

### [Method-1] Based on the Magnitude and Distance

Many studies have been conducted concerning the relationship between maximum distance from epicenter to the farthest slope failure and the earthquake magnitude. The results were compiled into the "epicentral distance to farthest slope failure chart". The charts by several researchers are not the same, reflecting the difference in the sites and databases, although they are useful to delineate the approximate area that needs slope failure analysis.

Slope failure is known to occur repeatedly at the same site. Therefore, the overlay of the epicentral distance to the farthest slope failure chart and slope failure experience based on past earthquakes is effective to estimate slope failure due to the scenario earthquakes.

### [Method-2] Estimate the Stability by Scoring Several Items

The slope height, slope angle, soil type, rainfall, etc. are identified as topographical and geological factors that may affect slope failure. If the relationship between these factors and slope failure experience is known in the study area, high slope failure potential areas can be delineated on a geological map using this relationship.

### [Method-3] Numerical Analysis using Cohesion and Internal Friction Angle

This method involves detailed individual slope stability analysis at each slope. Site-specific individual parameters, i.e. slope angle, cohesion of soil, internal friction angle, etc. are necessary. The existing methods are calibrated and adopted over a limited area; therefore, calibration based on experience of slope failure in the study area is necessary.

The necessary data in each method, which is outlined above, and the grade of necessity is shown in Table 3-5.

Table 3-5 Necessary Data and their Availability - Slope Stability -

| Data | Necessity in each Method | | | Availability in Algiers | |
|---|---|---|---|---|---|
| | Method-1 | Method-2 | Method-3 | | |
| | Based on the Magnitude and Distance | Estimate the Stability with Scoring of Several Items | Numerical Analysis with Cohesion and Internal Friction Angle | Quality | Quantity |
| Maximum Slope Failure Distance - Magnitude Relation Chart | ○ | ○ | | × | × |
| | | | | | |
| Slope Failure History Map | ○ | ○ | ◎ | × | × |
| | | | | | |
| Topographical Map | | ◎ | △ | ◎ | ○ |
| Geological Map | | ◎ | △ | △ | ○ |
| Rainfall Map | | ○ | △ | × | × |
| Hydrological Map | | △ | △ | × | × |
| | | | | | |
| Digital Elevation Model (DEM) | | △ | ◎ | △ | ◎ |
| Soil Test result (Cohesion, Internal Friction angle) | | | ◎ | ○ | ○ |
| | | | | | |

◎ : The data in Study Area is indispensable
○ : Necessary but can be estimated or substituted by the data in other area
△ : Desirable

◎ : OK
○ : Almost OK, but additional information is recommended
△ : Exist, but not enough
× : Not exist or scattered

**[In Algiers]**

The availability of the data is shown in Table 3-5.

In Algiers, [Method-3] is adopted based on the data availability. Two types of slope are found in Algiers, i.e. one is a steep slope composed of schist and calcareous sandstone and the second is a gentle slope composed of other soils. The expected hazard of a steep slope is collapse and a gentle slope is expected to produce a land slide, therefore, two numerical analysis methods were used respectively. The Wilson's method (Wilson et al. (1979)) (Figure 3-9) was used for steep slopes and Ansal and Siyahi's method (Ansal and Siyahi (1993)) (Figure 3-10) was used for gentle slopes.

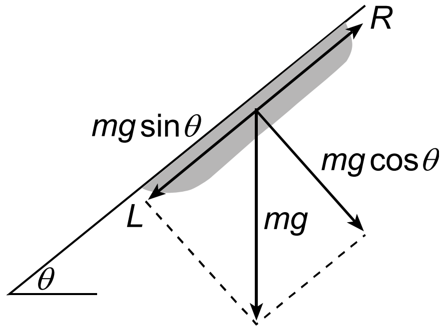



Figure 3-9 Model of potential landslide mass (Tanaka, 1982)

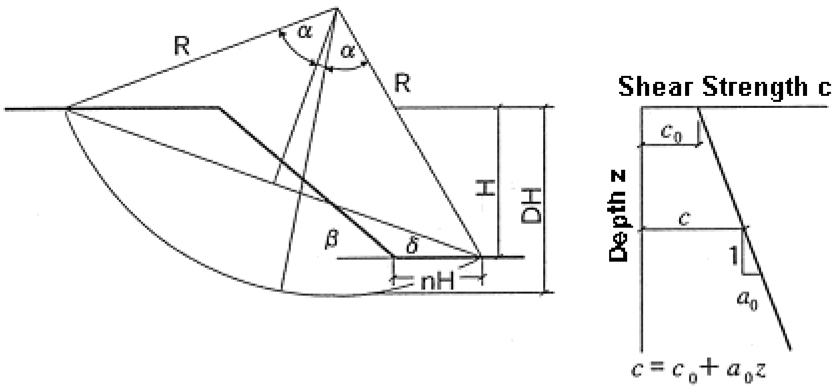



Figure 3-10 A typical section of slope (Koppula, 1984)

In Algiers, the slope failure potential of each grid was evaluated instead of individual slope stability because individual slopes have not been identified in Algiers. The slope angle was calculated in 5m intervals based on the DEM data in Algiers, however, if individual slopes are identified, they should be evaluated by site survey.

**[References]**


Ansal, A and B.G. Siyahi, 1994, Microzonation for landslides during earthquakes, 2nd Int. Conf. on Earthquake Resistant Construction & Design, Savidis(ed), Balkema, 151-157.

Ambraseys N.N., Douglas J., Sarma S.K. and Smit P.M., 2005, Equations for the Estimation of Strong Ground Motions from Shallow Crustal Earthquakes Using Data from Europe and the Middle East: Horizontal Peak Ground Acceleration and Spectral Acceleration. Bulletin of Earthquake Engineering, 3, pp.1-53.

Berge-Thierry, C., Cotton, F., Scotti, O., Griot-Pommera, D.-A., & Fukushima, Y., 2003, New empirical response spectral attenuation laws for moderate European earthquakes. Journal of Earthquake Engineering, 7(2), 193–222.

Borcherdt, R. D., and J. F. Gibbs, 1976, Effects of Local Geological Conditions in the San Francisco Bay Region on Ground Motions and Intensities of the 1906 Earthquake, Bull. Seism. Soc. Am., Vol. 66, 467-500.

Borcherdt, R. D., C. M. Wentworth, A. Janssen, T. Fumal, and J. Gibbs, 1991, Methodology for Predictive GIS Mapping of Special Study Zones for Strong Ground Shaking in the San Francisco Bay Region, Proc. 4th Intern'l. Conf. on Seismic Zonation, Vol.3, pp.545-552.

Fujimoto, K. and S. Midorikawa, 2003, Average Shear-Wave Velocity Mapping throughout Japan using the Digital National Land Information, Journal of JAEE, Vol. 3, No. 3, 13-27.

Irikura, K., 1986, Prediction of Strong Acceleration Motion Using Empirical Green's Function, Proc. 7th Japan Earthq. Symp., 151-156.

Japan Road Association, 1980, 1991, Specifications for Highway Bridges, Part V Earthquake Resistant Design.

Joyner, W. B. and T. Fumal, 1984, Use of Measured Shear-wave Velocity for Predicting Geological Site Effects on Strong Motion, Proc. 8th World Conf. on Earthq. Eng., Vol.2, 777-783.

Kamae, K., K. Irikura and Y. Fukuchi, 1991, Strong Motion Prediction during Large Earthquake based on the Scaling Law of the Earthquake, J. Struct. Constr. Eng., AIJ, No. 430, 1-9.

Koppula, S.D., 1984, Pseudo-static analysis of clay slopes to earthquakes, Geotechnique, 34, 71-79.

Laouami N., Slimani A., Bouhadad Y., and Nour A., 2005, Strong ground motions analysis of the 21st May 2003 Boumerdes (Algeria) earthquake (Mw=6.8) and elaboration of the Algerian attenuation law. Submitted for publication in Soil Dynamics and Earthquake Engineering Journal.

Matsuoka, M. and S. Midorikawa, 1994, GIS-Based Seismic Hazard Mapping using the National Land Information, Proc. 9th Japan Earthq. Symp.,E331-E336.

Midorikawa, S., 1987, Prediction of Isoseismal Map in the Kanto Plain due to Hypothetical Earthquake, Journal of Structure Engineering, Vol.33B, 43-48.

Midorikawa, S, M. Matsuoka and K. Sakugawa, 1994, Site Effects on Strong-motion Records Observed during the 1987 Chiba-ken-toho-oki, Japan Earthquake, Proc. 9 th Japan Earthq. Symp., Vol.3, 85-90.

Seed, H. B. and I. M. Idriss, 1971, Simplified Procedure for Evaluating Soil Liquefaction Potential, J. SMFD, ASCE, Vo. 97, No. 9, 1249-1273.

Tanaka, K., 1982, Seismic Slope Stability Map (Present situation and several mooted points), Journal of Japan Landslide Society, 19-2, 12-19. (in Japanese)

Wilson, R.C., G.F. Wieczorek, E.L. Horp, 1979, Development of Critteria for regional Mapping of Seismic Slope Stability, Abstract, 1979 Annual Meeting of the Geological Society of America.

# 4. Damage Function

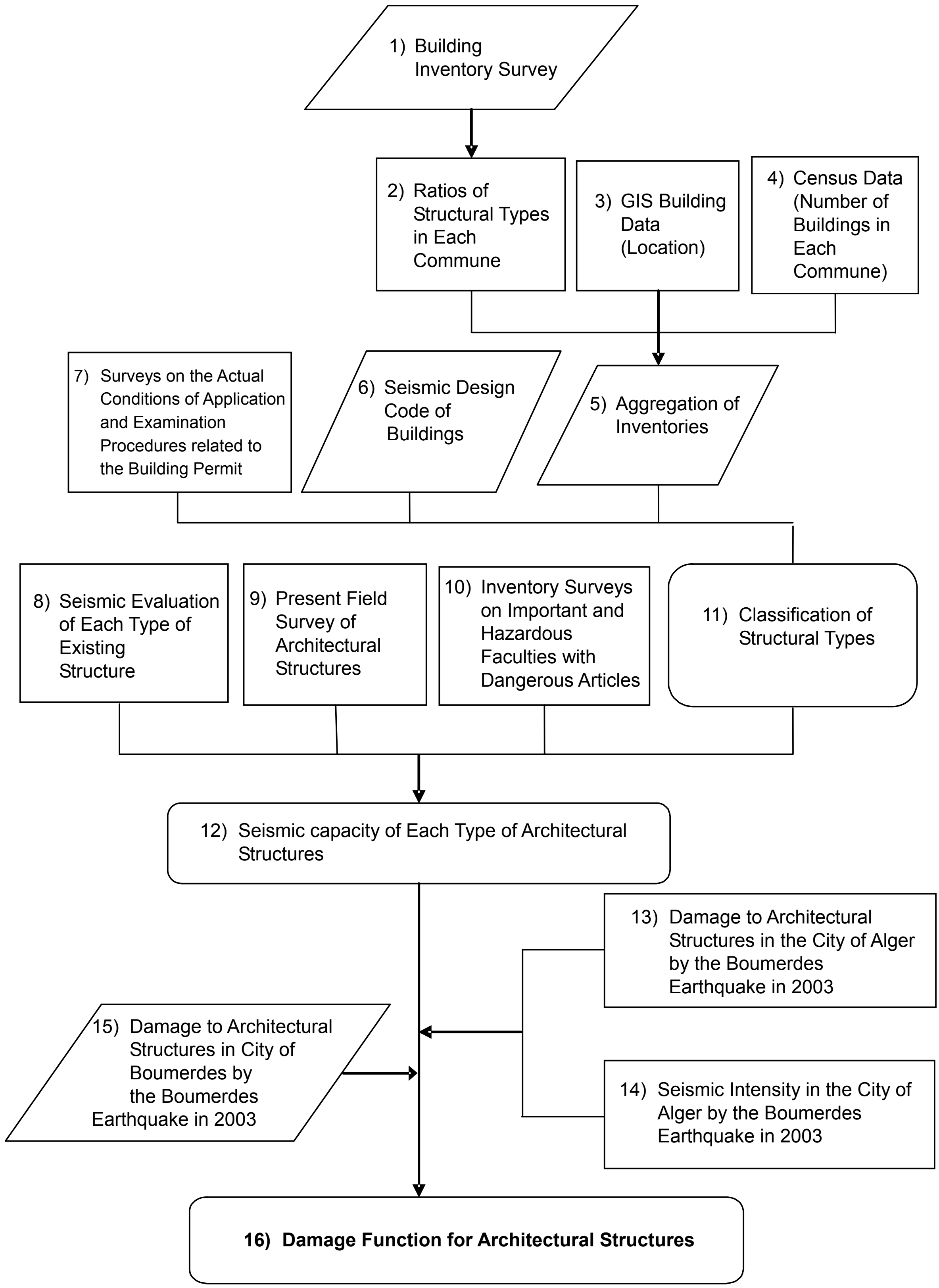
## 4-1 Building

The purpose of estimating seismic damage to architectural structures is to alleviate future suffering by estimating damage expected to be caused by great earthquakes in the region subject to the survey on the basis of the current vulnerability of the structures, drawing up short-, mid-, and long-term disaster prevention plans based on those estimates and implementing varied measures in order of priority.

More than 80% of all human injuries suffered in past earthquakes were due to entire or partial collapse of buildings.

Expected damage to architectural structures in this seismic microzoning survey was calculated quantitatively in reference to the scale of motions of an earthquake under a certain scenario, damage function of various structures, and the number of various structures. The most crucial aspect in this process is to determine the damage function of various architectural structures in the present situation, that is, to understand the statistically computed average seismic capacity of various structures. Seismic capacity of various structures differ largely depending on the materials of the structures, construction and design dates, the number of stories, construction quality, and type and condition of the bearing soil. Existing data regarding these items was gathered and made use of as much as possible, though it was still found to be necessary to conduct surveys on after examining the scope, nature, and accuracy of data available. The precision of the predictions of seismic capacity relies on the scope, nature, accuracy and other factors of the existing and newly obtained data.

Figure 4-1 shows a flow chart of the process of setting the damage function of architectural structures. The items in the figure are interrelated, but their cause/effect relationship is not constant. The choice and methods of use of resources related to the setting of the damage function are described below in order of the items.




Source: JICA Study Team

Figure 4-1 Flowchart of Damage Function Determination for Architectural Structures

### 1) Building Inventory Survey

A building inventory survey is generally conducted to determine the general classification and seismic capacity of structures. It requires the statistical quantities of each item within the areas (communes, zoning, etc.) to be expressed in a "damage map." The principal items of the building inventory survey, their relative importance and availability are shown in Table 4-1.

In general, it is rare that all the various kinds of data concerning seismic capacity are available, so that it is necessary to carry out surveys to obtain the missing information for microzoning.

For this Alger project, inventory surveys of architectural structures and important facilities and facilities with dangerous articles have been carried out. For details of the surveys, see Chapter 4 of the main report.

