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EARTHQUAKE RESISTANT DESIGN OF NUCLEAR FACILITIES WITH LIMITED RADIOACTIVE INVENTORY

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FOREWORD

The objective of this Technical Document is to present some guidelines for nuclear facilities with limited radioactive inventory (facilities which contain an amount of radioactive material significantly less than that of nuclear power plants e.g. research reactors up to a few MW for which this amount is about 1/1000) for the purpose of ensuring their seismic safety. The adopted approach is simplified and conservative with the emphasis on appropriate construction and detailing principles rather than sophisticated dynamic analysis.

The seismic design of nuclear power plants has been sufficiently addressed in the IAEA Safety Guides of the Nuclear Safety Standards (NUSS) programme.

The seismic design of nuclear facilities with limited radioactive inventory has either been left to the provisions made in national building codes and other industrial codes (not specifically intended for nuclear facilities), or it has been the subject of complex analysis of the type usually performed for nuclear power plants.

In the adopted approach there is generally sufficient conservatism to compensate for the expected discrepancy with the real behaviours of the structures in case of an earthquake. However, in some special cases an overall dynamic analysis may be required to give the more realistic behaviour of certain structures.

The document has been prepared by a group of consultants from different countries having experience in the assessment of seismic hazard, seismic design of buildings, equipment and distribution systems as well as radiological consequences involved under accident conditions for the subject facilities.

ABSTRACT

This document comprises the essential elements of an earthquake resistant design code for nuclear facilities with limited radioactive inventory. The purpose of the document is the enhancement of seismic safety for such facilities without the necessity to resort to complicated and sophisticated methodologies which are often associated with and borrowed from nuclear power plant analysis and design.

The first two sections are concerned with the type of facility for which the document is applicable and the radiological consideration for accident conditions. The principles of facility classification and item categorization as a function of the potential radiological consequences of failure are given in section 3. The design basis ground motion is evaluated in sections 4 - 6 using a simplified but conservative approach which also includes considerations for the underlying soil characteristics. Sections 7 and 8 specify the principles of seismic design of building structures and equipment using two methods, called the equivalent static and simplified dynamic approach. Considerations for the detailing of equipment and piping and those other than for lateral load calculations, such as sloshing effects, are given in the subsequent sections. Several appendices are given for illustration of the principles presented in the text. Finally, a design tree diagram is included to facilitate the user's task of making the appropriate selections.

CONTENTS

1. Introduction

Methods and procedures described in this document are considered to be applicable to the following facilities:

- research reactors of up to a few MW(th) continuous power
- hot cells with a comparable inventory of radioactive materials
- uranium fuel fabricating and purification plants
- repositories for spent fuel with at least three months of decay time.

Uranium mills and mill tailings deposits are considered outside the scope of this document, however, it is recognized that mill tailings deposits might need some kind of stability analysis and demonstration similar to those suggested herein.

The basic philosophy in presenting the simplified methods contained in this document is to minimize sophisticated calculations and emphasize the important construction and detailing principles which generally govern overall seismic safety. However for specific structural arrangements such that the simplified approach may not be valid and in cases where a large safety margin is required for nuclear safety reasons a more complete and refined approach, as those discussed in IAEA Safety Guides 50-SG-S1, Earthquakes and Associated Topics in Relation to Nuclear Power Plant Siting, 50-SG-S2, Seismic Analysis and Testing of Nuclear Power Plants and 50-SG-S8, Safety Aspects of the Foundations of Nuclear Power Plants, may be preferred.

It is intended here that the document be used as a guideline in the siting and design of new facilities. Retrofitting of older facilities designed using other criteria is not considered to be within the scope of this document.

Aside from prescribing procedures to calculate seismic loads on the structures and equipment, recommended practice for detailing of components and piping in terms of anchorage, joints and connections is indicated. A separate section on sloshing effects is included to be considered in the analysis of facilities having pools or tanks of large dimensions.

The nuclear facilities with limited radioactive inventory are classified and their structures and components are further categorized to take into account the consequences of failure. This is achieved in terms of the sophistication of the adopted methodology and the conservatism incorporated therein.

It is essential that good practices be observed to achieve adequate quality during all phases of the work, including site selection, design of the facility (including earthquake resistant design), material and equipment and their specifications, fabrication, construction, installation, commissioning and testing. In particular, design and construction control procedures should be implemented, with emphasis on independent evaluation of the adequacy of the work as it proceeds.

2. Radiological Considerations

The primary objective of earthquake resistant design of a nuclear facility is to prevent such damage to structures and equipment that could lead to significant exposures to plant personnel or members of the public. In the case of facilities with a limited radioactive inventory, the need for earthquake resistant design depends on the probability of seismic events and on the consequences of such events if no earthquake resistant design features were applied.

The radiological criteria for deciding the need for earthquake resistant design for events having a low probability should include:

(i) prevention of non-stochastic radiation effects to individuals from the event occurring, and

(ii) ensuring that the collective effective dose equivalent commitment to the exposed population from the event is kept at a sufficiently low level.

According to the Basic Safety Standards for Radiation Protection, IAEA Safety Series No. 9, non-stochastic effects to individuals are prevented if the annual dose equivalent to any organ does not exceed 0.05 Sv. Acceptability of collective dose may be considered on the basis of risk values given by the International Commission on Radiological Protection; the risk factor for severe radiation-induced detrimental effects due to irradiation has been estimated as 10^{-2} per Sv of effective dose equivalent. If the aforementioned organ dose limits are respected and collective effective dose equivalent commitment, from the event, is kept to a few man.Sv, significant off-site emergency action should not be necessary and it can be concluded that additional effort will not be required.

Moreover, additional effort will not be required for the earthquake resistant design* of a nuclear facility with limited radioactive inventory until it can be shown that:

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* This means that using conventional earthquake resistant building and other industrial codes is not sufficient.

(1) during the lifetime of the facility, seismic events exceeding the design values are not likely to occur (probability of occurrence less than 10^{-2} per annum), and;

(2) should such an event occur, the radiological consequences in the worst hypothetical case:

(a) would not result in exposure of any individual of the public exceeding:

- a committed dose equivalent to individual organs of 0.05 Sv;
- a committed effective dose equivalent of 0.1 Sv; and

(b) in the case of any group of individuals of the public, would not result in exceeding a collective effective dose equivalent commitment of a few man.Sv.

In the design of the facility, the radiological design objectives should be based on the system of dose limitations recommended in IAEA Safety Series No. 9 including the requirement of "keeping radiation exposures as low as reasonably achievable".

For all facilities within the scope of this document a safety report should be prepared justifying the design assumptions used and, in particular, the consequences of an earthquake in terms of possible damage to the facility and its radiological consequences. The reader is referred to Section 3 of the IAEA Code of Practice, "Safe Operation of Research Reactors and Critical Assemblies", Safety Series No. 35. The classification presented in Section 3 of this document, in terms of earthquake resistant design, should also be justified in the safety report.

3. Classification of the Facilities for Earthquake Resistant Design

Nuclear facilities with limited radioactive invnetory are classified as:

Class C - Facilities which have to be designed in accordance to the aseismic normal building and other industrial codes.

Class B - Facilities for which the following should be ensured. Civil structures will not collapse, the pool and other containment structures will preserve normal leak tightness and no disruption of the core or fuel resulting from falling debris (if this can result in criticality or melting of the fuel).

Class A - Facilities for which additional safety functions have to be ensured during and after an earthquake.

As criteria for deciding in which class the facility is to be included the following may be said:

- The facility may be included in Class C when the conditions of Section 2 are not violated even in case of collapse of buildings, loss of normal leak tightness of the pool or other containment structures and large disruption of the core or fuel.

- The facility will be included in Class B when the conditions of Section 2 are not violated, only if the building will not collapse, the pool or other containment structures will not lose normal leak tightness and no big debrie will fall on the fuel or the core.

- The facility will be included in Class A when the conditions of Section 2 are not violated, only if the building will not collapse, the pool or other containment structures will not lose normal leak tightness, no big debris will fall on the core and some additional safety functions (functioning of components or piece of equipment) have to be ensured during and after the earthquake.

For facilities of Class A or B, a further categorization of items is made as follows:

Category I: items whose functioning must be maintained in the event of the Design Basis Earthquake

Category II: items whose loss of function may be permitted but should be designed against collapse in the event of the Design Basis Earthquake

Non-Categorized: all other items

If the impairment of Category II or non-Categorized items results in the loss of function of Category I or Category II items respectively, these items should either be categorized in category I and category II respectively or the potential of damage should be appropriately evaluated and reduced by change of design or location.

4. Evaluation of Design Basis Earthquake

The design basis earthquake is evaluated on the basis of the maximum historical intensity. For this evaluation, the following procedure is applied:

- A zone having a radius of 100 to 200 kilometers from the site*1 (the higher value is applicable to areas with low seismicity) is selected.

- Using available publications and catalogues the maximum observed intensity in this area is established. The information should cover as much historical data as possible, in any case it should extend to at least 100 years.

- The design intensity level is selected using Table 4.1.