Table 4-1 Items of Building Inventory Survey

| Data Items | Relative Importance | Availability in the City of Alger | |
|---|---|---|---|
| | | Quality | Quantity |
| Location of building | ○○○ | ◎ | ◎ |
| Use of building (simple use, complex) | ○○○ | ○ | ○ |
| Zoning (legally specified, urban planning) | ○○ | × | × |
| Ownership（public, private） | ○ | ○ | ○ |
| Structural types (specific structural types, such as reinforced concrete, masonry, steel, wood, etc) | ○○○ | ○ | ○ |
| Construction and design date (revision of seismic design codes, building permit, change in construction quality, etc.) | ○○○ | ○ | ○ |
| Number of stories（above ground, basement) | ○○○ | ◎ | ◎ |
| Engineered or non-engineered structure | ○○ | ○ | ○ |
| Status of seismic retrofit | ○○○ | △ | △ |
| Planar and elevational imbalance | ○○ | ○ | ○ |
| Status of extension or reconstruction | ○ | △ | △ |
| Types and condition of soil | ○○○ | △ | △ |
| History of natural disasters（earthquake, fire, flood, etc.） | ○○ | △ | △ |
| Presence or absence of short columns and soft stories | ○○○ | △ | ○ |
| Condition of expansion joints (offset distance of floor, roof, wall) | ○ | △ | ○ |

Relative Importance is classified into the following three degrees according to its importance in determining the damage function.

- ○○○ : Very important item and should be surveyed without fail
- ○○ : Important and shall be obtained through interviews with the owner or building manager if possible
- ○ : general survey item and can be appraised by an investigator if the status is unclear

Availability indicates the quality and quantity of data that could be accessed and evaluated.

- ◎ : No problems（OK）
- ○ : Few problems, but additional information needed to be obtained through interviews.
- △ : There is information, but it is insufficient.
- × : There is no information, or information is contradictory or vague.

### 2) Ratios of the various Structural Types in Each Commune

In the microzoning survey in the city of Alger, communes are counted as individual units for displaying on the map. Once the unit to display on the map has been determined, it is necessary to obtain various data in accordance with the displaying unit. Hence, the ratio of the number of each type of architectural structure is required for each commune. In this Alger project, the results of the building inventory surveys have been adopted and the respective ratios were applied to each individual commune.

### 3) GIS Building Data (Locations)

The Alger project makes use of a 2003 digital map owned by URBANIS to count the number of existing architectural structures. Structures and facilities located on mesh lines or boundaries of communes are considered to be located in the areas where the center of the structures or facilities are placed. On this digital map, information concerning roads is not input as such, but areas where neither structures, lands nor any other premises are displayed are recognized automatically as roads or vacant grounds. On the GIS data map for this survey, road information has been newly input, together with displays of whether individual roads are national or public ones, the width, pavements, etc.

### 4) Census Data (Number of Buildings in Each Commune)

Census data was obtained from a national census that is carried out more or less every ten years and the most reliable survey results. The 1998 census, including the population of males and females, age composition, the number of residential and non-residential houses, the proportion of independent dwellings to the structures as a whole, the number of rooms per house, the spread ratio of housing facilities, and so on, has been used as a basic resource for this survey. Structural types were surveyed in the 1997 census, but not in the latest, 1998, census. The distribution of structural types has been obtained from the results of the building inventory survey.

### 5) Aggregation of Inventories

Ideally, initially obtained inventories should be aggregated based on structural types, and classified in detail as much as possible in terms of construction and design date, the number of stories, use of the structures, etc. The aggregation will be conducted in terms of each range displayed on the map. Calculations were made for each commune in the case of the city of Alger. Minor items in the aggregated values will be compiled so as to narrow down the number of major items to 10 or so (20 at most) and to examine the damage function.

### 6) Seismic Design Code of Buildings

The most important task in the process of estimating damage to structures is to understand the seismic capacity of the existing structures. The numerical values of quake-resistance standards adopted in various periods, and seismic capacity of structures the subject of building permit is available. While the seismic design codes are legally defined minimum levels, most architectural structures in any period have been constructed to just about meet such minimum levels or are slightly above.

A survey to determine the trend in changes in quake-resistance standards since 1981 when Algerian standards were officially established will be useful to categorize construction and design dates of structures and to understand seismic capacity of each category of the time periods. On the other hand, the seismic capacity of structures outside the scope of the seismic design codes or application for construction will be assumed comprehensively in accordance with the level of the codes applied to structures constructed in the same period and the results of interviews concerning the actual construction work of the structures in question.

**7) Surveys on the Actual Conditions of Application and Examination Procedures related to Building Permits**

Even if seismic design codes are established, and application for and examination of building permits are carried out, it is still necessary to examine carefully the actual condition of such application and examination, and what percentage of the structures concerned have been, in fact, subject to the application and examination because these elements vary depending on the period. In many cases, about 10 – 30 percent of all structures were supposed to be subject to such application and examination, of which between 50 – 100 percent were actually subject to the application and examination. In Alger, application and examination has been obligatory for all public structures, but not for private buildings. In 2003, the scope of the obligation was expanded to cover private structures, but their number is very small. It is said that a lot of buildings before 1980 and the private buildings after 1981 were non-engineered with a seismic design. To grasp these past situations becomes important information for classification of the structural types.

**8) Seismic Evaluation of Each Type of Existing Structure**

Seismic evaluation is a laborious task, but offers an opportunity to examine the seismic capacity of existing structures in detail and is also fairly useful in examining the damage function of structures. Although there are only a few countries with standards for seismic evaluation, several methods have been invented and put into practice under the initiatives of the governments, universities and other institutions of earthquake-prone countries.

Seismic evaluation has been implemented for five important structures in this Alger project. Of the five buildings, two are extremely old and built with the masonry construction method, whereas the remaining three were built with reinforced concrete, the latter are a hospital building designed in accordance with seismic design code, RPA83; and an apartment and a school building designed in accordance with RPA 88. In addition, one RC non-engineered apartment complex has been evaluated. The influence of concrete strength, whether it is normal or low, on seismic capacity has been evaluated.

Where RC structures are concerned, evaluation and retrofit design, and inspections for seismic capacity after retrofit work have been made by applying the Japanese standards for seismic evaluation (English version). More than one method has been proposed for retrofit work. Seismic capacity varies depending on the retrofit method adopted, which can be referred to when reflecting on the damage function.

In this Alger project, the structural index, "Is" as defined in the Japanese standards for seismic evaluation, which is applicable to existing buildings in Japan, was made use of to judge seismic capacity. In Japan, when the said standards for seismic evaluation were set up in the 1970s, "Is values" and the degree of damage to hundreds of architectural structures caused by Tokachi-oki Earthquake in 1968 and Miyagi-prefecture-oki Earthquake in 1978 were surveyed. In the meantime, universities and quite a few administrative organizations, paying attention to the fact that structures constructed based on the new code launched in 1981 survived the Kobe Great Earthquake in 1995, calculated the "Is values" of architectural structures that suffered in the earthquake, and published the results of their study on the relationship between "Is values" and the proportion of structures damaged. Seismic index of structure has been calculated based on the said standards and the seismic demand index has been evaluated by taking into account the seismic movement of scenario earthquakes in the seismic evaluation of major RC structures in Alger.

As for masonry buildings, seismic evaluation was made for two structures that were built using cobblestones: the current guest palace (Le Palais) constructed in the 1830s and the current building for the senate (le Senat) constructed before 1912. Referring to FEMA-310, the quake-resistance of these buildings was estimated considering the degree of average shear stress on bearing walls, making use of a static analysis method. Masonry joints used for both these buildings, a crucial determinant for seismic capacity of buildings, were lime-mortar, so that it was expected that the strength might be low. However, due to the lack of resources, such as the results of sampling tests concerning strength, the seismic evaluation was made upon the assumption that the degree of shear stress is 0.056N/mm$^2$.

### 9) Present Field Survey of Architectural Structures

Field survey is a visual inspection survey conducted by structural engineers, in which the actual condition of structures in almost all areas within the study area should, ideally, be inspected and, if possible, it is also useful to conduct interview surveys and observe structures outside of study area. In observing the actual condition of the structures, structural types, structural features, the number of stories, locations of structures, that is, new or old towns (construction date), soil condition, work condition of the structures, roads, empty land, greenery areas, etc. are all subject to comprehensive visual observation. This field survey gives useful information when selecting structural types for the damage function and understanding seismic capacity.

### 10) Inventory Surveys of Important and Hazardous Faculties with Dangerous Articles

Important Facilities are the venues and facilities which will be used as strategic command and reception centers and for rescue and relief efforts when a large earthquake occurs, such as buildings of government and administrative organizations, hospitals, schools (meeting halls and grounds), mosques, churches, sports facilities, parks, and other open spaces, which are equipped with necessary facilities for relief efforts and spacious enough to accommodate people affected by disasters and the homeless. Facilities with dangerous articles are facilities which are likely to cause human suffering, for example, fire, explosion, and leakage of gas or chemicals, when a huge earthquake occurs because of the presence of stored hazardous articles. The locations of both types of facilities are crucial, so it is useful to investigate in advance whether these facilities are appropriately located in consideration of emergency accommodation, first-aid action and medium and long term evacuation centers for people affected by the disasters. On the other hand, maps showing the locations of facilities with hazardous articles will be useful resources to see if an appropriate distance exists between such facilities and private residences, etc. In this project in Alger, since the region to be covered by the survey has been divided into 34 communes, it was impossible to see whether or not the allocation of these facilities was appropriate as shown above.

### 11) Classification of Structural Types

It is important for the damage estimation to classify architectural structures mainly in terms of structural material and type so that the difference in seismic capacity, or in other words, the difference in the damage ratio in the area concerned, can be appropriately estimated.

In this Alger project, structural types were initially divided into 11 groups in accordance with the building inventory survey (conducted for 35 items), but later this was reduced to 8 groups on the grounds that no clear difference in the damage ratio due to the number of stories was observed in the Boumerdes Earthquake and thus the number of stories might have little impact on the ratio. The comparison table of structural types is shown in Table 4-2.

Table 4-2 Classification and Comparison of Structural Types

| Initial Classification for the Building Inventory Survey | | | | Final Classification for the Damage Function for Architectural Structures | |
|---|---|---|---|---|---|
| Structure | | Stories | Type | Structure | Type |
| Masonry | Old Brick Masonry | 1,2 | 1 | Masonry At Casbah | 1 |
| | Simple stone | 1,2 | 2 | Stone & Brick Masonry | 2 |
| | | 3,4,5 | 3 | | |
| | | 6+ | 4 | | |
| | Un-reinforced Brick Masonry | 1,2 | 5 | | |
| | | 3,4,5 | 6 | | |
| | | 6+ | 7 | | |
| | Un-reinforced Stone Masonry with composite floor slab | 1,2 | 8 | | |
| | | 3,4,5 | 9 | | |
| | | 6+ | 10 | | |
| Reinforced Concrete (RC) | Pre-code RC frame | 1,2 | 11 | Pre-code RC frame | 3 |
| | | 3,4,5 | 12 | | |
| | | 6+ | 13 | | |
| | Low-code RC frame | 1,2 | 14 | Low-code RC frame | 4 |
| | | 3,4,5 | 15 | | |
| | | 6+ | 16 | | |
| | Moderate-code RC frame | 1,2 | 17 | Moderate-code RC frame | 5 |
| | | 3,4,5 | 18 | | |
| | | 6+ | 19 | | |
| | High-code RC frame | 1,2 | 20 | High-code RC frame | 6 |
| | | 3,4,5 | 21 | | |
| | | 6+ | 22 | | |
| | RC shear wall | 1,2 | 23 | Shear Wall & Mix. | 7 |
| | | 3,4,5 | 24 | | |
| | | 6+ | 25 | | |
| | RC frame and wall | 1,2 | 26 | | |
| | | 3,4,5 | 27 | | |
| | | 6+ | 28 | | |
| Steel | Steel frame | 1,2 | 29 | Steel | 8 |
| | | 3,4,5 | 30 | | |
| | | 6+ | 31 | | |
| | Steel with bracing | 1,2 | 32 | | |
| | | 3,4,5 | 33 | | |
| | | 6+ | 34 | | |
| Others | Block and others | --- | 35 | --- | --- |

The decisions in narrowing down the classification of structural types were made as follows after examining the seismic design odes for buildings and the situation of building permits which were brought to light as a result of the building inventory survey.

In the region surveyed, RC structures accounted for approximately two-thirds of all structures, while those consisting of RC shear walls along with combined structures of frames and shear walls accounted for 12%: those were thus separately classified. Of structures with RC moment frames, high-rise buildings (6 stories or more) accounted for a mere 5%, and most of these were built before 1999 when the damage ratio had been high. On the other hand, the proportion of RC buildings constructed after the seismic design code, RPA99, was adopted accounted for a mere 2%, but, in order to confirm the effect of the standards when disaster-preventive measures such as reinforcement of seismic capacity are implemented in future, this structural type was classified as one independent group. Thus, RC moment frame structures were classified into 4 types, which are: pre-code (non-engineered), low-code, moderate-code, and high-code, in accordance with the transition of seismic design codes.

Masonry construction buildings, though they currently account for some one third of all structures, will be constructed only rarely in the future. While concrete block structure are used for a small number of masonry construction buildings, stones (30%) and bricks (4%) are used for a considerably larger proportion of this type of structure. As for stone buildings, the number of colonial-style mid- and high-rise buildings (three-stories or higher) is almost the same as that of low-rise independent dwelling houses (two-stories or lower): There is not a clear difference in the damage ratio in Boumerdes Earthquake, so that colonial-style buildings and independent dwelling houses have been classified into one single, stone masonry buildings group. A majority of brick buildings are old, as seen in Kasbah, and the joints are mainly made of clay mortar, and deterioration from age is conspicuous due to the characteristics of the material: the damage rate for this type of structure is the highest, so that this type of structure has been classified as a single, independent group.

Steel structures account for only 1% of all structures, but most are newly built, and the number of steel structures seems likely to increase in the future. Thus, those are also treated as one single, independent group.

### 12) Seismic Capacity of Each Type of Architectural Structure

Understanding the seismic capacity of individual structural types classified in the previous section is the most important task throughout the process of estimating damage to architectural structures. As for the methods of examining seismic capacity of various structural types, there are several methods that academic institutions or governmental agencies have independently adopted other than those officially used in the U.S.A., countries in Europe, and Japan. Any method is applicable so long as it suits the types of architectural structures in the region surveyed and is able to evaluate the seismic performance appropriately.

Major methods to examine seismic capacity of existing buildings are described below:

A) In the U.S.A.,

- ATC sets standard values for individual structural types by applying the MMI scales or PGA to various levels of damage states, and provides references for assessment of seismic performance: ATC-13, 14, and 21.

- FEMA provides two methods: one whereby seismic capacity is estimated based on the story drift, and the other whereby the horizontal shear force is calculated to directly estimate seismic capacity: FEMA-154, 155, 178, 237 and 310.

B) In Japan, in the latter half of the 1970s, the standards of seismic evaluation and retrofit for both existing RC and steel buildings were established. In 2001, seismic evaluation standards for wooden structures, which are the most common in Japan, were established.

- Seismic capacity of RC buildings is evaluated in accordance with the "Is value" described in 8).
- Seismic capacity of steel buildings is evaluated based on the horizontal shear force.
- As for wooden buildings, the detail evaluation method based on the wall quantities is one of the most precise and common methods at the moment, apart from which, the "Calculation of Response and Limit Strength," the "Energy-Based Method," and the "Dynamic Response Analysis" are available for evaluation of seismic capacity.

C) In Europe, EMS has established a statistical standard damage function for individual structural types by applying the EMS scale to various levels of damage states, whereas VULNUS and FAMIVE evaluate seismic performance with collapse multipliers based on the safety criterion of the structures.

In 1977 in Japan, seismic evaluation standards for RC buildings were established and put into use, whereby the seismic index of structure (Is) is computed in accordance with the strength index (C), ductility index (F), shape index (SD) and time index (T) and seismic capacity is thereby evaluated. In 1978, seismic evaluation standards for steel buildings were established, whereby seismic capacity is evaluated based on horizontal shear force of existing architectural structure.

In this Alger project, comparative examinations were made on current seismic design code in Algeria (RPA99/V. 2003) and seismic evaluation standards using the “Is value” as in Japan, and, as a result, the Japanese standards applied for RC buildings have been adopted after adjusting the values of seismic demand index. Accordingly, the damage function was calculated by using the "Is" distribution of individual structural types presumed based on the result of seismic evaluation of RC moment frame structures and the survey on the damage ratio of masonry, RC shear wall, and steel structures. The details will be given in Chapter 6 of the main report.

**13) Damage to Architectural Structures in the City of Alger by the Boumerdes Earthquake in 2003, and 14) Its Seismic Intensity**

The damage ratios were derived from a series of damage functions as discussed in 12), along with the corresponding number of structures and the magnitude of earthquake motion, and the ratio was calculated based on the actual damage caused by the Boumerdes Earthquake in 2003. Using these ratios, calibration was carried out to verify the validity of these damage functions.

Although CGS and CTC have been jointly conducting a survey on damage to structures caused by that earthquake, the detailed results of analysis have not been released yet, so that the calibration was based on the preliminary information obtained concerning some regions affected. The preliminary information included a tabulation of the number of damaged structures by individual structural types, but the survey was limited to a finite geographic area and therefore, the number of structures outside the scope of the survey, in different words the safe houses was uncertain. Accordingly, the damage ratio was computed by taking the total number of structures in the GIS data referred to in 3) as the denominator and the damage grade, "4 + 5", as the numerator. Here it is possible for the number of structures – the numerator for the function to compute the damage rate – to take various numbers: case (1) damage grade "4 + 5"; case (2) damage grade "(3+4) / 2 + 5"; and case (3) damage grade "(3/3) + 4 + 5" and so on. In this survey, case (1) has been adopted.

In the meantime, the seismic intensity of only three regions where preliminary information was obtained averaged EMS7.9~8.2 and the maximum range was very small, and thus the damage curve could not be obtained from only the damage data. The value of the damage function finally adopted is the one from which the surveyed damage ratio at the average seismic scale was deduced from the engineering viewpoint based on the fact that the range of seismic intensities is large and survey points in the communes cannot be identified because the number of buildings surveyed was only 14 – 26% of the total.

### 15) Damage to Architectural Structures in the City of Boumerdes by the Boumerdes Earthquake in 2003

Although the calibration of the damage ratios at the average seismic intensity of EMS8 or so was conducted as shown in 13) and 14) above, it is desirable to verify these results with other seismic scales. Although it is not in the study area, another calibration was carried out at a point where the seismic intensity was EMS9 in reference to damage ratios in the city of Boumerdes within the range of average EMS9 seismic intensity which was near the epicenter of the Boumerdes Earthquake in 2003. This confirmed that the chosen damage curve took more or less acceptable values.

Ideally, calibrations of the damage curve should be made at as many points as possible. However, as a matter of fact, directly usable useful data is limited, so that actual calibrations are, in many cases, conducted with reference to data concerning different, similar cities. In this project, the findings of the survey on damage by the Boumerdes Earthquake by CGS were utilized effectively.

### 16) Damage Function for Architectural Structures

As examined in the procedures from 12) ~15), setting a damage function for architectural structures is one of the most important tasks for damage estimation and involves technical and statistical judgment. A vast range of advanced technologies are required in order to appropriately evaluate the seismic capacity of structures located within the study area and to establish a damage function within the statistically appropriate range. Damage estimations based on careful prior examinations, as in this case, may result in a discrepancy of ±50% - ±100%, if the values obtained are compared with actual values to be observed in a large earthquake in the near future. Disaster prevention plans should be implemented fully recognizing this fact.

## 4-2 Infrastructure

Where methods of estimating seismic damage to the infrastructure are concerned, quantitative estimation methods concerning earthquake damage have been invented by HAZUS and other institutions in Japan and the United States of America, and are actually used in practice. This section gives an account of estimation methods adopted in Japan.

Transportation means are classifiable into three types: land, maritime and air transportation. In line with this, the estimation methods will be presented in accordance with these individual types of transportation means.

### 4-2-1 Land Transportation

In regard to land transportation in urban areas, in order to predict the survival or loss of the function of roads after earthquake, it is effective to evaluate damage to bridges on the grounds that they are crucial for road transportation.

Damage estimations are, in many cases, addressed to individual bridges. The representative estimation methods are two types as shown below:

(1) Katayama's Method (created by Kubo and Katayama)

This method, using the procedure described below (see Figure 4-2), evaluates the probability of the superstructure of bridges falling off their supports.

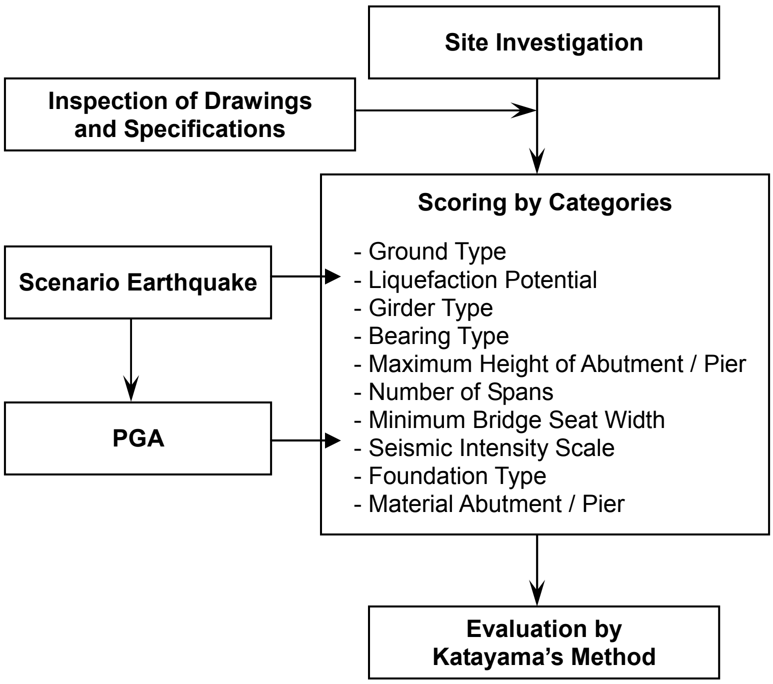



Figure 4-2 Flowchart of Stability Analysis of Bridges

1) Investigations are conducted into the structure of the bridges in question, and the surrounding ground conditions.

2) The earthquake ground motion (seismic intensity) and liquefaction potential at places where the bridges are built are measured.
3) Based on the above results, an appropriate category is selected for each item, and then the weighting value of the selected category is calculated by multiplying the category scores given for all items.
4) The values multiplied by the category scores are considered to be the values to evaluate possible risks of the bridge superstructures falling off their supports.

In Katayama's method, 10 items likely to affect the probability of a girder falling are studied. Each item consists of a number of categories selected without complex calculations. A score chart for bridge stability analysis is shown in Table 4-3. The categories described in Table 4-3 were adjusted from the original in consideration of the conditions in the study area.

Table 4-3 Score Chart for Stability Analysis of Bridges

| Item | Category | Category Score |
|---|---|---|
| Ground Type *1 | Stiff | 0.5 |
| | Medium | 1.0 |
| | Soft | 1.5 |
| | Very Soft | 1.8 |
| Liquefaction Potential *2 | No Liquefaction | 1.0 |
| | Possible Liquefaction | 1.5 |
| | Liquefaction | 2.0 |
| Girder Type | Arch or Rigid Frame | 1.0 |
| | Continuous | 2.0 |
| | Simple | 3.0 |
| Bearing Type *3 | with Specific Device (to prevent the girder from falling-off of the supports) | 0.6 |
| | Bearing (with clear design concept) | 1.0 |
| | two existing bearings that can move in an axial direction | 1.15 |
| Max. Height of Abut./Pier | less than 5 m | 1.0 |
| | 5 to 10 m | 1.35 |
| | more than 10m | 1.7 |
| Number of Spans | 1 span | 1.0 |
| | 2 spans or more | 1.75 |
| Min. Bridge Seat Width | Wide | 0.8 |
| | Narrow | 1.2 |
| Seismic Intensity Scale *4 (JMA) | 5 (less than 5.0) | 1.0 |
| | 5.5 (5.0 to less than 5.5) | 1.7 |
| | 6.0 (5.5 to less than 6.0) | 2.4 |
| | 6.5 (6.0 to less than 6.5) | 3.0 |
| | 7.0 (6.5 and more than 6.5) | 3.5 |
| Foundation Type | Pile Bent | 1.4 |
| | Others | 1.0 |
| Material of Abut./Pier | Plain Concrete or others | 1.4 |
| | Reinforced Concrete | 1.0 |

A category score, shown in Table 4-3, is given to each category as a weighting factor.

The result can then be determined by substituting the data into the following equation:

$$y_i = \prod_{j=1}^{N} \prod_{k=1}^{M_j} X_{jk}^{\delta_i(jk)}$$

where,

$y_i$ : Predictors of damage degrees of i-th bridges
N : Number of all items
$M_j$ : Number of categories of j-th item
$\delta_i(jk)$ : Dummy variable ($\delta_i(jk) = 1$; when the characteristics of the i-th bridge correspond to the category k in the item, $\delta_i(jk) = 0$; otherwise)
$X_{jk}$ : Category score for k-th category of the j-th item
$\prod_{j=1}^{N}$ : Multiplication sign from 1 to N-th value

The threshold value of the predictor*[5] to estimate the damage grade of bridges is based on 30 samples of damaged bridges observed during 3 earthquakes in Japan (1923 Kanto, 1948 Fukui, 1964 Niigata) as shown in Table 4-4.

Table 4-4 Definition of Damage Grade of Bridges

| Class of damage grade | | Threshold value of predictor |
|---|---|---|
| A | - High probability of girders falling off supports<br>- Great deformations generated<br>- Impossible to use for long term and required reconstruction | 30 and more |
| B | - Moderate probability of girders falling off supports<br>- Deformations generated<br>- Impossible to use temporarily and required repairing / rehabilitation | 26 to less than 30 |
| C | - Low probability of girders falling-off supports<br>- Small deformations generated<br>- Possible to basically use after inspection | less than 26 |

**[Commentary]**

This method was proposed making use of the findings of the detailed surveys of various kinds of damage to bridges by large-scale earthquakes occurring in Japan and the theory of quantification. It does not incorporate data concerning recent damage because it was invented 30 years ago, but it is still commonly used due to its high versatility.

The advantages of this estimation method include the facts that it enables the structure of bridges and the characteristics of ground conditions, and in particular, the results of seismic microzoning, to be reflected in the estimation results; and that it also enables the determination of coefficients incorporating the special features of the region in question via making use of data concerning damage to the region so as to adjust the weighting values, i.e., coefficients, and the threshold values.

For this study, the following coefficients have been adjusted through a verification method using the records of damage caused by the Boumerdes Earthquake in collaboration with the JST and its counterpart.

*1: Ground Type

| Original | Modified Category |
|---|---|
| Stiff | Stiff / Hard: Slightly / No Weathered Rock |
| Medium | Medium: Weathered / Moderately Weathered Rock |
| Soft | Soft: Deposited Soil / Diluvium |
| Very Soft | Very Soft: Deposited Soil / Alluvium |

*2: Liquefaction Potential

| Original | Modified Category |
|---|---|
| No Liquefaction | No Liquefaction |
| Possible Liquefaction | Possible Liquefaction: $0 \le P_L < 15$ |
| Liquefaction | Liquefaction: $15 \le P_L$ |

*3: Bearing Type

<table>
<tr><th colspan="2">Original</th><th colspan="3">Modified</th></tr>
<tr><th>Category</th><th>Score</th><th colspan="2">Category</th><th>Score</th></tr>
<tr><td>With Specific Device (to prevent the girder from falling-off of the supports)</td><td>0.6</td><td colspan="2">With Specific Device (to the girder prevent from falling-off of the supports)</td><td>0.6</td></tr>
<tr><td>Bearing (with clear design concept)</td><td>1.0</td><td colspan="2">Bearing (with clear design concept)</td><td>1.0</td></tr>
<tr><td>Two existing bearings that can move in an axial direction</td><td>1.15</td><td colspan="2">Two existing Bearings (that can move in an axial direction)</td><td>1.15</td></tr>
<tr><td></td><td></td><td rowspan="2">Aseismic System</td><td>System for preventing girders from Falling off the supports</td><td>0.6</td></tr>
<tr><td></td><td></td><td>Bearing with Rubber</td><td>0.9</td></tr>
</table>

*4: Seismic Intensity Scale

<table>
<tr><th colspan="2">Original</th><th colspan="2">Modified</th></tr>
<tr><th>Category</th><th>Score</th><th>Category</th><th>Score</th></tr>
<tr><td>5 (less than 5.0)</td><td>1.0</td><td>MSK < 7.885 (JMA: less than 5.0)</td><td>1.0</td></tr>
<tr><td>5.5 (5.0 to less than 5.5)</td><td>1.7</td><td>7.885 ≤ MSK < 8.680 (JMA: 5.0 to less than 5.5)</td><td>2.1</td></tr>
<tr><td>6.0 (5.5 to less than 6.0)</td><td>2.4</td><td>8.680 ≤ MSK < 9.475 (JMA: 5.5 to less than 6.0)</td><td>2.4</td></tr>
<tr><td>6.5 (6.0 to less than 6.5)</td><td>3.0</td><td>9.475 ≤ MSK < 10.270 (JMA: 6.0 to less than 6.5)</td><td>3.0</td></tr>
<tr><td>7.0 (6.5 and more than 6.5)</td><td>3.5</td><td>10.270 ≤ MSK (JMA: 6.5 and more than 6.5)</td><td>3.5</td></tr>
</table>

*5: Threshold Value

| Class of Damage Grade | Original | Modified Value |
|---|---|---|
| A | 30 and more | 30 and more |
| B | 26 to less than 30 | 22 to less than 30 |
| C | less than 26 | less than 22 |

(2) The Japan Road Association Method

This is a method whereby the seismic resistance of bridges is evaluated in accordance with an evaluation value calculated in accordance with the characteristics of the structure of the bridges, the year when the construction codes and standards applied were published and the ground conditions, all of which are determined in reference to the bridge specification sheets (see Table 4-5).

As for the bridge inspection sheets, organizations in charge of them carry out inspections of bridges on a regular basis and compile the results as a database.