Table 4.1

Range of Maximum Historical Intensity, I_{max} *2	Design Intensity Level
VII < I_{max} ≤ VIII	1
VIII < I_{max} ≤ IX	2
IX < I_{max}	3

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*1 If the site is in a relatively quiet seismic zone and the historical seismicity is mainly due to neighbouring active zones, the area for which historical seismicity data is collected could be appropriately adjusted by considering the shape and dimensions of the particular seismotectonic setting.

*2 Modified Mercalli intensity scale.

5. Geotechnical Investigations

Geological and geotechnical investigations at the site are performed with the following objectives:

(a) the assessment and alleviation of possible geological/ geotechnical problems involving surface rupture due to faulting, liquefaction, collapse and slope instability.

(b) the evaluation of the soil characteristics so that a reasonable soil categorization can be achieved (see Section 6, Table 6.2).

(c) the evaluation of geotechnical parameters to be used in the design of the foundation.

The amount of geotechnical investigations to be performed should be based on the extend of potential problems, the available data and the type and size of the facility. For a more comprehensive treatment of the subject, the reader is referred to IAEA Safety Series No. 50-SG-S1, 50-SG-S2 and 50-SG-S8.

Geotechnical investigations should be carried out to evaluate the bearing capacity and other soil parameters for foundation and building design. These will primarily involve borehole drillings in sufficient number and to sufficient depth, depending on the soil conditions. For competent rock sites where the rock formation continues to sufficient depth drilling may not be necessary.

It is recommended that the soil profile is physically identified (i.e. through drilling) to a depth equal to at least one half of the maximum foundation dimension. The depth to firm bearing strata* should also be determined using geological inference, boreholes or geophysical methods.

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* For facilities within the scope of this document firm bearing strata is defined as layers for which SPT count is at least 30 for a minimum of 5 meters continuously. SPT count 30 is approximately equivalent to ultimate shear strength of 2 kg/cm^2 for clay or $\phi = 40^o$ for sand (ϕ is the angle of internal friction).

In parallel with the foundation investigations and the use of available geological/geotechnical data, studies should be performed at the site to assess possible hazards resulting in permanent soil deformations (including surface rupture, liquefaction, collapse, slope instability).

If these investigations indicate particular problems (taking into consideration the load resulting from the design basis earthquake) further studies should be conducted. In this context, simplified methodologies for the assessment of liquefaction potential is given in Appendix A.

The walls of Category I structures should not be in direct contact with the soil mass because the evaluation of dynamic soil pressures to embedded structures can be very complex and may require a substantial effort. The walls of Category I structures should be separated from the soil by Category II structures. If this is not practical, then dynamic soil data, which are outside the scope of this document, are required, in particular for Category I buried piping. For applicable methods the reader is referred to IAEA-50-SG-S2.

6. Design Basis Ground Motion

The design acceleration values corresponding to the design intensity levels of Section 4, are given in Table 6.1

Table 6.1

Design Intensity Level	Design Acceleration for Firm Bearing Strata a_b , in g's
1	0.08
2	0.15
3	0.30

These acceleration values are applicable at the firm bearing strata. The amplification effect of geological formations overlying the firm bearing strata is considered in terms of the coefficient γ as follows:

$$a_g = \gamma a_b \qquad (6.1)$$

where a_g is the design ground acceleration and the value of γ is given in terms of the soil categorization indicated in Table 6.2.

Table 6.2

Soil Categorization	Categ. 1	Categ. 2	Categ. 3
Description	Firm Bearing Strata	Other soil than those defined in Categories 1 and 3	Fill ground or Alluvium ground which is thicker than 25 m
γ	1.0	1.25	1.50

A more detailed evaluation may be performed to establish the design ground acceleration. However the method should be reviewed and approved by the regulatory body.

7. Building Design

7.1 General Considerations

The earthquake resistant design of buildings should be performed according to the classification of Section 3.

Class A : Simplified Dynamic Approach should be used.

Class B : Equivalent Static Approach may be used.

The storey lateral load, F_i is given by:

$$F_i = a_g \cdot C_i \cdot W_i \tag{7.1}$$

where,

a_g = design acceleration applicable at the ground surface level and defined by Eq. (6.1)

$C_i = C_{si}$ or C_{Di}: coefficient related to the building characteristics and defined by Eqs (7.3) and (7.4) for the two approaches respectively

W_i = the total weight of the building at i-th floor to be calculated by,

$$W_i = G_i + n\, P_i \tag{7.2}$$

where, G_i and P_i are dead and live loads for the i-th floor and n is given in Table 7.1.

Table 7.1

Use of Floor	Live Load Coefficient n
Heavy weight components such as large baggages in the wide space	0.50
Medium weight components in medium size space.	0.30
Light weight components such as desks, books and lockers in small size space.	0.25

Note:
The weight of the water in the reactor pool should be considered as dead load. The weight of permanently installed equipment and distribution systems at maximum normal operating weight should also be considered as dead load.

7.2 Equivalent Static Approach

The seismic coefficient C_{si} (the additional subscript s is introduced to denote "static" approach) of Equation 7.1 is defined as follows:

$$C_{si} = \frac{S_i}{\mu} \qquad (7.3)$$

where,

S_i = the coefficient giving the distribution of the seismic coefficient throughout the height of the building and defined in Table 7.2.

μ = the ductility coefficient defined in Table 7.3. μ is taken as 1.0 for Category I structures (see Section 3).

Table 7.2

Building Level	S_i*
Embedded Part	0.75
Ground Level	1.00
Centre of Gravity Level	1.50
Top of Building	2.25

Note:
Linear interpolation may be used to determine S_i for intermediate elevations.

Table 7.3

Type of Structural System	Vertical Seismic Resisting System	Coefficient μ
BEARING WALL SYSTEM: A structural system bearing walls providing support for all, or major portions of the vertical loads.	Light framed walls with shear panels	3.2
Seismic force resistance is provided by shear walls or braced frames.	Shear walls	
	Reinforced concrete	2.2
	Reinforced masonry	1.7
	Braced frames	2.0
	Unreinforced and partially reinforced masonry shear walls	1.0

* Regardless of the location of the centre of gravity, the embedded part should have an S_i value ≥ 0.75.

Table 7.3 continued

BUILDING FRAME SYSTEM: A structural system with an essentially complete Space Frame providing support for vertical loads.	Light framed walls with Shear panels	3.5
Seismic force resistance is provided by shear walls or braced frames	Shear walls	
	Reinforced concrete	2.7
	Reinforced masonry	2.2
	Braced frames	2.5
	Unreinforced and partially reinforced masonry shear walls	1.0
MOMENT RESISTING FRAME SYSTEM: A structural system with an essentially complete Space frame providing support for vertical loads.		
Seismic force resistance is provided by Moment Frames capable of resisting the total prescribed forces	Moment frames	
	Steel	2.2
	Reinforced concrete	1.0
DUAL SYSTEM: A structural system with an essentially complete Space Frame providing support for vertical loads.	Shear walls	
	Reinforced concrete	4.0
	Reinforced masonry	3.0
A Special Moment Frame shall be provided which shall be capable of resisting at least 25 percent of the prescribed seismic forces. The total seismic force resistance is provided by the combination of the Special Moment Frame and shear walls or braced frames in proportion to their relative rigidities.	Wood sheather shear panels	4.0
	Braced frames	3.0
INVERTED PENDULUM STRUCTURES*: Structures where the framing resisting the total prescribed seismic forces acts essentially as isolated cantilevers and provides support for vertical load.	Moment Frames	
	Structural steel	1.0

* Inverted pendulum structures cannot be used as Category I Structures. A somewhat increased value of μ may be used for reinforced concrete or steel moment frame structures if special provisions are taken for ductility (see for example Applied Technology council ATC 3-06).

7.3 Simplified Dynamic Approach

The seismic coefficient C_{Di} (the additional subscript D is introduced to denote "dynamic" approach) of Eq. 7.1 is defined as:

$$C_{Di} = (D_1 \cdot D_2 \cdot D_3)/\mu_d \qquad (7.4)$$

For $T_b \leq 0.1$ s; $\mu_d = 1$

$0.1s < T_b \leq 0.5$ s; $\mu_d = (2\mu - 1)^{1/2}$

$0.5s < T_b$; $\mu_d = \mu$

where μ is defined in Section 7.2, and

D_1: Standard Response Factor to be determined from Fig. 7.1

D_2: Coefficient related to damping to be determined from Tables (7.4) and (7.5).