Table 4-5 Items included in the Bridge Inventory Survey

| Main Item | Sub Item |
|---|---|
| Superstructure | (1) Year that code / standard was first applied<br>(2) Design/construction of superstructure<br>(3) Material of superstructure<br>(4) System to prevent girders falling off of the supports |
| Displacement at substructure | (5) Type of substructure<br>(6) Height of bridge<br>(7) Ground type<br>(8) Liquefaction potential |
| Strength at termination point of main reinforcement | (9) Ratio of span share<br>(10) Crack by flexural tension at termination point<br>(11) Safety factor at base and termination point<br>(12) Safety factor for yield strength at termination point of main reinforcement<br>(13) Shear unit stress |
| Deformation of substructure | (14) Deformation at bearing<br>(15) Deformation of substructure body<br>(16) Deformation of foundation<br>(17) Deformation of girders, etc.<br>(18) Design/construction of substructure |

**[Commentary]**

Since bridges are constructed in reference to a certain set of guidelines, it is easy to assume, to some extent, the aseismic characteristics of the bridges if the guidelines referred to are specified. At the same time, since bridges are public structures, their forms vary little, unlike public housing whose forms vary extensively due to the construction methods. Therefore, in countries where guidelines on the construction of bridges have been long established, like Japan, referring to such guidelines can serve as an important parameter in predicting the degree of damage. Accordingly, this method is unique in that it makes use of the type of guidelines as a parameter.

The construction date is also used as a crucial parameter in the damage estimation method adopted by HAZUS. Because the idea that the newer bridge has the stronger seismic resistance seems correct, this approach can be considered to be effective.

In Japan, every bridge is subject to regular seismic inspections, and the results of those inspections are recorded in the bridge specification sheets which are kept at the individual administration offices. In such an environment, this method is fairly useful and makes evaluating the quake resistance relatively easily. However, it would take considerable time and expense to execute such an evaluation without those specification sheets as it would be necessary to begin by drawing up such specification sheets.

Bridge specification sheets must be revised on a regular basis, and made use of for the maintenance of bridges. In line with this, it is necessary to obtain a consensus among various circles (public organizations, educational institutes, and the economic circles) when drawing up new bridge specification sheets.

Currently in Algeria, specification sheets in the form adopted in Japan are unavailable; therefore, this damage estimation method is inappropriate.

### 4-2-2 Marine Transportation

Damage estimations concerning maritime transportation address the port facilities because these play the central role in this type of transportation. While a port consists of various facilities, including piers, landing bridges, loading and unloading facilities, warehouses and storage tanks, damage to berthing facilities are in many cases evaluated because these play the most fundamental and crucial role, in particular in the recovery and rehabilitation activities.

The estimation method actually adopted was proposed in accordance with, in principle, records of earthquake disasters in the past. In the following part of this section, two estimation methods are explained: one which makes use of the relationship between the deformation volume and the degree of damage to quays and other parts of ports which are obtained through detailed examinations of damage; and the other which is based on the peak ground acceleration (hereinafter referred to as “PGA”), a general indicator of the relative destructive potential of earthquakes, and the degree of risk of liquefaction.

(1) Uwabe’s Method

Uwabe’s method (1983) begins with estimations of the volumes of structural deformation in reference to the ratio (Fc) of an action seismic coefficient (Ke) to a failure seismic coefficient (Kc). Then, damage to the facilities in question is estimated based on the correlation between the damage grade and the structural deformation volume, which have been obtained as a result of regression analyses on records of port facilities damaged by 17 major earthquakes – from the Kanto earthquake in 1923 to the Miyagi offshore earthquake in 1978.

The procedure of Uwabe’s method is shown below.

1) Action Seismic Coefficient: Ke

$$Ke = \begin{cases} \alpha / g & (\alpha < 200\text{gal}) \\ 1/3 \cdot (\alpha / g)^{1/3} & (\alpha \geq 200\text{ gal}) \end{cases}$$

Ke : Action seismic coefficient that acts on a structure in an earthquake
$\alpha$ : Peak ground acceleration at a port facility (gal)
g : Gravity acceleration (980 gal)

2) Failure Seismic Coefficient: Kc

Kc : Failure seismic coefficient: The minimum seismic coefficient for which the safety factor of stability analysis in conformity with the guideline for port facilities in Japan is less than 1 in an earthquake

3) Fc

Fc = Ke / Kc

4) Estimated Deformation Volume

| Gravity Type | |
|---|---|
| Objective Variable | Regression Equation |
| Maximum amount of swelling (cm) | Dx = -113.8 + 124.4 Fc |
| Crown settlement (cm) | Sp = -50.9 + 57.1 Fc |
| Amount of swelling / Height of structure (%) | R = -12.7 + 14.5 Fc |
| Accumulating displacement (cm) | Da = -127.5 + 148.5 Fc |
| Sheet Pile Type | |
| Objective Variable | Regression Equation |
| Maximum amount of swelling (cm) | Dx = -1.6 + 34.9 Fc |
| Average amount of swelling (cm) | Dm = -15.9 + 9.5 Fc |
| Settlement in apron (cm) | Se = -5.3 + 14.7 Fc |
| Amount of swelling / Height of structure (%) | R = -1.5 + 5.8 Fc |
| Accumulating displacement (cm) | Da = -2.0 + 44.0 Fc |

5) Relationship between Damage Grade and Deformation Volume

[Gravity Type]

| Damage Grade | Maximum amount of swelling (cm) | Average amount of swelling (cm) | Crown settlement (cm) | Settlement in apron (cm) | Angle of overturning (degrees) |
|---|---|---|---|---|---|
| 0 | 0 | 0 | 0 | 0 | 0 |
| I | ≤ 25 | ≤ 25 | ≤ 30 | ≤ 50 | ≤ 5 |
| II | 25 – 70 | ≤ 40 | ≤ 50 | 50 – 80 | 1 – 8 |
| III | 70 – 200 | 40 – 200 | ≤ 100 | 80 – 100 | 2 – 15 |
| IV | 200 ≤ | 200 ≤ | 100 < | 100 ≤ | 15 ≤ |

[Sheet Pile Type]

| Damage Grade | Maximum amount of swelling (cm) | Average amount of swelling (cm) | Crown settlement (cm) | Settlement in apron (cm) | Angle of overturning (degrees) |
|---|---|---|---|---|---|
| 0 | 0 | 0 | 0 | 0 | 0 |
| I | 0 – 30 | ≤ 10 | ≤ 30 | ≤ 20 | ≤ 3 |
| II | 30 – 100 | 10 – 60 | ≤ 40 | ≤ 50 | ≤ 5 |
| III | 100 – 200 | 60 – 120 | ≤ 50 | 50 – 100 | ≤ 10 |
| IV | 200 ≤ | 120 ≤ | 50 < | 100 ≤ | 10 < |

6) Evaluation of Damage Grade

| Damage Grade | Damage Condition | Remarks |
|---|---|---|
| 0 | - No damage | |
| I | - No deformation in main body<br>- Deformation begins to appear in facilities. | |
| II | - Deformation appears in main body | After easy repairs, available to start operation |
| III | - Maintains its shape but incurs heavy damage | Malfunctions |
| IV | - Total collapse | |

**[Commentary]**

In applying this method to the damage estimation, it is necessary to obtain the failure seismic coefficient (that is, the failure seismic coefficient, Kc) where the safety margin of the structure in question drops below 1. This figure is available in Japan so long as the design standards referred to can be obtained, but it is necessary, in general, to conduct a stability computation, for which it is necessary to obtain geological information and properties of the area surrounding individual facilities, as well as the current situations of these facilities. If such information is unavailable from the existing documents and materials, it is necessary to also conduct geotechnical investigations.

In this Study, this method was not adopted because it was found that the necessary information cannot be obtained from the existing documents and materials alone and thus additional investigations would be required, and also because the method does not take the impact of liquefaction into account. It should be noted that, before adopting this method, it would be necessary to thoroughly examine the applicability to Algeria in reference to disaster cases in the past in the country.

(2) A Method Using the Relationship between PGA / Liquefaction and Damage Grade

This method makes use of the relationship (see Table 4-6) between PGA / liquefaction potential and damage grade, which were determined in consideration of records of damage to harbors and ports by the Kobe Earthquake and other earthquakes in the past.

Table 4-6 Damage to Ports by Earthquakes in the Past

| | Ground Acceleration (gal) | | | | |
|---|---|---|---|---|---|
| | 0 to 150 | 150 to 200 | 200 to 300 | 300 to 450 | more than 450 |
| Liquefying soil | 0 | 1 | 2 | 3 | 3 |
| Non liquefying soil | 0 | 0 | 1 | 2 | 3 |

Damage grade 0 : No damage
Damage grade 1 : Slight damage: cracking and deformation of sub-structures
Damage grade 2 : Moderate damage: deformation of main-structures
Damage grade 3 : Heavy damage: serious deformation of main-structures and the loss of function

**[Commentary]**

In this method, the ground acceleration and liquefaction potential derived from the results of a microzoning study are reflected in the damage estimation. Because of its simplicity, it is suitable for the estimation of preliminary vulnerability distribution concerning the port facilities so long as verification can be made with records of earthquakes occurring in the region in question in the past.

This method has been adopted for this Study as a result of verification of the appropriateness in reference to damage caused by the Boumerdes Earthquake.

### 4-2-3 Aerial Transportation

Where aerial transportation is concerned, it is necessary to estimate damage to airport facilities, though there have not been many cases where airports were damaged by earthquakes. Even in the small number of cases where damage was observed, it was minor and thus quantitative estimations of damage to airports are not normally conducted.

In line with this, the Study team has sorted past cases from around the world, found the relationship between PGA and damage grade (see Table 4-7), and proposed a tabulation of damage estimation (see Table 4-8).

Table 4-7 Records of Damage to Airports due to Past Earthquakes

| Earthquake | Airport | Damage Grade | Damage | Observed or Estimated Acceleration |
|---|---|---|---|---|
| 1989 Loma Prieta Earthquake (USA) | San Francisco Airport (International) | 1 | - Hair cracks in runway<br>- Non-structural damage to terminal<br>- Ceilings in control tower fell<br>- Windowpanes of control tower shattered<br>- Airport was closed for 13 hours | 323 gal |
| 1993 Kushiro-oki Earthquake (Japan) | Kushiro Airport (International) | 1 | - Minor cracks in the slopes | 520 gal |
| 1993 Hokkaido Nansei-oki Earthquake (Japan) | Okushiri Airport (Commuter) | 2 | - 20m crack in runway<br>- Airport was closed for 4 days<br>- Damage to landing indicator lights | 392 gal |
| 1995 Kobe Earthquake (Japan) | Kansai Airport (International) | 0 | - No damage | 169 gal |
| 2000 Tottori-ken Seibu Earthquake (Japan) | Yonago Airport (Local) | 2 | - Cracks in runway<br>- Airport was closed for 5 days | 546 gal |
| 2001 Geiyo Earthquake (Japan) | Hiroshima Airport (Local) | 0 | - No damage | 298 gal |
| | Nishi Hiroshima Airport (Commuter) | 1 | - Minor damage | 298 gal |
| | Matsuyama Airport (Local) | 1 | - Minor damage | 298 gal |
| 2001 Seattle Earthquake (USA) | Seattle Sea-Tac Airport (International) | 1 | - Damage to control tower | 194 gal |
| | King County Airport (Boeing Field) | 2 | - Major cracks in runway | 267 gal |

Note
Damage Grade 0 : No Damage
Damage Grade 1 : Minor Damage, Airport will not closed more than 1 day
Damage Grade 2 : Major Damage, Airport will be closed for several days

Table 4-8 Relationship between PGA and Damage Grade

| PGA (gal) | 0 to 200 | 200 to 300 | more than 300 |
|---|---|---|---|
| Damage Grade | 0 | 1 | 2 |

**[Commentary]**

This method has been adopted for this Study as a result of verification of the appropriateness in reference to damage caused by the Boumerdes Earthquake.

## 4-3 Lifelines

It is well known that Japan and the United States of America have methods of seismic damage estimation for lifeline facilities and other infrastructure. In this guideline we introduce the method that is applied in Japan.

The lifelines are classified into five categories: water supply, sewerage, electricity supply, gas supply, and telecommunications. In this section, we introduce the method of damage estimation for each of the above mentioned lifelines.

### 4-3-1 Water Supply

A water supply system consists of various structures and facilities as shown in Figure 4-3.

(1) Water storage facility: To store raw water, it consists of a dam, etc.

(2) Intake facility: To intake raw water from a river, etc. it consists of an intake tower, well, etc.

(3) Headrace facility: To send water taken by the intake facility to a water treatment facility, it consists of a headrace tunnel, etc.

(4) Water treatment facility: To purify water for drinking, it consists of a treatment plant, etc.

(5) Water conveyance facility: To convey drinkable water to a water distribution facility, it consists of a water conveyance pipeline.

(6) Water distribution facility: To distribute water depending on demand, it consists of a distribution reservoir, a water distribution pipeline, etc.

(7) Water supply facility: To supply water from the distribution pipeline to houses, buildings, etc., it consists of a water supply pipeline, equipment, etc.

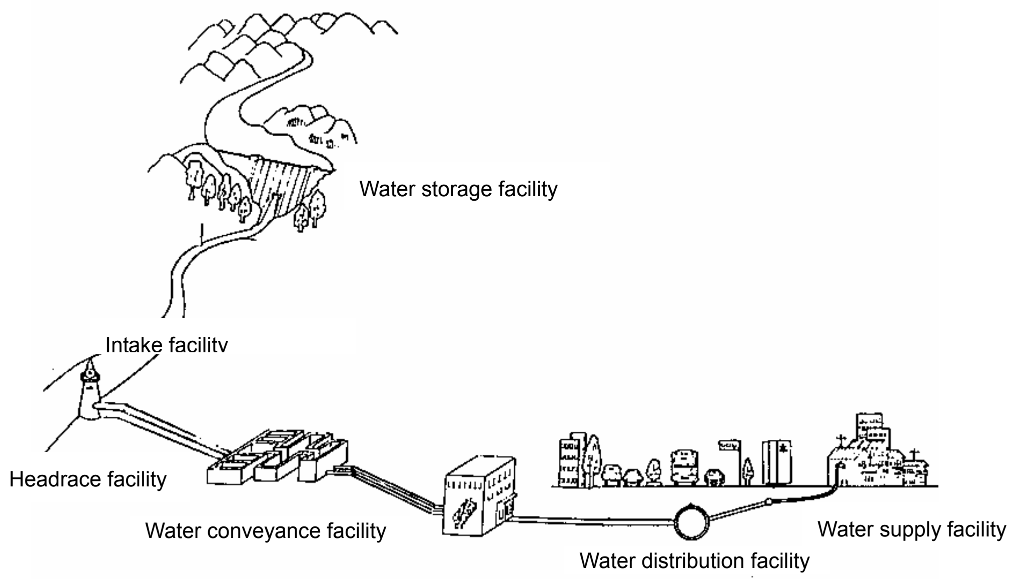



Figure 4-3 Schematic Diagram of Water Supply System

"Workshop on Urban Disaster Prevention and Environment (1988) : Earthquake and Urban Lifeline, Kyoto University Publishing Inc."

Characteristics of the resistance of each facility to earthquakes are summarized as follows;

- Water storage, intake, water treatment and distribution facilities
  These facilities are basically designed in conformity with an aseismic code. It is expected that only slight to moderate damage that does not cause malfunctioning will occur due to an earthquake of a magnitude similar to the past earthquake records. However, it is a possibility that there are some facilities constructed to the old codes. In this case, it is recommended to conduct an aseismic evaluation of them.

- Water treatment facility
  In many cases, the water treatment facilities depend on electric power as a power source. Hence, it is necessary to check/estimate the operating condition during an electric outage.

- Water conveyance, water distribution and water supply facilities
  The major part of the water conveyance, water distribution and water supply facilities are buried pipelines. Out of all the facilities in the water supply system, these facilities have been reported to have suffered the most damage in the past earthquakes.

As mentioned the above, buried pipelines have been reported as having been extensively damaged in the past. Moreover, damage to the pipelines adversely affects various functions. Hence, a damage estimation of the buried pipeline is to be conducted for a damage evaluation of the water supply system.

The damage is estimated by damage points per pipeline length. The damage estimation uses a standard damage ratio based on the past earthquake damage records and a modification coefficient for ground type along with liquefaction potential, pipeline material and pipeline diameter that are independently considered aseismic characteristics. The following three types of formulas are different expressions; however, the concept is the same.

[Municipality in Japan]

$R_{fm} = R_f * C_g * C_p * C_d$

where

$R_{fm}$ : Damage ratio (points/km)
$R_f$ : Standard damage ratio (points/km)
$C_g$ : Modification coefficient for ground type with liquefaction potential
$C_p$ : Modification coefficient for pipeline material
$C_d$ : Modification coefficient for pipeline diameter

[Japan Water Works Association (1998)]

$R_w = C_g * C_l * C_p * C_d * R_{sw}$

where

$R_w$ : Damage ratio (points/km)
$R_{sw}$ : Standard damage ratio (points/km)
$C_g$ : Modification coefficient for ground type
$C_l$ : Modification coefficient for liquefaction potential
$C_p$ : Modification coefficient for pipeline material
$C_d$ : Modification coefficient for pipeline diameter

[Japan Water Research Center (2000)]

(Damage Points) = $C_p * C_d * C_l * S_d * L$

where

$S_d$ : Standard damage ratio (points/km)
$C_p$ : Modification coefficient for pipeline material
$C_d$ : Modification coefficient for pipeline diameter
$C_l$ : Modification coefficient for liquefaction potential
L : Pipe length in subject grid by each pipe material and diameter (km)

The standard damage ratio and the modification coefficient are variously recommended as follows. To select an equation / coefficient from the various recommendations, it is necessary to decide based on a synthesizing judgment of current conditions in the objective area, along with calibration with the past earthquake damage records, and so on.

(1) Standard Damage Ratio

There are three types of indexes to calculate the standard damage ratio as follows;

(1-1) Peak Ground Acceleration (hereinafter referred to as "PGA")

The standard damage ratio by the peak ground acceleration method is derived from the damage records of buried pipelines at the San Fernando earthquake compiled by Kubo and Katayama (1975) as shown in Figure 4-4.

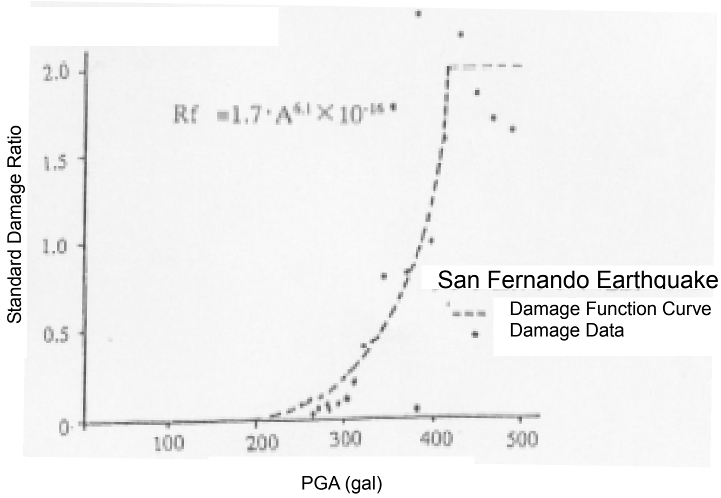



Figure 4-4 Relationship between Peak Ground Acceleration and Standard Damage Ratio for buried Pipelines based on the San Fernando Earthquake (1971)

The equation for the standard damage ratio is as follows;

$R_f = 1.7 * A^{6.1} * 10^{-16}$ ---------- (maximum $R_f = 2.0$)

$R_f$ : Standard Damage Ratio (points/km)
A : Peak ground acceleration (gal)

(1-2) Peak Ground Velocity (hereinafter referred to as "PGV")

There are four formulas for the standard damage ratio by the peak ground velocity method.

1) Based on the damage records of the water supply pipelines at the Kobe Earthquake and other earthquakes.

$R_f = 2.24 * 10^{-3} (V - 20)^{1.51}$

$R_f$ : Standard Damage Ratio (points/km)
V : Peak ground velocity (cm/sec)

2) Based on the damage records of the water supply pipelines in Nishinomiya city at the Kobe Earthquake.

$$R_f = \begin{cases} (V-20) * 0.0125 * 0.8 & \text{(DIP – A, K, T)} \\ (V-70) * 0.0125 * 0.8 & \text{(DIP – S, P)} \\ (V-20) * 0.0125 * 3.0 * (2/3) & \text{(CIP – A)} \\ (V-20) * 0.0125 * 0.8 & \text{(SP)} \end{cases}$$

$R_f$ : Standard Damage Ratio (points/km)
V : Peak ground velocity (cm/sec)
DIP : Ductile cast iron pipe
CIP : Cast iron pipe
SP : Steel pipe
A, K, T, S, P : Shape of connection

3) Based on the damage records of the water supply pipelines at the Kobe Earthquake compiled by the Japan Water Works Association (1998).

$$R_{sw} = \begin{cases} 0 \text{ ---------------------------------- } (V_{max} < 15 \text{ cm/sec}) \\ 3.11 * 10^{-3} (V_{max} - 15)^{1.30} \text{ ------ } (V_{max} \geq 15 \text{ cm/sec}) \end{cases}$$

$R_{sw}$ : Standard Damage Ratio (points/km)
$V_{max}$ : Peak ground velocity (cm/sec)

4) Based on the damage records of the water supply pipelines at the Kobe Earthquake compiled by the Japan Water Research Center (2000).

$$S_d = 6.33 * 10^{-5} V^{2.10} \text{ ----------- } (V \leq 110 \text{ kine})$$

$S_d$ : Standard Damage Ratio (points/km)
$V$ : Peak ground velocity (kine)

(1-3) SI Value

The standard damage ratio by SI value is derived from the damage records of the water supply pipelines at the Kobe Earthquake.

$$R_f = 0.025 * SI - 0.51 \text{ ---------- } (\text{maximum } R_f = 1.5)$$

$R_f$ : Standard Damage Ratio (points/km)
SI : SI value (cm/sec)

(2) Modification Coefficient for Ground Type with Liquefaction Potential

This coefficient is based on regional features. Table 4-9 is tabulated for the coefficients applied in municipalities in Japan, and Table 4-10 and Table 4-11 shows values recommended by the Japan Water Works Association and the Japan Water Research Center, respectively.

Table 4-9 Modification Coefficient for Ground Type with Liquefaction Potential in each Municipality in Japan

<table>
<tr><th>Ground Type</th><th>AKITA, NIIGATA, HIROSHIMA, MIYAZAKI Prefecture</th><th colspan="2">Ground Type</th><th>SENDAI City</th></tr>
<tr><td>Hill</td><td>0.5</td><td colspan="2">Hill</td><td>0.4</td></tr>
<tr><td>Plateau</td><td>0.5</td><td colspan="2">Plateau</td><td>0.5</td></tr>
<tr><td>Alluvial Plain</td><td>1.0</td><td colspan="2">Alluvial Plain</td><td>1.0</td></tr>
<tr><td>Very Soft Ground</td><td>2.0</td><td colspan="2">Very Soft Ground,<br>Land fill</td><td>2.0</td></tr>
<tr><th>Ground Type</th><th>MIYAGI Prefecture</th><th colspan="2">Ground Type</th><th>FUKUI Prefecture</th></tr>
<tr><td>Before Alluvium</td><td>0.5</td><td colspan="2">Diluvium</td><td>0.5</td></tr>
<tr><td>Alluvial Ground</td><td>1.0</td><td colspan="2">Alluvium</td><td>1.0</td></tr>
<tr><td>Humus Soil</td><td>2.0</td><td colspan="2">Alluvium (Humus)</td><td>2.0</td></tr>
<tr><td>Land fill</td><td>2.0</td><th colspan="2">Ground Type</th><th>AOMORI Prefecture</th></tr>
<tr><th>Ground Type</th><th>SAITAMA Prefecture</th><td colspan="2">Diluvium and Better Ground</td><td>0.5</td></tr>
<tr><td>Dc, Ds, Dg</td><td>0.5</td><td rowspan="3">Alluvium</td><td>Loam</td><td>0.9</td></tr>
<tr><td>Lm</td><td>0.9</td><td>Clay, Sand</td><td>1.0</td></tr>
<tr><td>Ac, As</td><td>1.0</td><td>Humus</td><td>2.0</td></tr>
<tr><td>Ap</td><td>2.0</td><th colspan="2">Ground Type</th><th>YAMAGUCHI Prefecture, HIROSHIMA City</th></tr>
<tr><th>Ground Type</th><th>NAGANO Prefecture</th><td colspan="2">Type1</td><td>0.6</td></tr>
<tr><td>Type1</td><td>0.6</td><td colspan="2">Type2</td><td>1.3</td></tr>
<tr><td>Type2</td><td>1.3</td><td colspan="2">Type3</td><td>1.3</td></tr>
<tr><td>Type3</td><td>1.3</td><td colspan="2">Type4</td><td>1.9</td></tr>
<tr><td>Type4</td><td>1.9</td><td colspan="2">Boundary of Ground Type</td><td>2.5</td></tr>
</table>

<table>
<tr><th>Index of Liquefaction Potential ($P_L$)</th><th>0</th><th>5</th><th>10</th><th>15</th><th>20</th></tr>
<tr><td>TOKYO 97, KAWASAKI City</td><td>1.0</td><td>1.2</td><td>1.5</td><td colspan="2">3.0</td></tr>
<tr><td>SAPPORO City</td><td>1.0</td><td>1.1</td><td>1.3</td><td colspan="2">2.1</td></tr>
<tr><td>SHIZUOKA Prefecture</td><td>1.0</td><td>1.0</td><td>2.9</td><td colspan="2">4.7</td></tr>
<tr><td>MIYAGI, MIYAZAKI Prefecture, SENDAI City</td><td>-</td><td>-</td><td colspan="2">2.9</td><td>4.7</td></tr>
<tr><td>AOMORI, AKITA, SAITAMA, HIROSHIMA Prefacture</td><td>-</td><td>-</td><td>2.9</td><td colspan="2">4.7</td></tr>
<tr><td>FUKUI Prefecture</td><td>-</td><td>-</td><td>2.5</td><td colspan="2">3.5</td></tr>
<tr><td>NIIGATA Prefecture</td><td>-</td><td colspan="2">-</td><td colspan="2">3.0</td></tr>
</table>

If both values (coefficient of the ground type and the liquefaction potential) are given in the above table for a municipality, the bigger value is applied.

Table 4-10 Modification coefficient for Ground Type and Liquefaction Potential recommended by the Japan Water Works Association

| Ground Type | $C_g$ |
|---|---|
| Mountains, Land fill in Mountains | 1.1 |
| Flatland in terraces, Hills | 1.5 |
| Old channel, Back marsh, Land fill in plains, Beach ridge | 3.2 |
| Flatland in bottom of gorge, Fan, Cliff, Natural levee (developed part, un-developed part) | 1.0 |

| Index of Liquefaction Potential | | $C_l$ |
|---|---|---|
| Liquefaction Potential | High ($P_L$>15) | 2.4 |
| | Moderate (5<$P_L$≤15) | 2.0 |
| | Low (0≤ $P_L$≤5) | 1.0 |

Table 4-11 Modification coefficient for Liquefaction Potential recommended by the Japan Water Research Center

| Liquefaction Potential | $C_l$ |
|---|---|
| No (0≤ $P_L$≤5) | 0.9 |
| Partially (5<$P_L$≤15) | 1.0 |
| Totally ($P_L$>15) | 1.6 |

(3) Modification Coefficient for Pipeline Material and Pipeline Diameter

This coefficient is based on analysis of the past earthquake damage records. There are two ways to decide the coefficient applied by the municipalities: one way is applying the product of the pipeline material coefficient and the pipeline diameter coefficient (see Table 4-12 to Table 4-13), another way is individually applying the coefficients (see Table 4-14 to Table 4-17). The Japan Water Association and Water Technology Foundation have recommended the latter way (see Table 4-18 to Table 4-19)

Table 4-12 Modification Coefficient for Pipeline Material and Pipeline Diameter applied in FUKUOKA Prefecture

| Type \ Diameter (mm) | ≤ 75 | 100 - 125 | 150 - 350 | 400 - |
|---|---|---|---|---|
| Asbestos Cement | 10.2 | 5.3 | 3.9 | 3.3 |
| PVC | 2.6 | 1.9 | 1.9 | - |
| Cast Iron | 1.4 | 1.0 | 0.8 | 0.3 |
| Ductile Cast Iron | 1.1 | 0.5 | 0.5 | 0.1 |
| Steel with Screw Joint | 10.5 | 5.5 | 4.0 | 3.4 |
| Welding Steel | 0.5 | 0.3 | 0.2 | 0.1 |

Table 4-13 Modification Coefficient for Pipeline Material and Pipeline Diameter applied in MIE Prefecture

| Type \ Diameter (mm) | ≤ 75 | 100 - 150 | 200 - 250 | 300 - 450 | 500 - |
|---|---|---|---|---|---|
| Ductile Cast Iron | 2.1 | 1.0 | 1.0 | 1.0 | 0.1 (0.2) |
| Cast Iron | 1.7 | 1.2 | 1.1 | 0.6 | 0.2 |
| Steel | 2.8 | 1.5 | 1.3 | 0.9 | 0.8 |

Table 4-14 Modification Coefficient for Pipeline Material and Pipeline Diameter applied in TOKYO 97, SAPPORO City and KAWASAKI City

| Pipe Material | Pipe Diameter φ (mm) | Coefficient |
|---|---|---|
| Ductile Cast Iron | φ ≤ 75 | 0.6 |
| | 100 <φ ≤ 450 | 0.3 |
| | 500 <φ ≤ 900 | 0.09 |
| | 1,000 <φ | 0.045 |
| Cast Iron | φ ≤ 75 | 1.7 |
| | 100 <φ ≤ 250 | 1.2 |
| | 300 <φ ≤ 900 | 0.4 |
| | 1,000 <φ | 0.15 |
| Steel | φ ≤ 75 | 0.84 |
| | 100 <φ ≤ 250 | 0.42 |
| | 300 <φ | 0.24 |
| PVC | φ ≤ 75 | 1.5 |
| | 100 <φ | 1.2 |
| Asbestos Cement | φ ≤ 75 | 6.9 |
| | 100 <φ ≤ 250 | 2.7 |
| | 300 <φ | 1.2 |