The load distribution throughout the height of the building is given by D_3 and is defined as,

$$D_3 = 1 + 0.5\ (h_x/H_b) \qquad (7.5)$$

where,

h_x*: height of the point or storey under consideration with respect to the ground level

H_b : height of the building

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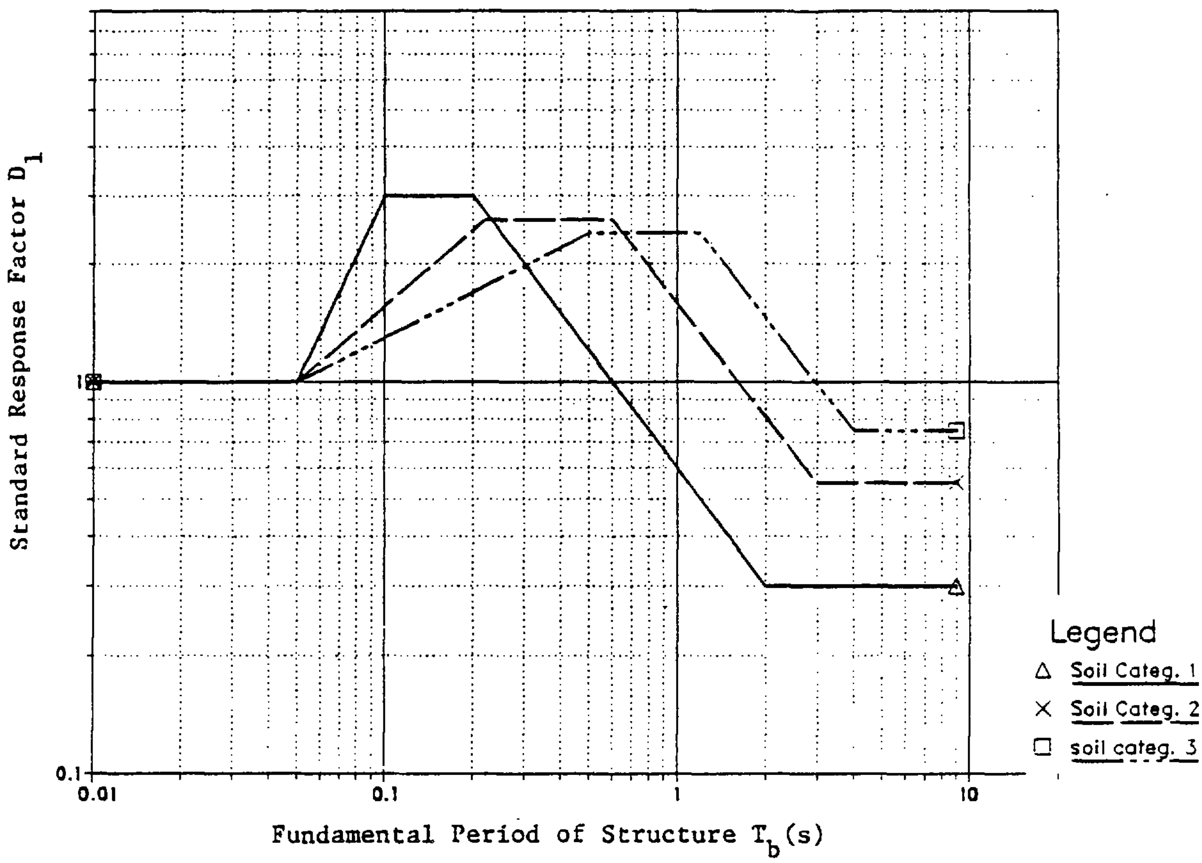
* h_x can be negative for below grade locations, however $D_3 \geq 0.75$ should be maintained.

Design Intensity Levels 1 and 2



Design Intensity Level 3

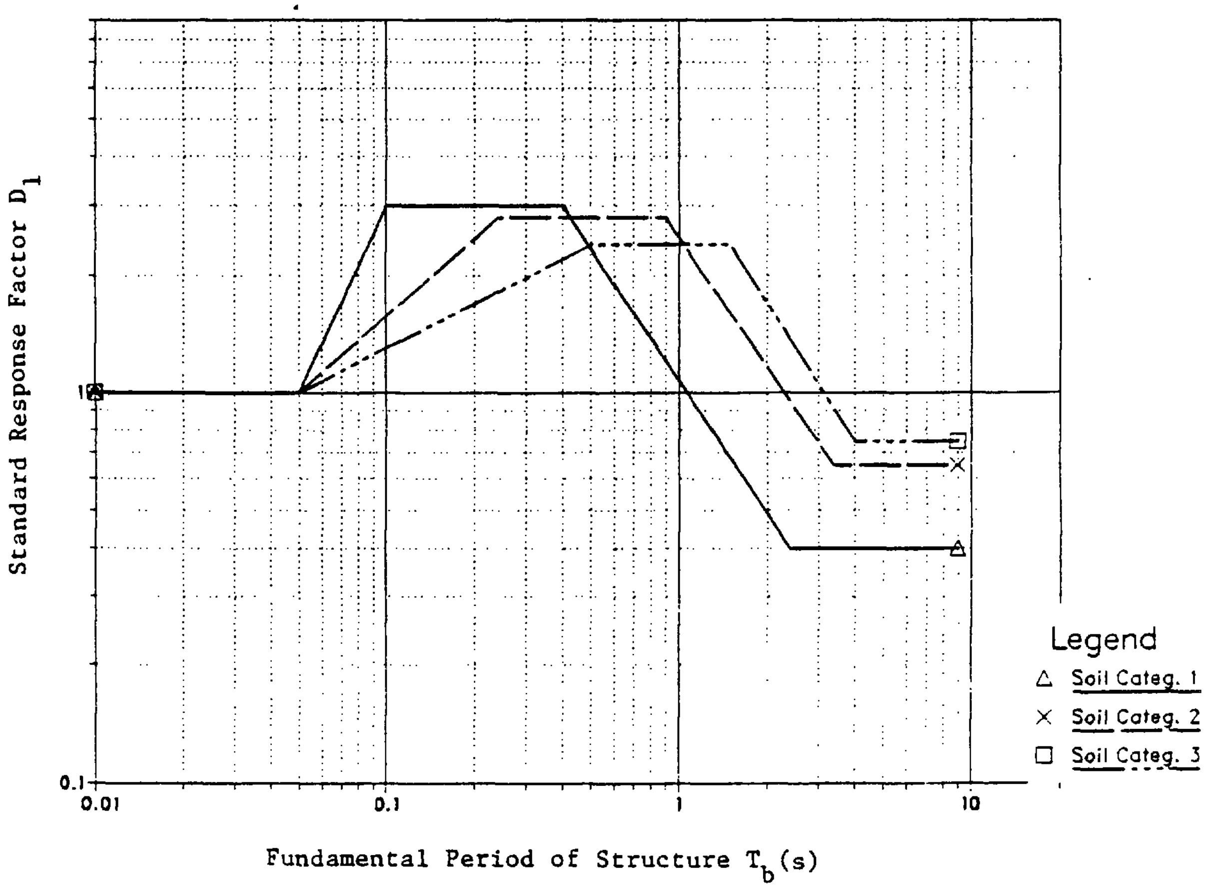


FIG. 7.1 RESPONSE CURVES FOR DIFFERENT INTENSITIES AND SOIL CATEGORIES

Table 7.4

Type of Structure	Percentage of critical damping β (%)
steel structure with braces, and bolted or riveted steel structure	5
welded steel structure	3
reinforced concrete or steel concrete structure	5
reinforced concrete or steel concrete structure with shear wall	10

Note:
These values may be increased by 2% and 5% for Category 2 and Category 3 soils (table 6.2) respectively, in order to take into account the additional damping provided by soil.

Table 7.5

Damping coefficient β (%)	0.5	1.0	2.0	3.0	5.0	7.0	8.0	10.0	12	13	15
D_2	1.73	1.55	1.32	1.19	1.00	0.88	0.84	0.78	0.74	0.72	0.70

In order to use Figure 7.1 to determine the Standard Response Factor, the fundamental period of the structure, T_b, is required. For rectangular shape structures, T_b can be calculated using,

$$T_b = \alpha H_B (D_B)^{-1/2} \tag{7.6}$$

where H_B : height of the building, (m)

D_B : dimension of the building in a direction parallel to the applied lateral loads, (m)

α = 0.10 for rigid Reinforced Concrete structures, and equal 0.12 for ordinary Reinforced Concrete structures

If the fundamental period of the structure, T_b, is shorter than 0.3 s, the product D_1 D_2 should not be less than 1.5.

8. Equipment Design

For equipment design, dynamic approach is necessary. The seismic coefficient C_{Di} from the simplified dynamic approach Eq. (7.4) is used to evaluate the horizontal and vertical seismic loads on the equipment, F_{Eh} and F_{Ev} respectively, at the height of the equipment, h_x.

$$F_{Eh} = C_{Di} \cdot D_4 \cdot W_e \tag{8.1a}$$

$$F_{EV} = 0.5\, F_{Eh} \tag{8.1b}$$

where,

W_e = weight of equipment

Each horizontal component F_{Eh} should be taken separately along the principal horizontal axes of the equipment and combined with the vertical component F_{Ev} taken in the most conservative of the two possible directions.

D_4 = Amplification factor due to floor response spectrum as shown in Figure 8.1 and $D \geq 1.0$.

$$\text{with } A_f = [(\beta_b' + \beta_e')^2 + m_r]^{-1/2} \tag{8.2}$$

$\beta_b' = \beta_b \mu_b$ and $\beta_e' = \beta_e \mu_e$ for $T_b \geq 0.5$ s

$\beta_b' = \beta_b(2\mu_b - 1)^{1/2}$ and $\beta_e' = \beta_e(2\mu_e - 1)^{1/2}$ for $0.5 > T_b > 0.1$ s

$\beta_b' = \beta_b$ and $\beta_e' = \beta_e$ for $T_b \leq 0.1$ s

β_b = damping for building as defined in Table 7.4

β_e = damping for equipment

m_r = mass ratio m_e/m_b

m_e = mass of equipment

m_b = effective mass of building interacting with the equipment (in most cases m_b may be reasonably assumed to be the mass of the building at the equipment level).

μ_b = ductility coefficient defined for the building as defined in Table 7.3

μ_e = ductility coefficient defined for the equipment with μ_e = 3.0 for distribution systems and 2.0 for equipment. Larger values may be used based on experimental verification.

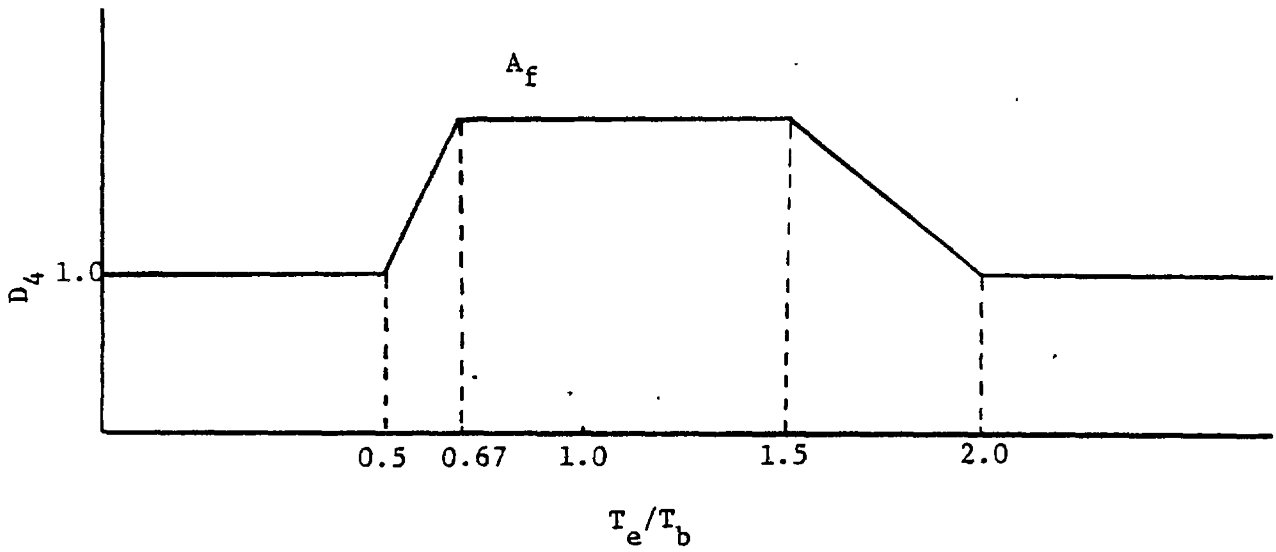


Figure 8.1

T_e : Fundamental period of equipment

T_b : Fundamental period of building

In cases where T_e is not evaluated, T_e should be assumed to be equal to T_b.

9. Detailing Considerations for Equipment and Distribution Systems

9.1 General Principles

Most failures related to equipment and distribution systems occur at their connections to the supporting structure. These failures are at least as dependent on the displacement demand imposed by differential support movements, (seismic anchor motions) as they are on inertia forces in the equipment. Therefore, the anchoring devices for equipment, components and piping of Class A facilities should be designed for maximum ductility as well as to resist the horizontal and vertical seismic loads induced by the structural response. The mounting of the anchoring devices is as important for seismic safety as their design.

It should be understood that positive anchorage in the form of anchor bolts should be used to carry uplift due to overturning effects for all equipment. Anchor bolts in the absence of engineered shear keys should be designed to carry applicable shear, except where the shear friction capacity between the component and its foundation can be shown to carry applicable lateral loads with a safety factor of at least 2.0.

For equipment, components and piping of Class B facilities positive anchorage and earthquake resistant ductile detailing are recommended even if not required.

9.2 Capacity of Anchor Bolts

Capacities of anchor bolts of various type and size and under different loading and geometric conditions are typically given in National Codes and manufacturer installation specification. Additional guidance is given in Appendix B.

9.3 Computation of Tensile Force on Anchoring Device

The tensile (pulling) forces which develop in the anchor bolts due to uplift depend on the configuration of the anchored equipment, base flexibility and the bolt pattern. Guidance for their determination are typically given in national codes. Additional guidance is provided in Appendix C.

9.4 Distribution Systems Including Piping, Raceways and Ducts

In principle, these structures should be supported to avoid excess response to earthquake motion. This response is determined by both inertia force and differential support (seismic anchor) motions. The piping span for dead weight should not be longer than the length shown in Table D.1. in Appendix D according to its diameter. Recommendation for seismic support spacings, for distribution systems, are given in Appendix D.

Branch connections, connections between components as well as that of component to wall, ceiling and/or floor should be flexible. Some details are shown in the Figures in Appendix E. If a component is supported by shock isolation mounts, the vibratory motion (e.g. rocking and translation) of the component should be taken into account in the design of the shock mounts (e.g. to avoid components overturning or jumping off its mounts). In addition, all connections made to such a component must be flexible enough to accommodate relative movement without excessive force or stress to such connections.

It is recommended that piping be designed such that possible leakages may be detected after an earthquake.

9.5 Overhead Cranes, Bridges and Platforms

Overhead cranes, bridges, platforms, etc., located above critical components should be designed to resist the seismic force induced by the acceleration specified for the earthquake resistant design of Category I building structure, even if its operation is not essential.

9.6 Other Overhead Items

Provisions should be taken to preclude danger due to breaking of plaster, window glass, lighting fixtures, and other brittle components above the critical components. (See Table B.1 Design Provision in Appendix B).

10. Increase in Allowable Stresses

National building codes or standards for materials such as concrete and steel generally specify an increase in allowable stresses for extreme loads such as earthquakes. These increased values may be used for the earthquake resistant design of both Category I and Category II structures.

If such an increase is not specified in the national code or standard, then an increase of 33% may be taken in the allowable stresses for concrete (compressive and shear), structural steel (tensile, compressive and shear) as well as foundation soil (bearing capacity). However, applied buckling loads should not exceed 0.75 the critical buckling load. Alternatively, if load factor and strength design procedures are used the overall load factor may be taken as 1.25.

High strength concrete and mild steel should be used as per national building codes or standards.

The maximum stress ranges in piping considering combined axial (membrane) and bending (primary plus secondary) due to earthquake and other applicable loads should not exceed 2.0 f_y (specified minimum yield stress) of the piping material at the temperature. Higher stresses are allowed if they are substantiated by a fatigue and ratcheting analysis.

11. Sloshing Effects

Sloshing may be produced in a pool or tanks by strong earthquakes. This phenomenon may generate waves in the pool which may interact very strongly with the bridge, from which the reactor and its control system are suspended and submerged structures near to the water surface. Sloshing can begin several tens of seconds after arrival of the first higher frequency seismic waves at the site.

In order to evaluate the height of the wave produced due to sloshing in open pools several parameters are calculated depending on the design intensity level, shape of the pool and the sloshing period.

Sloshing period T_s may be calculated using the following formula,

$$T_s = 2\pi\,[3.68\,(\zeta_1/D_p)\,.\,\tanh\,(3.68\,H_p^{1/2}/D_p)]^{-1}$$

(for circular pools)

$$T_s = 2\pi\,[3.16\,(\zeta_1/L_p)\,.\,\tanh\,(3.16\,H_p^{1/2}/L_p)]^{-1/2}$$

(for rectangular pools)

where H_p is the depth of the pool and D_p and L_p are the diameter and length of the circular and rectangular pools respectively, in meter.

The co-efficient ζ_1 is defined as a function of the design level intensity as shown in Table 11.1.

TABLE 11.1

Design Intensity Level	ζ_1
1	0.33
2	0.50
3	1.00

The value of ζ_1 may be adjusted taking into consideration the source mechanism and the characteristics of the propagation path of the seismic waves.

The design basis ground displacements, D_{OH} and velocity, V_{OH}, are calculated using,

$$D_{OH} = \zeta_1 \cdot 60 \quad (cm) \tag{11.2a}$$

$$V_{OH} = \zeta_1 \cdot 50 \quad (cm/s) \tag{11.2b}$$

The maximum displacement of the centre of gravity of the water (for non-shallow pools) is given by an amplification factor ζ_2 such that,

$$D_g = \zeta_2 \cdot D_{OH} \tag{11.3}$$

where,

D_g = maximum displacement of the centre of gravity of the water

ζ_2 = a coefficient equal to about 10 (unless calculated differently by a reasonable hydrodynamic method).

The applicable formula to calculate the maximum sloshing wave height, h_s (cm) is given in Table 11.2 for different period ranges and for pool or tank shapes.