Table 4-15 Modification Coefficient for Pipeline Material and Pipeline Diameter applied in SHIZUOKA Prefecture

| Pipe Material | Coefficient of Pipe Material | Coefficient of Pipe Diameter | |
|---|---|---|---|
| Steel (screw) | 10.0 | < 100 mm | 1.3 |
| | | 100 mm ≤ | 0.75 |
| Steel (welded) | 0.1 | < 1,000 mm | 1.0 |
| | | 1,000 mm ≤ | 0.5 |
| Cast Iron | 1.0 | < 400 mm | 1.5 |
| | | 400 – 1,000 mm | 0.3 |
| | | 1,000 mm ≤ | 0.15 |
| Ductile Cast Iron | 0.25 | < 500 mm | 1.3 |
| | | 500 – 1,000 mm | 0.3 |
| | | 1,000 mm ≤ | 0.15 |
| Asbestos Cement | 3.0 | < 100 mm | 2.3 |
| | | 125 – 250 mm | 0.9 |
| | | 300 mm ≤ | 0.4 |
| PVC | 1.5 | < 100 mm | 1.1 |
| | | 100 mm ≤ | 0.9 |

Table 4-16 Modification Coefficient for Pipeline Material applied in each Municipality

<table>
<tr><th>Pipe Material</th><th>MIYAGI</th><th>AOMORI SAITAMA</th><th>KANAGAWA</th><th>AKITA NAGANO</th><th>HIROSHIMA</th><th>NIIGATA* MIYAZAKI</th><th>VILLE SENNDAI</th><th>YAAGUCHI</th><th>FUKUI</th></tr>
<tr><td>Cast Iron</td><td colspan="9">1.0</td></tr>
<tr><td>Ductile Cast Iron</td><td colspan="8">0.2</td><td>0.3</td></tr>
<tr><td>Steel</td><td colspan="3">-</td><td colspan="3">2.0</td><td colspan="3">-</td></tr>
<tr><td>Welded Steel</td><td colspan="8">0.1</td><td>0.2</td></tr>
<tr><td>Steel with Screw</td><td>2.0</td><td>-</td><td>2.0</td><td colspan="5">-</td><td>2.8</td></tr>
<tr><td>Stainless Steel</td><td>0.1</td><td colspan="8">-</td></tr>
<tr><td>Socketed Steel</td><td colspan="2">-</td><td>0.8</td><td colspan="5">-</td><td>0.8</td></tr>
<tr><td>Lead</td><td>0.8</td><td colspan="4">-</td><td>1.0*</td><td colspan="3">-</td></tr>
<tr><td>PVC</td><td colspan="5">1.5</td><td colspan="2">1.0</td><td>0.8</td><td>1.2</td></tr>
<tr><td>Asbestos Cement</td><td colspan="4">4.0</td><td>2.0</td><td>1.0</td><td>4.0</td><td>1.3</td><td>2.8</td></tr>
<tr><td>Polyethylene</td><td colspan="2">-</td><td colspan="4">0.1</td><td colspan="2">-</td><td>0.2</td></tr>
<tr><td>Concrete</td><td colspan="5">-</td><td>1.0</td><td colspan="3">-</td></tr>
<tr><td>Main Line</td><td>0.1</td><td colspan="8">-</td></tr>
</table>

*NIIGATA : The coefficient of lead pipe is not established.

Table 4-17 Modification Coefficient for Pipeline Diameter applied in each Municipality

<table>
<tr><th>Pipe Diameter (mm)</th><th colspan="17">100 200 300 400 500 600 700 800 900 1,000 1,100</th></tr>
<tr><td>MIYAGI</td><td>1.3</td><td colspan="5">0.9</td><td colspan="2">0.6</td><td colspan="3">0.5</td><td colspan="6">0.3</td></tr>
<tr><td>NAGANO</td><td colspan="3">1.33</td><td colspan="2">-</td><td colspan="4">0.67</td><td colspan="2">-</td><td colspan="4">0.50</td><td colspan="2">0.33</td></tr>
<tr><td>KANAGAWA</td><td>1.3</td><td colspan="10">0.8</td><td colspan="4">0.4</td><td colspan="2">0.2</td></tr>
<tr><td>AKITA<br>HIROSHIMA<br>SENNDAI (City)</td><td colspan="4">1.2</td><td colspan="5">0.6</td><td colspan="2">-</td><td colspan="4">0.4</td><td colspan="2">0.2</td></tr>
<tr><td>AOMORI</td><td colspan="4">1.2</td><td colspan="7">0.6</td><td colspan="5">0.4</td><td>0.2</td></tr>
<tr><td>SAITAMA</td><td colspan="4">1.2</td><td colspan="5">0.6</td><td colspan="2">-</td><td colspan="4">0.4</td><td>-</td><td>0.2</td></tr>
<tr><td>NIIGATA</td><td colspan="2">2.0</td><td colspan="2">*</td><td colspan="5">0.6</td><td colspan="2">-</td><td colspan="4">0.4</td><td>-</td><td>0.2</td></tr>
<tr><td>FUKUI</td><td colspan="4">1.0</td><td colspan="5">0.6</td><td colspan="2">-</td><td colspan="4">0.4</td><td>-</td><td>0.2</td></tr>
<tr><td>MIYAZAKI</td><td colspan="8">1.0</td><td colspan="4">0.5</td><td colspan="3">0.4</td><td colspan="2">0.2</td></tr>
<tr><td>YAMAGUCHI</td><td>1.0</td><td colspan="4">0.8</td><td colspan="2">0.6</td><td>-</td><td colspan="4">0.4</td><td>-</td><td>0.3</td><td>-</td><td colspan="2">0.1</td></tr>
<tr><td>HIROSHIMA (City)</td><td>1.0</td><td colspan="4">-</td><td colspan="2">0.6</td><td>-</td><td colspan="4">0.4</td><td>-</td><td>0.3</td><td>-</td><td colspan="2">0.1</td></tr>
</table>

* NIIGATA: The coefficient for 100 to 125 mm diameter is 1.5.
"-" shows that no coefficient value was given for the corresponding pipe diameter.

Table 4-18 Modification Coefficient for Pipeline Material and Pipeline Diameter recommended by the Japan Water Works Association

| Pipe Material | $C_p$ |
|---|---|
| Welded Steel | 0.3 |
| Ductile Cast Iron | 0.3 |
| Cast Iron | 1.0 |
| PVC | 1.0 |
| Asbestos Cement | 1.2 |
| Steel with Screw | 2.0 |
| Polyethylene | 0.1 |
| Other Pipe Materials | 1.0 |
| Unknown | 1.0 |

| Pipe Diameter | $C_d$ |
|---|---|
| 500 mm and more | 0.5 |
| 200 – 450 mm | 0.8 |
| 100 – 150 mm | 1.0 |
| 75 mm and less | 1.6 |

Table 4-19 Modification Coefficient for Pipeline Material and Pipeline Diameter recommended by the Japan Water Research Center

| Pipe Material | $C_p$ |
|---|---|
| Ductile Cast Iron (A, K, T)* | 0.3 |
| Ductile Cast Iron (S, SII)* | 0.0 |
| Cast Iron | 1.0 |
| Welded Steel | 0.3** |
| PVC | 1.0 |
| Steel with Screw | 4.0** |
| Asbestos Cement | 2.5** |

* : A, K, T, S, SII: Joint shape
**: Reference value due to only obtaining short length of data.

| Pipe Diameter | $C_d$ |
|---|---|
| 75 mm | 1.6 |
| 100 to 150 mm | 1.0 |
| 200 to 250 mm | 0.9 |
| 300 to 450 mm | 0.7 |
| 500 to 600 mm | 0.5*** |

**: Reference value due to only obtaining small amount of data.

**[Commentary]**

(1) Standard Damage Ratio

There are three ways, as mentioned above, to calculate the standard damage ratio.

According to a recent study, damages due to earthquakes have a better correlation with the peak ground velocity than the peak ground acceleration. It is also reported that the SI value has a very high correlation with the amount of damage. It is expected that the standard damage ratio as calculated using the peak ground velocity or the SI value will be applied with increasing frequency in the near future.

Meanwhile, the standard damage ratio by the peak ground acceleration has been applied in many municipalities in Japan and it is easier to use this method in order to compare / discuss the adequacies in the present situation.

In this Study, we applied the standard damage ratio as calculated using the peak ground acceleration, because distribution of the peak ground acceleration was obtainable in the hazard maps.

(2) Modification Coefficient for Ground Type with Liquefaction Potential

Various values of determining the modification coefficient for ground type with liquefaction potential have been applied.

In this Study, a moderate value from among the above mentioned values was used in calculating the standard damage ratio using the peak ground acceleration as it was not possible to obtain enough information to determine the value more accurately.

(3) Modification Coefficient for Pipeline Material and Pipeline Diameter

Various values of the modification coefficient for pipeline material and pipeline diameter have been applied as well as the modification coefficient for ground type with liquefaction potential.

In this Study, a moderate value from among the above mentioned values was used in calculating the standard damage ratio using the peak ground acceleration method as it was not possible to obtain enough information to determine the value more accurately.

### 4-3-2 Sewerage

A sewerage system consists of the following structures and facilities (see Figure 4-5).

(1) Drainage facility: To drain sewage and rainwater from houses / facilities through a buried drainage pipeline, an open channel, etc.

(2) Collecting facility: To collect the sewage and the rainwater from the drainage facility using a collecting conduit with manholes, open conduits, etc.

(3) Pump facility: To pump the collected sewage and rainwater to a higher elevation in order to adjust a natural flow gradient.

(4) Purification facility: To purify the collected sewage and rainwater and to discharge the purified water to a river / sea.

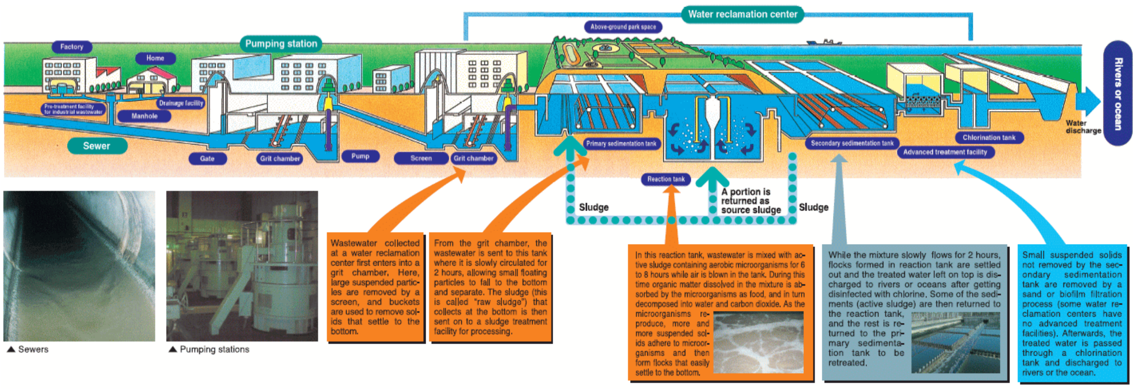



Figure 4-5 Schematic Diagram of a Sewerage System

(Reference: Bureau of Sewerage Tokyo Metropolitan Government, http://www.gesui.metro.tokyo.jp/)

Among the above mentioned facilities, the buried pipelines (the buried drainage pipelines and the collecting conduits) and the manholes have been reported as suffering a great deal of damage due to past earthquakes while the pump facility and the purification facility are generally constructed in conformity with aseismic code and therefore suffer less damage. Hence, a damage estimation of the sewerage pipelines was conducted for the damage evaluation of the sewerage system.

In the recent past there were far fewer sewerage systems in Japan and therefore, there are not a great many damage records regarding sewers available. Consequently, a damage estimation method similar to the method applied for the water supply pipelines has been applied for the sewerage pipelines (see the section 4.3.1) in many studies.

**[Commentary]**

As mentioned above, damage estimation for the sewerage system is the same as the water supply pipelines under the present conditions. However, it is expected that an individual method for the sewerage system will be established in near future.

In this Study, we were only able to obtain data regarding the buried main line network, in which the pipeline diameter is around 1 m and more. Hence, we judged that the qualitative damage evaluation is better than the damage estimation for the water supply pipeline.

### 4-3-3 Electric Power Supply

The electric power supply system consists of the following structures and facilities (see Figure 4-6)

(1) Power plant: To generate electric power (a nuclear power plant, a thermal power plant, a hydraulic power plant, etc.)

(2) Electric power transmission facility: To transmit electric power from the power plant (high voltage cables, transmission line towers, conduit, transformer substations, etc.)

(3) Electric power distribution facility: To distribute the medium and low voltage electric power to consumers such as households, factories and so on (medium and low voltage cables, electric poles, protection pipes, transformer substations for power distribution, etc.)

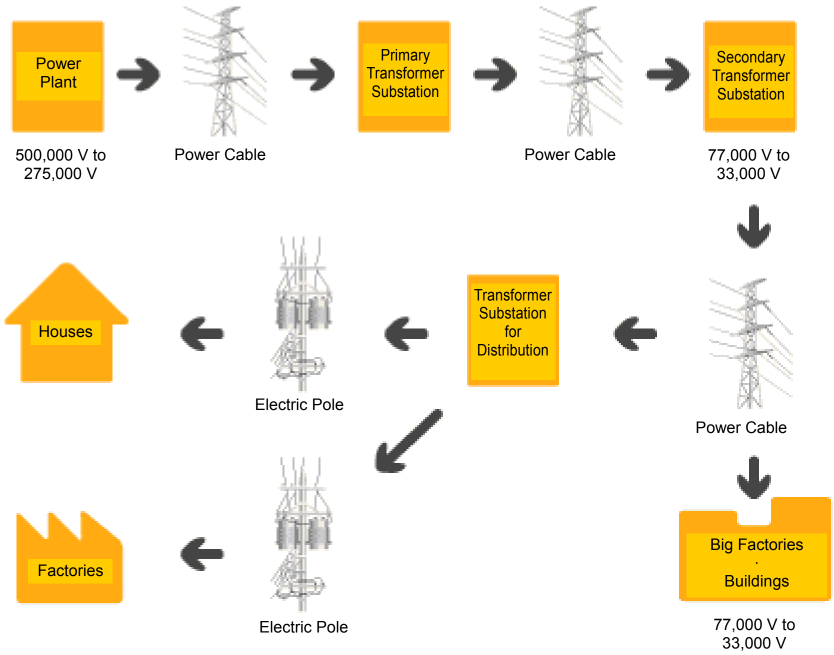



Figure 4-6 Schematic Diagram of Electric Power Supply System

(Reference: Chubu Electric Power Co., Inc, https://link.chuden.jp/)

Among the above mentioned facilities, the medium and low voltage cables and the electric poles have been reported as suffering a great deal of damage due to earthquakes, while the power plant, the electric power transmission facilities related with high voltage and the transformers are generally constructed in conformity with an aseismic code and therefore suffer less damage. Hence, damage estimations for the electric poles and the medium and low voltage cables were conducted for the damage evaluation of the electric power supply system.

The damage estimations are generally conducted for aerial cables and underground cables separately. The damage is estimated so that the length of the aerial cable damaged and the underground cable damaged is calculated separately for each grid and then they are summed.

The following shows the damage estimation method in the sequence for the electric poles, the aerial cables and the underground cables.

(1) Electric Poles

The damage estimation for the electric poles applies two methods.

(1-1) Method of using Formula based on the Standard Damage Ratio

This method has two types: the first was used prior to the Kobe Earthquake and the second takes into consideration the damage records of the Kobe Earthquake.