Table 11.2

Applicable Formula to Calculate h_s

Sloshing Period / Pool Shape	$T_s > 7.5$ s	$T_s \leq 7.5$ s
Circular	$1.686.10^{-2}.\zeta_2 \dfrac{D_p}{T_s^2} \cdot D_{OH}$	$2.683.10^{-3}.\ \zeta_2 \dfrac{D_p}{T_s} \cdot V_{OH}$
Rectangular	$2.178.10^{-2}.\zeta_2 . \dfrac{L_p}{T_s^2} \cdot D_{OH}$	$3.466.10.^{-2}.\ \zeta_2 . \dfrac{L_p}{T_s} . V_{OH}$

D_p, L_p and h_s all in centimeters.

The phenomenon of sloshing may impose very large loads on bridges, roofs, platforms, etc., which can be of the order of 1000 Kg/m^2. This possibility should be provided for in the design.

Provision should be made for water overflowing pools (reactor or fuel storage) and potential loss of essential coolant or shielding. In addition, the risk of radioactivity escaping from the facility should be taken into account in the event of earthquake-induced sloshing. Consideration should be given to containing such overflow and returning it to the pool.

Sloshing in closed tanks can produce a strong vacuum behind the surface wave. Vacuum breakers should be considered in the tops of tanks to avoid collapse in the event of a large earthquake.

12. Other Considerations

12.1 Horizontal Torsional Moments

Provisions should be made for the increase in shear resulting from the horizontal torsion due to an eccentricity between the centre of mass and the centre of rigidity. Negative torsional moments should be neglected.

12.2 Overturning and Uplift

Structures should be designed to resist the overturning and uplift effects caused by the earthquake forces specified with due consideration of gravity forces and buoyancy. For the seismic inertia loads considered at sites with low or moderate seismicity, it is not anticipated that net uplift or overturning forces will occur.

In cases where there is significant eccentricity, additional torsional loads due to this eccentricity should be considered.

12.3 Seismic Scram System

For Class A reactors a seismic scram system should be provided. For other facilities consideration should be given to automatic actions to attain a safe state in case of an earthquake if risk and practicability evaluations require these. It is recommended that the trigger level is approximately at 0.025 g ground acceleration or slightly higher than this level, depending on environmental noise conditions.

12.4 Common Foundation Level

It is recommended that foundation elements for structures important to safety be located at the same elevation.

12.5 Precast Panels

Precast panels (or other elements) should be connected such that they behave as an integral unit during an earthquake.

12.6 Light Ceiling

Heavy ceilings should be avoided in order to preclude the occurrence of catastrophic failures of structures and equipments from collapse of the ceiling.

APPENDIX A - SIMPLIFIED PROCEDURES FOR ASSESSMENT OF LIQUEFACTION POTENTIAL

Saturated alluvial sandy layers which have the water table within 10 m from the ground surface, and have D_{50}-values on the grain size accumulation curve between 0.02 and 2.0 mm, are vulnerable to liquefaction for the depth between 0 and 20 m. The liquefaction potential of these layers can be estimated according to any of the two Methods outlined in this Appendix.

A.1 Estimation of Liquefaction (Method 1)

For soil layers which are judged to be vulnerable, liquefaction potential should be checked based on liquefaction resistance factor F_L defined by the following equations

$$F_L = R/L \tag{A.1}$$

where

F_L: liquefaction resistance factor

R : resistance of soil elements to dynamic loads, and

$$R = R_1 + R_2 \tag{A.2}$$

R_1 and R_2 should be determined in accordance with Figs A.1 and A.2 respectively

L: dynamic loads to soil elements induced by earthquake motion

$$L = r_d\, K_s \frac{\sigma_v}{\sigma'_v} \tag{A.3}$$

$$r_d = 1.0 - 0.015z \tag{A.4}$$

z : depth from the actual ground surface (m)

K_s: seismic coefficient for evaluation of liquefaction, and is taken as:

Design Intensity Level 1, $K_s = 0.13$
Design Intensity Level 2, $K_s = 0.15$
Design Intensity Level 3, $K_s = 0.17$

σ_v: total overburden pressure (kg/cm^2)
σ'_v: effective overburden pressure at the static condition (kg/cm^2)

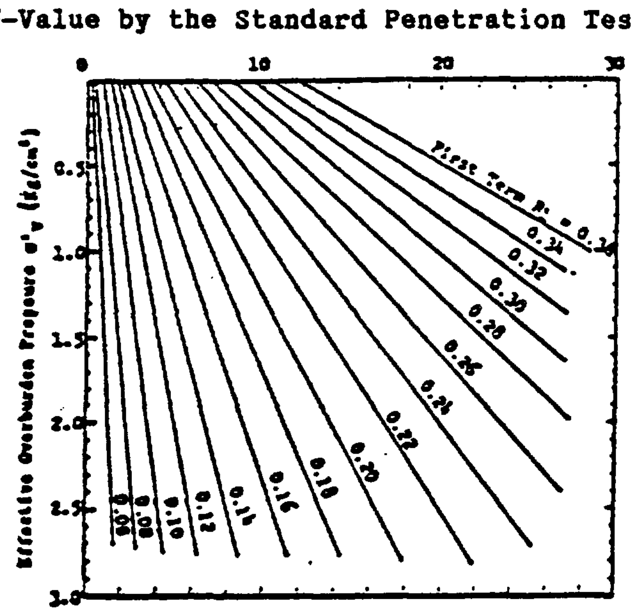


Fig. A.1 R_1 -Value (First Term of Resistance R).

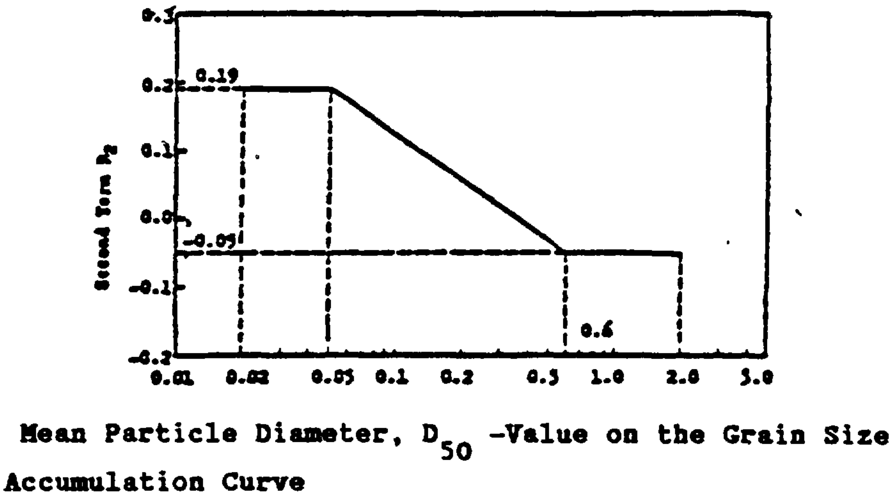


Fig. A.2 R_2-Value (Second Term of Resistance R).

Soil layers having liquefaction resistance factor F_L smaller than 1.0 should be judged to liquefy during earthquakes. Figs. A.1 and A.2 are graphic illustrations of the first term R_1 and the second term R_2 represented in the following equations which were proposed based upon the results of laboratory dynamic triaxial tests on soil specimens taken from several sites in Japan.

$$R = 0.0882 \left[\frac{N}{\sigma'_v + 0.7}\right]^{1/2} + 0.19 \qquad (0.02\ mm \leq D_{50} \leq 0.6mm)$$

$$R = 0.0882 \left[\frac{N}{\sigma'_v + 0.7}\right]^{1/2} + 0.225 \log \left(\frac{0.35}{D_{50}}\right) \qquad (0.05\ mm < D_{50} \leq 0.6mm)$$

$$R = 0.0882 \left[\frac{N}{\sigma'_v + 0.7}\right]^{1/2} - 0.05 \qquad (0.6\ mm < D_{50} \leq 2.0mm)$$

N: value obtained from the standard penetration test.

A modified value, Na, can be obtained from equation (A -6).

A.1.1 Treatment of Soil Layers which were Judged to Liquefy

For those soil layers which were judged to liquefy according to the estimation of sub-section A.1 and are within 20 m of the actual ground surface, bearing capacities and other soil constants should be either neglected or reduced in the seismic design, by multiplying the original bearing capacities by reduction factors D_E which are determined in accordance with F_L -values in Table A.1.

Table A.1 F_L -D_E Relation

F_L	Depth, Z (m)	Reduction Factor, D_E
$F_L \leq 0.6$	$Z \leq 10$	0
	$10 < Z \leq 20$	1/3
$0.6 < F_L \leq 0.8$	$Z \leq 10$	1/3
	$10 < Z \leq 20$	2/3
$0.8 < F_L \leq 1.0$	$Z \leq 10$	2/3
	$10 < Z \leq 20$	1
$1.0 < F_L$	--	1

A.2 Estimation of Liquefaction (Method 2)

An alternative simplified procedure to evaluate liquefaction potential of sandy soils is as follows:

The cyclic shear stress ratio developed in the field due to earthquake excitations can be computed from:

$$(\tau_d/\sigma'_o) = \gamma_n (a_g/g)\ (\sigma_o/\sigma'_o)\ \gamma_d \qquad \text{(A-5)}$$

in which

a_g = design horizontal acceleration defined in Chapter 6, equation (6.1)
g = gravity acceleration
σ_o = total overburden pressure (Kgf/cm^2)
σ'_o = effective overburden pressure (Kgf/cm^2)
γ_d = stress reduction factor defined by γ_d = 1-0.015z
z = depth from the ground surface in meter
γ_n = 0.1(M-1)
M = magnitude of the biggest earthquake which can conceivably cause liquefaction at the site

The difference in number of cycles of stress due to different magnitude earthquakes is taken into account in Eq. A-5 with the reduction factor γ_d.

The soil resistance to liquefaction, τ_l/σ'_o, can be correlated with some form of modified penetration resistance according to the following procedure.

(a) Compute SPT N_a value normalized to the effective overburden pressure and fines content by:

$$N_a = 1.7\ N/\ (\sigma'_o + 0.7) + \Delta N_f \qquad \text{(A-6)}$$

in which N = measured SPT N-value
ΔN_f = modification factor in terms of fines content (percentage of fines smaller than 0.074mm) as shown in Fig. A-3

(b) Determine liquefaction resistance with limiting strain potential of γ-percent by Fig. A-4. The limiting strain potential is the maximum cyclic shear strain likely to be developed by the earthquake excitations.

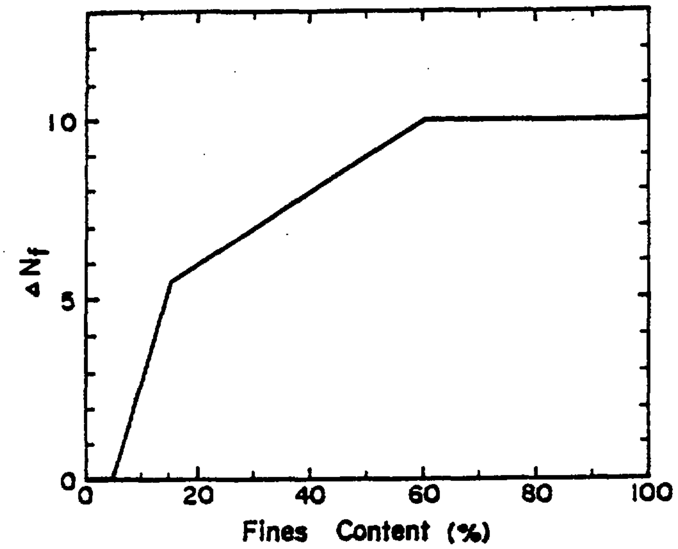


Fig. A-3 Relationship between ΔN_f and fines content

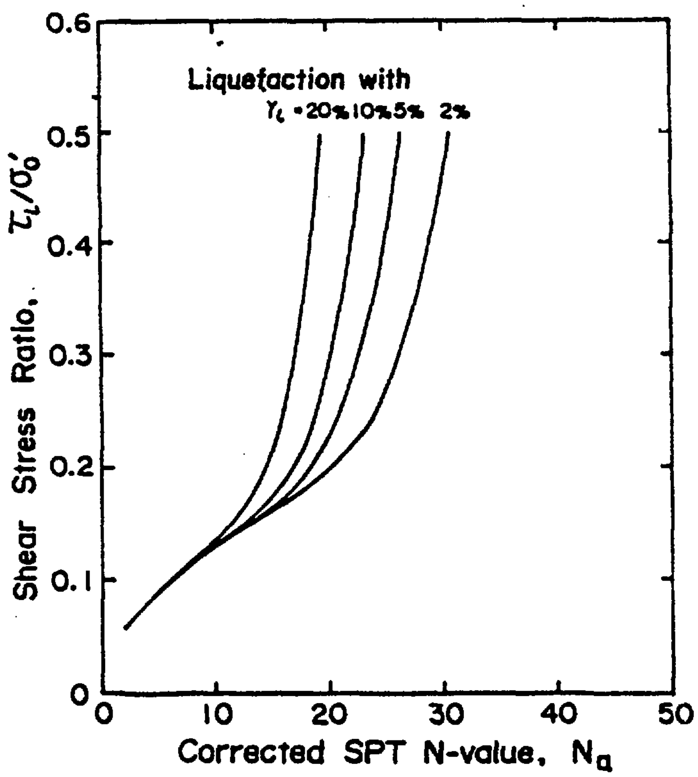


Fig. A-4 Relationship between cyclic stress ratio, N_a -values and limiting strain potential of sandy soil deposits

The factor of safety against liquefaction with limiting strain potential of γ-percent, F_1 can be determined by

$$F_1 = \frac{\tau'_1/\sigma'_o}{\tau_d/\sigma'_o} \tag{A-7}$$

Soil layers with a safety factor less than one can be considered to liquefy during the earthquake. Since the damage of liquefaction takes very different forms depending on the shear strain developed, the following guideline may be tentatively given to specify the degree of liquefaction.

Factor of Safety	Shear strain	Degree of liquefaction
F_1 <1	20%	Extensive
$F_1 = 1$	5 - 10%	Intermediate
$F_1 = 1$	2 - 5%	Slight
F_1 >1	2%	No significant

Treatment of soil layer which is judged to liquefy. The soil layer which is judged to liquefy should be densified or stabalized so that it will withstand earthquake shaking.

APPENDIX B - CAPACITY OF ANCHOR BOLTS

Cast-in anchors, with at least 6 diameters embedded length, should be used wherever possible, to obtain maximum pull-out strength. Where it is absolutely necessary to use drilled-in (expansion type) anchors, they should be of a proven type, easy to install and resistant to slippage or loosening under severe vibration or impact loading.

Where high-strength studs or anchor bolts are applied, they should be pre-loaded to 90% of their specified minimum yield strength to reduce the risk of prying or uplift during an earthquake, minimizing fatigue effects and as a convenient means of pre-testing the anchor connection. Where low-carbon, structural-steel anchors are used and/or sustained pre-loading is not dependable, the effects of prying and fatigue should be taken into account. In addition, consideration should be given to the adverse effects of flexible mounting plates, long-reach bolts or flexible concrete slabs in which anchor bolts are installed.

Transverse shear forces should be assumed to be applied directly to the bolts, unless shear keys are provided. Without such keys, shearing, as well as tension stresses, should be taken into account, using suitable interaction formulae (square law relationship).

Consideration should be given to the use of redundant anchors.

Care should be taken in spacing of anchors and in the distance between anchors and any free concrete edge, wall or corner, to ensure maintenance of adequate pull-out strength. Otherwise, a suitable strength-reduction factor must be applied to such anchors. Consideration should also be given to bolts for anchoring equipment to floors and especially walls and ceilings, where adequate spreader plates are used on the opposite side of the floor, wall or ceiling.

For cast-in-place anchors, the minimum factor of safety against failure in any mode, including pull out, should be 2.5.

For drilled-in, expansion-type anchors, the minimum factor of safety, against failure in any mode, including pull out, should be 4.

TABLE B.1

EARTHQUAKE RESISTANT DESIGN PROVISIONS

Key:

EQ = Earthquake SQ = Seismically Qualified NSQ = Not Seismically Qualified

Item	Design Provision
- Concrete block partition walls	Properly reinforce walls, dowel to floor or tie into steel work, to avoid collapse in an EQ
- Instrument stands and equipment platforms.	Add cross bracing; brace back to wall if tall. Anchor well to resist EQ forces and overturning moments.
- Cable trays (stacked trapeze-type and cantilevered)	Add bracing. Tie back to wall at suitable intervals and 90^{o} turns. Add protective covers, including fire protection. Lockweld joints.
- Drilled-in expansion anchors in lieu of cast-in (where essential)	Qualify by testing. Cast-in anchors recommended. High-strength anchor bolts preferred (pre-loaded). Redundant anchors desirable. Avoid grouted-in anchors. Through-wall anchors are best.
- Conventional pipe hangers	Add lateral restraints at suitable intervals. Replace rigid rods by swivel type. Avoid use of threading in the plane of maximum stress.
- Tank and equipment supports.	Add bracing. Double up anchors, with suitable spacing. Tie back to wall where bracing is insufficient or tank is too tall.

Key:

EQ = Earthquake SQ = Seismically Qualified NSQ = Not Seismically Qualified

Item	Design Provision
- Tall, overhung valves and valve operators	Add lateral restraints or motion-limiting stops, as necessary, to limit EQ-induced stresses.
- Overhead ductwork	Strengthen (lock) duct joints. Add end restraints. Use adequate supports. Use backup supports, where consequences of falling in an EQ are serious.
- Cantilevered small valves, gauges, fittings, etc.	Restrain valves or use short connections to avoid snapping off in an EQ.
- Small branch pipe or tubing connections	Motion limits. Good anchorage. Proper flexibility to allow for differential movement in an EQ.
- Long, vertical pipes supported at top and bottom only.	Use lateral restraints at suitable intervals, to provide for horizontal EQ effects.
- Overhead lighting (tubular fluorescent and mercury-vapour bulbs)	Use lateral restraints. Close chain hooks. Protective covers be well fastened.
- Electrical equipment cabinets, consoles, racks and centres	Use stiff frames. Strong hinges/latches (2 or more). Well anchored. Tie cabinets together across the tops. Reinforce tops against falling objects.
- Field-run tubing, small piping and electrical conduits, small valves and fittings	Route carefully or protect well to avoid impact interaction with larger pipes, ducts, etc. during an EQ. Use adequate clamps and supports.