1) Method before Kobe Earthquake (1991)

$$N_h = C_{gl} * R(A) * N$$

$$R(A) = \begin{cases} 0 & (A < 150 \text{ gal}) \\ 0.0053A - 0.795 & (150 \le A < 300 \text{ gal}) \\ 0.8 & (300 \text{ gal} \le A) \end{cases}$$

$N_h$ : Number of electric pole damaged
$C_{gl}$ : Modification coefficient for ground type with liquefaction potential (same as the water supply pipeline)
R(A) : Standard damage ratio
A : Peak ground acceleration (gal)
N : Number of poles

2) Method with Consideration of Kobe Earthquake (1997)

$$N_d^P = C_l * R / 100 * N + N_f * N$$

$N_d^P$ : Number of electric pole damaged
$C_l$ : Modification coefficient for liquefaction potential (same as the water supply pipeline)
R : Standard damage ratio

| JMA Intensity / Municipality | 5 + and less | 6 - | 6 + | 7 |
|---|---|---|---|---|
| AOMORI Prefecture | 0.00 | 0.47 | | 6.68 |
| TOKYO 97 | 0.00 | 0.55 | | No setting |
| SAPPORO City | 0.00 | 0.47 | 2.86 | 6.68 |
| KAWASAKI City | 0.00 | 0.47 | | No setting |

N : Number of poles
$N_f$ : Burned out ratio

(1-2) Method considering the Relationship Matrix between Earthquake Motion and Damage Ratio

This method has two types: one uses the peak ground acceleration and the other uses the intensity scale (Japan Meteorology Agency intensity scale, hereinafter referred to as "JMA").

1) Method using PGA

This method has two types: one is based on the Niigata Earthquake and the Miyagi Offshore Earthquake and the other additionally takes into consideration the Kobe Earthquake.

1-1) Method based on Niigata Earthquake and Miyagi Offshore Earthquake (1986)

Table 4-20 Number of Electric Poles Damaged per 100 Poles based on Damage Records of Niigata Earthquake and Miyagi Offshore Earthquake

| PGA (gal) / Electric Pole | | KANAGAWA, AKITA, TOYAMA, FUKUI, MIYAZAKI Prefecture | | | | |
|---|---|---|---|---|---|---|
| | | ≤150gal | 151 – 300gal | 301 – 400gal | 401gal≤ | Liquefaction Area: $P_L$≥15 |
| Broken Poles | Concrete | 0.00 | 0.00 | 0.01 | 0.03 | 0.9 |
| | Wood | 0.00 | 0.00 | 0.01 | 0.02 | 0.2 |
| Poles Collapsed | Concrete | 0.00 | 0.00 | 0.03 | 0.10 | 3.4 |
| | Wood | 0.00 | 0.00 | 0.01 | 0.03 | 0.9 |

1-2) Method based on Niigata Earthquake, Miyagi Offshore Earthquake and Kobe Earthquake (1998)

Table 4-21 Number of Electric Poles Damaged per 100 Poles based on Damage Records of Niigata Earthquake, Miyagi Offshore Earthquake and Kobe Earthquake

| PGA (gal) / Electric Pole | | SAITAMA Prefecture | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | ≤150gal | 151 – 300gal | 301 – 400gal | 401 – 600gal | 601 – 800gal | 801 gal≤ | Liquefaction Area: $P_L$≥15 |
| Broken / Collapsed Poles | Concrete | 0.0 | 0.0 | 0.04 | 0.13 | 1.0 | 3.2 | 4.3 |
| | Wood | 0.0 | 0.0 | 0.02 | 0.05 | 0.4 | 1.3 | 1.1 |

2) Method using JMA

This method has two types: one is based on the Nihon-kai Chubu Earthquake and the Kobe Earthquake and the other is based on the Kobe Earthquake.

It is noted that there is another method that takes into consideration the spread of fire, however, this method strongly reflects the Japanese housing conditions (there are a plenty of wooden houses). Consequently, this method is omitted here.

2-1) Method based on Nihon-kai Chubu Earthquake and Kobe Earthquake

Table 4-22 Ratio of Electric Poles Damaged based on Damage Records of Nihon-kai Chubu Earthquake and Kobe Earthquake

| JMA Intensity | Standard Damage Ratio | FUKUOKA Prefecture | |
|---|---|---|---|
| | | Coefficient of Liquefaction | Damage Ratio |
| 5 + and less | No damage | - | No damage |
| 6 - | 0.13 % | $0.98 + 0.014 P_L$ | $0.13 + 0.0018 P_L$ |
| 6 + and more | 0.49 % | $0.99 + 0.006 P_L$ | $0.49 + 0.0029 P_L$ |

2-2) Method based on Kobe Earthquake

Table 4-23 Ratio of Electric Poles Damaged based on Damage Records of Kobe Earthquake

| JMA Intensity | TOCHIGI Prefecture | |
|---|---|---|
| | Installation of transformer, other equipment | |
| | Exist on pole | Nothing on pole |
| 7 | 1.8 % | 1.3 % |

(2) Aerial Cables

The damage estimation for the aerial cables applies two methods.

(2-1) Method using the Damage Estimation for Electric Poles

The method applies 4 types of formula. Each formula has differences; however, the basic concept is that the damage is calculated by multiplying the number of poles damaged and a damage ratio that takes into consideration a relationship between the poles and the aerial cables.

The following shows the formulas applied by each municipality.

1) TOKYO 91, MIYAGI, KANAGAWA, YAMANASHI, SHIZUOKA Prefectures

$N_d^C = 0.5$ [span/pole] * L [m/span] * $N_d^P$

$N_d^C$ : Length of damaged cable (km)
L : Average cable length per span
$N_d^P$ : Number of poles damaged

2) TOKYO 97

$n_d^C = a * N_d^P / L$

$n_d^C$ : Cable damage ratio (km)
a : Ratio of damaged cables per pole, a = 0.396 (based on the Kobe Earthquake damage records )
$N_d^P$ : Number of poles damaged
L : Cable length (km)

3) SAPPORO City

$N_d^C = L * N_d^P / N + N_f * L$

$N_d^C$ : Length of damaged cable (km)
L : Cable length (km)
$N_d^P$ : Number of poles damaged
N : Number of poles
$N_f$ : Burned out ratio

4) KAWASAKI City

$N_d^C = a * N_d^P * L$

$N_d^C$ : Length of damaged cable (km)
A : Ratio of damage cable per 1 pole, a = 0.5 (based on damage records on the past earthquakes)
$N_d^P$ : Number of poles damaged
L : Average cable length per span

(2-2) Method considering the Relationship Matrix between PGA and the Damage Ratio

This method has two types: the first was used prior to the Kobe Earthquake and the second considers the damage records of the Kobe Earthquake.

1) Method before Kobe Earthquake (1986)

Table 4-24 Cable Damage Ratio (Span/100 Poles)

| Electric Pole \ PGA (gal) | KANAGAWA, AKITA, FUKUI, MIYAZAKI Prefecture | | | | |
|---|---|---|---|---|---|
| | ≤150gal | 151 – 300gal | 301 – 400gal | 401gal ≤ | Liquefaction Area: $P_L$≥15 |
| Concrete | 0 | 0.01 | 0.32 | 1.2 | 11.0 |
| Wood | 0 | 0.002 | 0.05 | 0.18 | 2.6 |

2) Method Considering the Kobe Earthquake (1997)

Table 4-25 Cable Damage Ratio (Span/100 Poles) based on Damage Records of Kobe Earthquake

| Electric Pole \ PGA (gal) | | SAITAMA Prefecture | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | ≤150gal | 151 – 300gal | 301 – 400gal | 401 – 600gal | 601 – 800gal | 801 gal≤ | Liquefaction Area: $P_L$≥15 |
| Broken / Collapsed Cables | Concrete | 0.0 | 0.01 | 0.32 | 1.20 | 8.5 | 27.0 | 11.0 |
| | Wood | 0.0 | 0.002 | 0.05 | 0.18 | 1.3 | 4.1 | 2.6 |

(3) Underground Cable

This method has two types: the first was used prior to the Kobe Earthquake and the second considers the damage records of the Kobe Earthquake.

(3-1) Method before Kobe Earthquake (1991)

$$L_c = C_{gl} * R(A) * L$$

$$R(A) = \begin{cases} 0 & (A < 200 \text{ gal}) \\ 0.002A - 0.4 & (200 \leq A < 300 \text{ gal}) \\ 0.2 & (300 \text{ gal} \leq A) \end{cases}$$

$L_c$ : Length of damaged cable (km)
$C_{gl}$ : Modification coefficient for ground type with liquefaction potential (same as the water supply pipeline)
$R(A)$ : Standard damage ratio
$A$ : Peak ground acceleration (gal)
$L$ : Cable length (km)

(3-2) Method Considering the Kobe Earthquake (1997)

The method uses the same formula; however, a different standard damage ratio is applied.

The damage estimation for underground telecommunication cables described afterwards applies the same method in many cases. Here, the standard damage ratio is also described.

$N_d = C_l * R / 100 * L$

$N_d$ : Length of damaged cable (km)
$C_l$ : Modification coefficient for liquefaction potential (same as the water supply pipeline)
L : Cable length (km)
R : Standard damage ratio

| JMA Intensity | Standard Damage Ratio | | |
|---|---|---|---|
| | TOKYO 97 | AOMORI Prefecture* | SAPPORO City* |
| 5 and less | 0.00 | 0.00 | 0.00 |
| 6 - | 0.30 | 0.30 | 0.30 |
| 6 + | | | 2.00 |
| 7 | - ** | 4.70 | 4.70 |

*: For telecommunication
**: Not reported

**[Commentary]**

The low voltage cable is included in the objective facility for the damage estimation for the electric power supply system. However, in this Study the damage estimation for the low voltage cable was not conducted because of following reasons.

- The low voltage cables in the Wilaya of Algiers are very complicated and their digitization was not possible.
- The cables are generally distributed along the buildings.
- The damage to low voltage cable is estimated as being comparable to building damage.

The procedure for the damage estimation in this Study is shown below.

(1) Electric Poles

When the damage to the electric pole is estimated, it is necessary to know the distribution (number) of the electric poles. However, this data was not available in this Study.

Therefore, the damage to electric poles was treated as the damage ratio according to the method described in (1) – (1-1) – 2). Here, the spread of fire was not considered because of the building conditions in the Wilaya of Algiers.

$$N_d^P = C_l * R / 100 * N + N_f * N$$
$$= C_l * R / 100 * N + 0$$
$$= x * N \text{ ------------------------- } (x = C_l * R / 100)$$

In this calculation, the standard damage ratio was calculated based on the related data.

(2) Aerial Cables

The damage to aerial cables was estimated by the above mentioned electric pole damage ratio and the method described in (2) (2-1) -2). This was based on the assumption that the ratio of the number of electric poles and the aerial cable length is constant (wherever the cable length of 1 span is equal).

$$n_d^C = a * N_d^P / L$$
$$= a * x * (N / L)$$
$$= b \quad \text{--------------------------} \ (b = a * x, N / L = \text{const.})$$

Consequently, the damage to aerial cables was estimated by multiplying the calculated damage ratio and total length of the aerial cable in each grid.

(3) Underground Cables

The damage estimation for the underground cables was the method described in (3) – (3-2), because the method is applied in many cases. In this calculation, the standard damage ratio was calculated based on the related data as well as the electric poles.

### 4-3-4 Gas Supply

A gas supply system consists of the following structures and facilities (see Figure 4-7)

(1) Gas product plant: To refine the gas.
(2) High pressure gas facility: To transmit high pressure gas to a transmitting station (a high pressure gas pipeline, a transformer, etc.)
(3) Medium pressure gas facility: To store the gas and distribute the same to various locations (a medium pressure gas pipeline, a gas storage unit, a transformer, etc.)
(4) Low pressure gas facility: To distribute the gas to consumers such as households, buildings, etc. (a low pressure gas pipeline, a regulator / controller equipment、etc.)

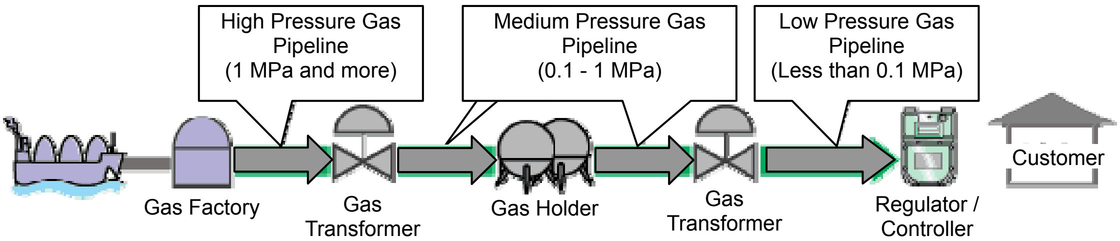



Figure 4-7 Schematic Diagram of Gas Supply System

(Reference: Osaka Gas Co., Ltd, http://www.osakagas.co.jp)

Among the above mentioned facilities, the medium and low pressure gas pipeline and the regulator equipment have been reported as suffering a great deal of damage due to past earthquakes while the gas product plant, the high pressure gas facility and the transformer are generally constructed in conformity with an aseismic code and therefore, suffer less damage. Hence, the damage estimation for the medium and low pressure gas pipelines was conducted for damage evaluation of the gas supply system.

The damage is estimated by damage points per pipeline length as well as the water supply pipeline. The damage estimation uses a standard damage ratio based on the past earthquake damage records and a modification coefficient for the ground type with liquefaction potential, pipeline material and pipeline diameter that are considered independent aseismic characteristics. The following formula shows the basic concept of the damage estimation.

$R_{fm} = R_f * C_g * C_p * C_d$

where

$R_{fm}$ : Damage ratio (points/km)
$R_f$ : Standard damage ratio (points/km)
$C_g$ : Modification coefficient for ground type with liquefaction potential
$C_p$ : Modification coefficient for pipeline material
$C_d$ : Modification coefficient for pipeline diameter

The standard damage ratio and the modification coefficient are variously recommended as follows. To select an equation / coefficient from the various recommendations, it was necessary to choose based on a synthesizing judgment of current conditions in the objective area, calibration with the past earthquake damage records, and so on.

(1) Standard Damage Ratio

There are three types of indexes to calculate the standard damage ratio as follows;

(1-1) Peak Ground Acceleration

The standard damage ratio based on the peak ground acceleration is derived from the damage records of buried pipelines in the San Fernando earthquake compiled by Kubo and Katayama (1975) as shown in Figure 4-8.

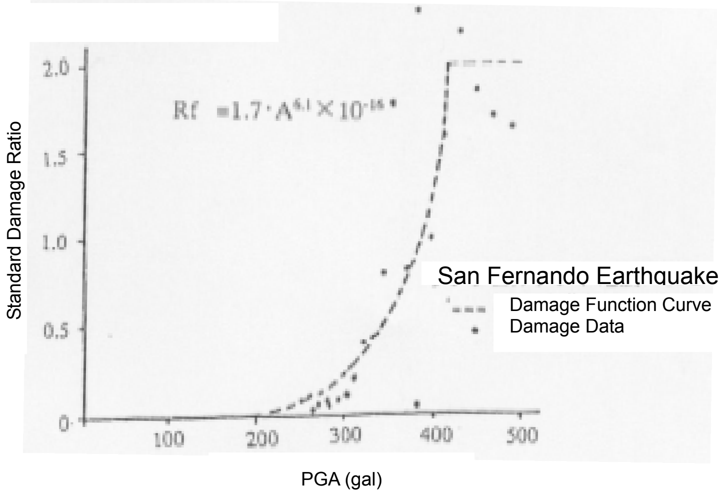



Figure 4-8 Relationship between Peak Ground Acceleration and Standard Damage Ratio for buried Pipelines based on the San Fernando Earthquake (1971)

The equation for the standard damage ratio is as follows;

$R_f = 1.7 * A^{6.1} * 10^{-16}$ ---------- (maximum $R_f = 2.0$)
$R_f$ : Standard Damage Ratio (points/km)
$A$ : Peak ground acceleration (gal)

(1-2) Peak Ground Velocity

The standard damage ratio based on the peak ground velocity is derived from the damage records of the gas supply pipelines in the Kobe Earthquake.