Key:

EQ = Earthquake SQ = Seismically Qualified NSQ = Not Seismically Qualified

Item	Design Provision
- Local overhead coolers, heaters, intercoms, etc.	Strong supports. Use lateral bracing, especially where flexibly supported. Add back-up supports where consequences of falling in an EQ are serious.
- Water, fuel or lubricant lines and storage tanks.	Adequate support bracing. Use protective curbs and proper drainage, sprinklers, halon or other fire-protection feature to mitigate effects of an EQ.
- High-pressure gas storage bottles.	Secure bottles to storage racks at top and bottom. (Fallen bottles could become missiles in an EQ)
- Building to building clearance.	Provide enough rattle space or use soft caulking.
- Safety-related equipment close to NSQ equipment or structures.	Increase normal separation. Cage or barricade NSQ equipment. Protect safety-related equipment. Secure NSQ structures or equipment to prevent collapse. Add redundant or diverse safety-related equipment (well separated). Use fail-safe equipment.
- Storage batteries	Strengthen battery racks and anchors. Tie batteries to racks (top and bottom). Place batteries closer to floor level.
- Radioactive fuel/waste storage, hot cells, ventilated glove boxes, etc.	Secure storage areas. Confine and SQ cooling and shielding water systems. SQ ventilation systems, where necessary for safety.

Key:

EQ = Earthquake SQ = Seismically Qualified NSQ = Not Seismically Qualified

Item	Design Provision
- Cranes, hoists, jibs, moving bridges or working platforms	Make design provisions for tethering or clamping hoists/cranes in a safe position when out of service. Lower loads onto safe areas when hoisting/handling operations are over (administrative).
- Ladders, handrails, guard rails, stairways, etc.	Secure and lock handrails, ladders, etc. Mount equipment on separate SQ supports.
- Instrument air reservoirs.	Properly support supply-side check valves. Improve anchorage.
- Building wall penetrations	Use adequate clearance around penetrations, sealed with flexible, fireproof "boots" on the inside; or weld penetrations to embedments on the inside and use soft bedding, on the outside, with flexible terminations or bellows.
- Electrical equipment connections	Add short length of armoured, sheathed cable at all connection points, looped to avoid tension during an EQ. Bottom connections recommended (less concern for relative movements in an EQ).
- False or suspended ceilings. Loose furniture.	Secure ceilings and furniture close to sensitive equipment. Add curbs and railings around critical control consoles to prevent impact from furniture moving in an EQ.

Note: See illustrations of certain design provisions described above in Fig. B.1.

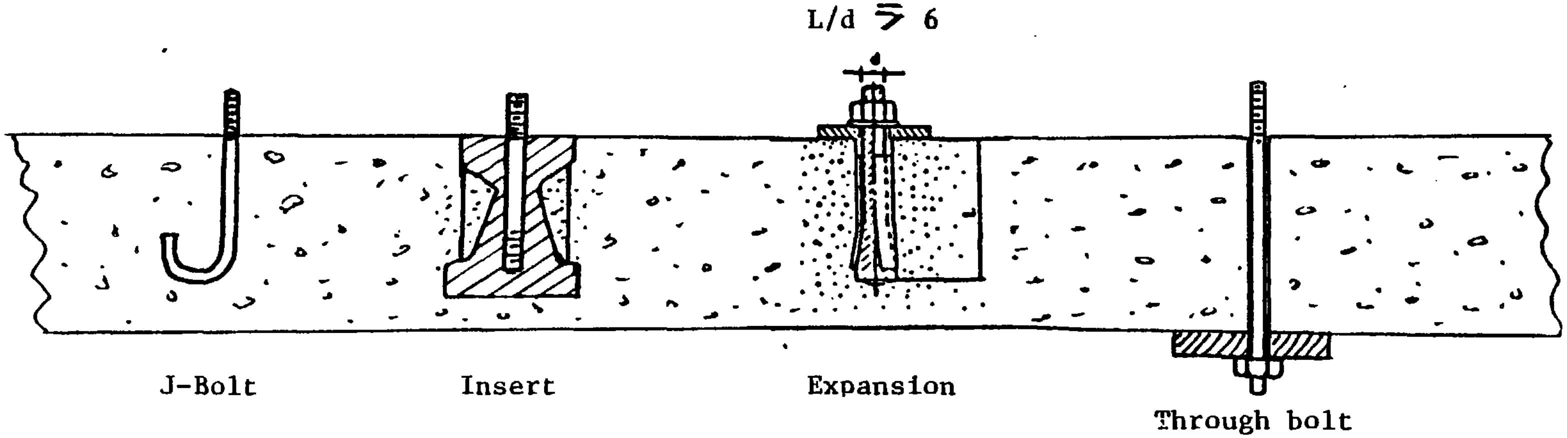


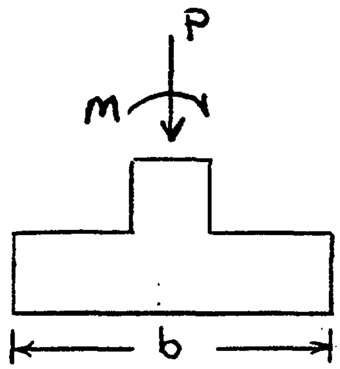
Fig. B-1 Anchor Bolts

APPENDIX C

Computation of Tensile (Pulling) Force on Anchor Device

The following procedures are considered acceptable methods of determining tensile and compressive forces on anchoring devices.

I. Simplified Conservative Method



$$T_b = \frac{M}{(d/_2 \cdot n)}$$

where:

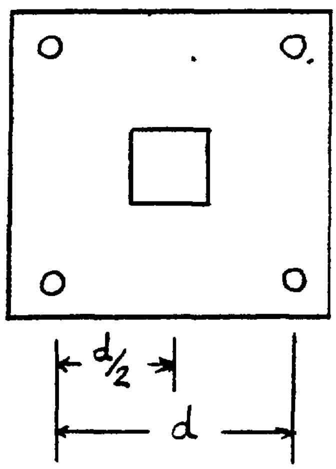
M = applied overturning moment

d = distance between bolts

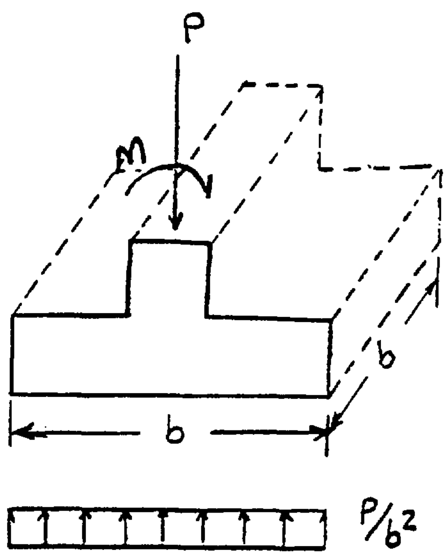
b = width of base or foundation

n = number of bolts

T_b = tensile force per bolt



II. Elastic Method - Square Foundations



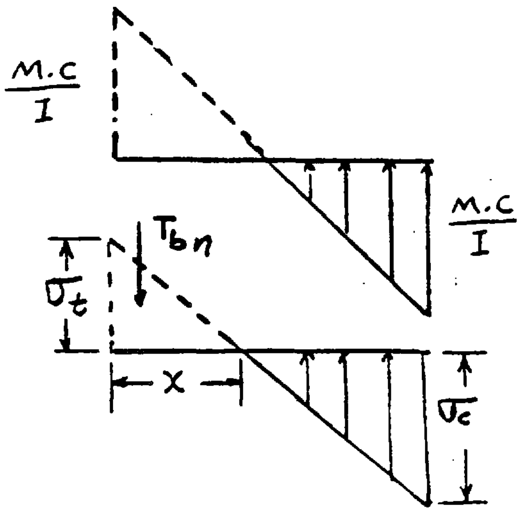
$$\sigma_c = (M.C/I) + (P/b^2)$$

$$\sigma_t = (M.C/I) - (P/b^2)$$

$$x = b \cdot \frac{\sigma_t}{\sigma_c + \sigma_t} = b \cdot \frac{\sigma_t}{2M.C/I}$$

then:

$$T_b n = 1/2 \quad \left(\frac{b \cdot \sigma_t}{2\ M.C/I} \right) \sigma_t \cdot b$$



where:

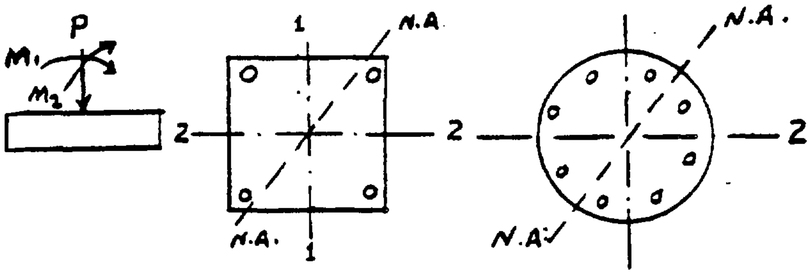
C = distance from neutral axis to the critical side edge

I = moment of inertia of the section.