$R_f = 3.89 * 10^{-3} (V - 20)^{1.51}$

$R_f$ : Standard Damage Ratio (points/km)

V : Peak ground velocity (cm/sec)

(1-3) SI Value

The standard damage ratio based on the SI value is derived from the damage records of the gas supply pipelines in the Kobe Earthquake.

In this method, two formulas are applied by the municipalities.

1) FUKUOKA Prefecture

$R_f = 0.025 * SI - 0.76$ ---------- (maximum $R_f = 1.8$)

$R_f$ : Standard Damage Ratio (points/km)

SI : SI value (cm/sec)

2) NIIGATA, HIROSHIMA Prefecture, HIROSHIMA City

$R_f = 0.025 * SI - 0.5$ ---------- (maximum $R_f = 1.75$)

$R_f$ : Standard Damage Ratio (points/km)

SI : SI value (cm/sec)

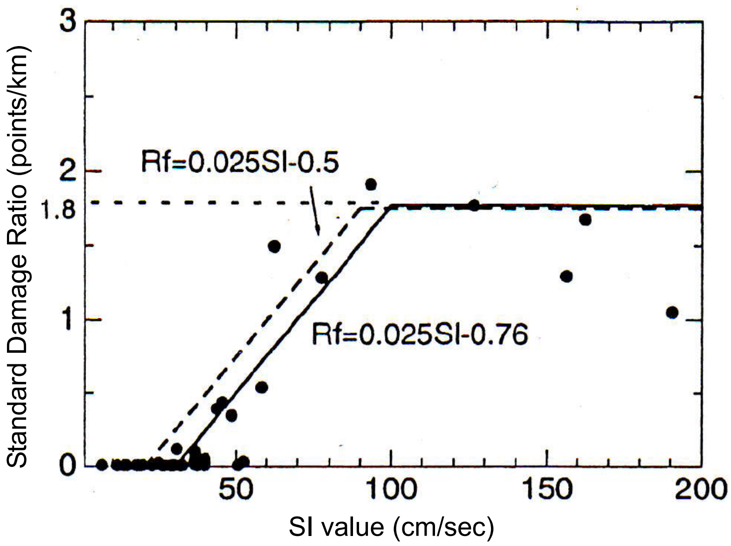



Figure 4-9 Relationship between SI Value and Damage Ratio of Steel Pipeline with Screw Joint in the Kobe Earthquake

(Reference: Gas Seismic Countermeasure Committee, 1996)

(2) Modification Coefficient for Ground Type with Liquefaction Potential

The modification coefficient for ground type with liquefaction potential is the same as the water supply pipeline (see the section 4.3.1) in many cases.

(3) Modification Coefficient for Pipeline Material and Pipeline Diameter

This coefficient is based on analysis of the past earthquake damage records. There are two ways to decide the coefficient applied by the municipalities: one way is by applying the product of the pipeline material coefficient and the pipeline diameter coefficient (see Table 4-26 to Table 4-27), another way is by applying the coefficients individually (see Table 4-28 to Table 4-29).

Table 4-26 Modification Coefficient for Pipeline Material and Pipeline Diameter applied in SHIZUOKA Prefecture

| Pipe Material | Coefficient of Pipe Material | Coefficient of Pipe Diameter | |
|---|---|---|---|
| Steel (screw) | 5.0 | < 100 mm | 1.3 |
| | | 100 mm ≤ | 0.75 |
| Steel ( welded) | 0.1 | | 1.0 |
| Cast Iron | 1.0 | < 400 mm | 1.5 |
| | | 400 – 1,000mm | 0.3 |
| | | 1,000 mm ≤ | 0.15 |
| Ductile Cast Iron | 0.25 | < 500 mm | 1.3 |
| | | 500 – 1,000mm | 0.3 |
| | | 1,000 mm ≤ | 0.15 |
| Asbestos Cement | 3.0 | < 100 mm | 2.3 |
| | | 125 – 250mm | 0.9 |
| | | 300 mm ≤ | 0.4 |
| PVC | 1.5 | < 100 mm | 1.1 |
| | | 100 mm ≤ | 0.9 |

Table 4-27 Modification Coefficient for Pipeline Material and Pipeline Diameter applied in MIE Prefecture

| | | Pipe Diameter | | | |
|---|---|---|---|---|---|
| | | 75 mm | 100 mm | 150 mm | 200 mm |
| Type of Pipe | Medium Pressure | 0.03 | 0.03 | 0.03 | 0.03 |
| | Main Low Pressure | 0.5 | 0.5 | 0.2 | 0.2 |
| | Branch Low Pressure | 1.0 | 1.0 | 1.0 | 1.0 |

Table 4-28 Modification Coefficient for Pipeline Material applied in Municipalities: Medium Pressure Gas

<table>
<tr><th>Pipe Material</th><th>MIYAGI</th><th>KANAGAWA</th><th>AKITA<br>SAITAMA<br>NAGANO<br>MIYAZAKI</th><th>FUKUI</th><th>NIIGATA</th><th>TOKYO 97<br>KAWASAKI</th><th>HIROSHIMA</th><th>FUKUOKA</th></tr>
<tr><td>Steel with Screw</td><td colspan="2">-</td><td>1.0</td><td>1.4</td><td>0.50</td><td>0.01</td><td>1.00</td><td>-</td></tr>
<tr><td>Welded Steel</td><td colspan="4">0.05</td><td>0.025</td><td>0.01</td><td>0.00</td><td>0.01</td></tr>
<tr><td>Steel with Mechanical</td><td colspan="2">-</td><td>0.1</td><td>0.125</td><td>0.05</td><td>0.01</td><td>-</td><td>0.01</td></tr>
<tr><td>SGM Steel</td><td colspan="5">-</td><td>0.01</td><td>0.055</td><td>-</td></tr>
<tr><td>Cast Iron with Mechanical</td><td>1.0</td><td colspan="3">0.1</td><td>0.05</td><td>0.02</td><td>0.029</td><td>0.13</td></tr>
<tr><td>Gas Type Cast Iron</td><td colspan="5">-</td><td>0.02</td><td>0.087</td><td>0.30</td></tr>
<tr><td>Water Type Cast Iron</td><td colspan="2">-</td><td colspan="2">0.5</td><td>0.25</td><td>0.02</td><td>-</td><td>0.30</td></tr>
<tr><td>Ductile Cast Iron</td><td colspan="2">-</td><td>0.1</td><td>-</td><td>0.05</td><td>0.02</td><td colspan="2">-</td></tr>
<tr><td>Cast iron with Faucet Joint</td><td colspan="5">-</td><td>0.02</td><td>0.391</td><td>-</td></tr>
<tr><td>Polyethylene</td><td colspan="2">-</td><td>0.05</td><td>0.1</td><td>0.00</td><td colspan="3">-</td></tr>
<tr><td>PVC</td><td colspan="2">-</td><td>0.75</td><td>-</td><td>0.375</td><td colspan="3">-</td></tr>
<tr><td>Asbestos Cement</td><td colspan="2">-</td><td>2.0</td><td>-</td><td>1.00</td><td colspan="3">-</td></tr>
<tr><td>Unknown, Other</td><td>-</td><td>0.1</td><td>0.5</td><td>-</td><td>0.05</td><td colspan="3">-</td></tr>
</table>

Table 4-29 Modification Coefficient for Pipeline Material applied in Municipalities: Low Pressure Gas

<table>
<tr><th>Pipe Material</th><th>MIYAGI</th><th>KANAGAWA</th><th>AKITA<br>SAITAMA<br>NAGANO<br>MIYAZAKI</th><th>SENDAI (City)</th><th>FUKUI</th></tr>
<tr><td>Welded Steel</td><td colspan="4">0.1</td><td>0.2</td></tr>
<tr><td>Steel with Mechanical</td><td colspan="2">0.15</td><td colspan="2">0.2</td><td>0.25</td></tr>
<tr><td>SGM Steel</td><td colspan="5">-</td></tr>
<tr><td>Steel with Screw</td><td colspan="3">2.0</td><td>1.0</td><td>2.8</td></tr>
<tr><td>Cast Iron with Mechanical</td><td colspan="3">0.2</td><td>-</td><td>0.3</td></tr>
<tr><td>Water Type Cast Iron</td><td colspan="3">1.0</td><td>-</td><td>1.0</td></tr>
<tr><td>Ductile Cast Iron</td><td colspan="2">-</td><td colspan="2">0.2</td><td>-</td></tr>
<tr><td>Polyethylene</td><td colspan="3">0.1</td><td>0.01</td><td>0.2</td></tr>
<tr><td>PVC</td><td>-</td><td colspan="2">1.5</td><td colspan="2">-</td></tr>
<tr><td>Asbestos Cement</td><td colspan="2">-</td><td>4.0</td><td colspan="2">-</td></tr>
<tr><td>Gas Type Cast Iron</td><td colspan="5">-</td></tr>
<tr><td>Cast iron with Faucet Joint</td><td colspan="3">-</td><td>0.6</td><td>-</td></tr>
<tr><td>Unknown, Other</td><td>-</td><td>0.2 / 1.5 *</td><td>1.0</td><td colspan="2">-</td></tr>
<tr><th>Pipe Material</th><th>AOMORI</th><th>FUKUOKA</th><th>TOKYO 97<br>KAWASAKI<br>(City)</th><th>NIIGATA</th><th>HIROSHIMA</th></tr>
<tr><td>Welded Steel</td><td>0.1</td><td>0.12</td><td>0.02</td><td>0.05</td><td>0.00</td></tr>
<tr><td>Steel with Mechanical</td><td>-</td><td>0.07</td><td>0.02</td><td>0.10</td><td>-</td></tr>
<tr><td>SGM Steel</td><td colspan="4">-</td><td>0.055</td></tr>
<tr><td>Steel with Screw</td><td>0.5</td><td colspan="4">1.0</td></tr>
<tr><td>Cast Iron with Mechanical</td><td>-</td><td>0.33</td><td>-</td><td>0.10</td><td>0.029</td></tr>
<tr><td>Water Type Cast Iron</td><td>1.0</td><td>0.74</td><td>-</td><td>0.50</td><td>-</td></tr>
<tr><td>Ductile Cast Iron</td><td>0.2</td><td>-</td><td>0.05 / 0.02 **</td><td>0.10</td><td>-</td></tr>
<tr><td>Polyethylene</td><td>-</td><td>0.12</td><td colspan="2">0.00</td><td>-</td></tr>
<tr><td>PVC</td><td>1.5</td><td>3.2</td><td>0.70</td><td>0.75</td><td>-</td></tr>
<tr><td>Asbestos Cement</td><td>4.0</td><td>1.0</td><td>-</td><td>2.00</td><td>-</td></tr>
<tr><td>Gas Type Cast Iron</td><td>-</td><td>0.50</td><td>0.23</td><td>-</td><td>0.087</td></tr>
<tr><td>Cast iron with Faucet Joint</td><td colspan="2">-</td><td>0.46</td><td>-</td><td>0.391</td></tr>
<tr><td>Unknown, Other</td><td colspan="3">-</td><td>1.00</td><td>-</td></tr>
</table>

Remarks: *: Main low pressure gas pipeline = 0.2, branch low pressure gas pipeline = 1.5
**: Gas type ductile cast iron = 0.05, cast iron with Mechanical = 0.02

**[Commentary]**

The low pressure gas pipeline is included in the objective facility for the damage estimation of the gas supply system. However, in this Study the damage estimation of low pressure gas pipeline was not included, because SONELGAZ is replacing the low pressure gas pipelines with medium pressure pipelines in the Wilaya of Algiers.

The regulator and related low pressure facilities are installed beside houses / buildings; hence, the damage to them is estimated as being comparable to the building damage.

The followings are points of concern for the damage estimation.

(1) Standard Damage Ratio

There are three ways, as mentioned above, to calculate the standard damage ratio.

As for the water supply pipelines, according to a recent study, damages resulting from earthquakes have better correlation with the peak ground velocity than the peak ground acceleration. It is also reported that the SI value has a very high correlation with the amount of damage. It is expected that the standard damage ratio as calculated using the peak ground velocity or the SI value will be applied with increasing frequency in the near future.

Further, the standard damage ratio as determined using the peak ground acceleration has been applied in many municipalities in Japan and it is easier to compare / discuss the adequacies in the present situation.

In this Study, we applied the standard damage ratio calculated using the peak ground acceleration, because the distribution of the peak ground acceleration was available from the hazard maps.

(2) Modification Coefficient for Ground Type with Liquefaction Potential

The modification coefficient for ground type with liquefaction potential was applied the same value for the water supply pipelines.

(3) Modification Coefficient for Pipeline Material and Pipeline Diameter

In this Study, a moderate value from among the above mentioned values was used as it was not possible to obtain enough information to determine the value more accurately as was done for the water supply pipelines.

### 4-3-5 Telecommunications

A telecommunication system consists of the following facilities.

(1) Key station: To connect / switch telecommunications (a telephone exchange office, a telephone exchange relay center, a base transceiver station, optic fiber cables, etc)

(2) Network cables and their supports: To connect between a key station and a subscriber such as household, office and so on (electric poles, aerial cables, underground cables, etc.)

Among the above mentioned facilities, the network cables and the electric poles have been reported as suffering a great deal of damage due to earthquakes while the key stations are generally constructed in conformity with an aseismic code and, therefore, suffer less damage. Hence, a damage estimation of the network cables and the electric poles was conducted for the damage evaluation of the telecommunication system as was done for the electric power supply system.

The method of damage estimation for the telecommunication cables and the poles is generally the same as for the electric power supply cables and poles (see the section 4.3.3).

**[Commentary]**

Recently, the rapid spread of mobile phones and the internet is a cause of a different kind of seismic damage (collapse of mobile phone antennas, etc) than in the past. Development of the information and communication fields is helpful for the rescue and relief efforts during an earthquake, while damage to them may cause further confusion due to telephone congestion, etc. As stated above, improvement of the aseismic performance of the telecommunication system has been proceeding; however, modification/development of the damage estimation method is required.

# 5. Damage Calculation

The calculation of damage using a damage function, which is explained in Chapter 4, requires distribution of the seismic motion used in the damage function, namely, acceleration and/or seismic intensity as well as distribution of buildings, infrastructures, and lifelines (water, sewerage, electricity). For example, in Algiers, buildings were classified into eight types based on the structure and building code, therefore the number of buildings in each 250 m grid by eight building classes is necessary.

The damage in each 250 m grid is calculated by multiplying the damage function by the inventory. The result may be added up in administrative units to make it easy to understand.