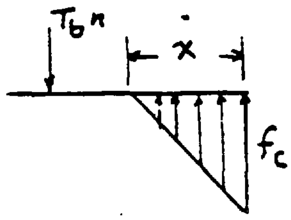
Allowable

$$f_c \leq 0.35\ f'_c$$

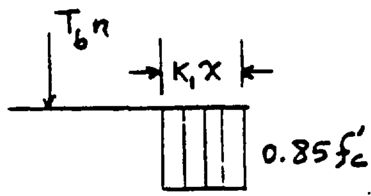
III. Concrete Column Analogy



Elastic Design



Strength Design



Determination of T_{bn} and T_b is based on methods of designing reinforced concrete columns in accordance with National Building Codes.

Other appropriate methods may also be used. In developing these tensile loads the equilibrium of tensile and compressive forces developed in the anchorage system as well as equating the applied moment to the couple formed by the section tensile force T_{bn} and compressive force should be considered. The distribution of section tensile force T_{bn} to individual bolts should be a function of tensile strain in the bolt which in turn is a function of an individual bolt's distance from the neutral axis (N.A.).

APPENDIX D

DETAILING OF DISTRIBUTION SYSTEMS

Span lengths for dead weight pipe supports are normally governed by deflection or sag limitation between supports. Such sag or vertical deflection limits are typically 1/100 of the span as provided in Table D.1. This limit results in a bending stress in the pipe due to weight 1.0 mg, of $0.05f_y$ where f_y is the yield strength of the piping material.

For seismic or lateral restraint of piping a maximum span of 4 times clad weight spacing is recommended. Since seismic bending stress varies as the square of the span length a 1.0 mg lateral load coefficient would result in a limiting

$$(4)^2 \; [\, 0.0\ 5f_y] = 0.80 \; f_y$$

bending stress in the pipe.

Four duct lateral restraints should be provided on a maximum spacing of 2 times dead weight support spacing when connections between duct sections is provided by frictions. If positive connectors are used which develop the axial and bending capacity of the duct a maximum lateral support spacing 4 times dead weight spacing of duct support is recommended.

For raceways a maximum spacing of 6 times dead weight spacing is recommended. In live lateral restraints stops should be provided at all changes of direction. Additional support requirement may be necessary when fire proofing is used.

Table D.1

<u>Standard Span Length of Piping Dead Weight Support</u>

Diameter	mm	20	25	32	40	50	65	80	100	125	150	200	300	600
	in	3/4	1	1 1/4	1 1/2	2	2 1/2	3	4	5	6	8	12	24
Span length	m	1.8		2.0			3.0			4.0		5.0	7.0	10.0

The use of brittle materials like ordinary cast iron, fibre reinforced plastics (FRP), is prohibited for piping, flanges and valves, for safety related piping.

APPENDIX E

CONNECTING CABLE AND PIPING BETWEEN COMPONENTS

Some recommended practice on detailing of connecting cable and piping between components are given in Figures E.1 - E.3.

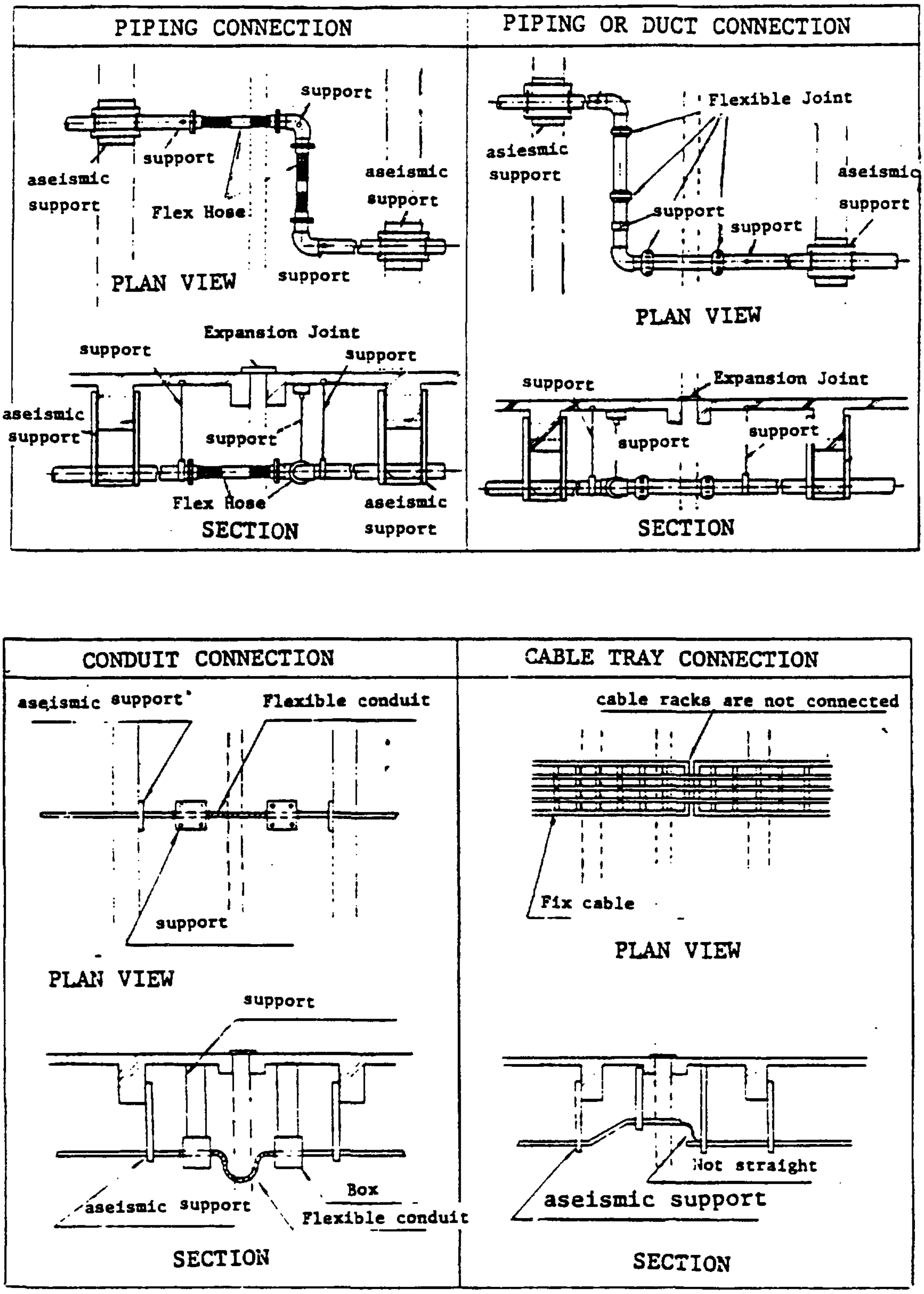
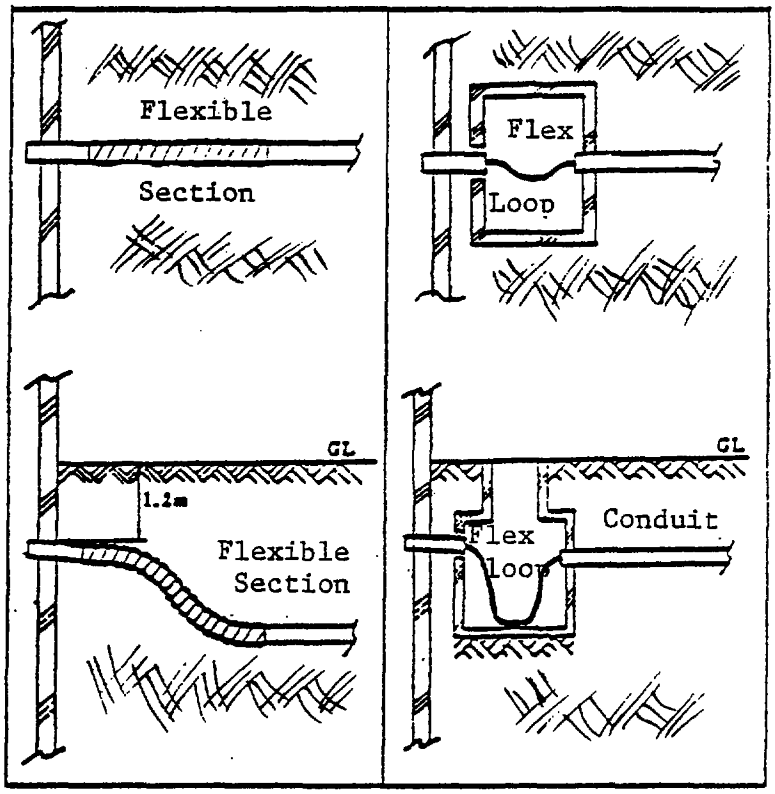
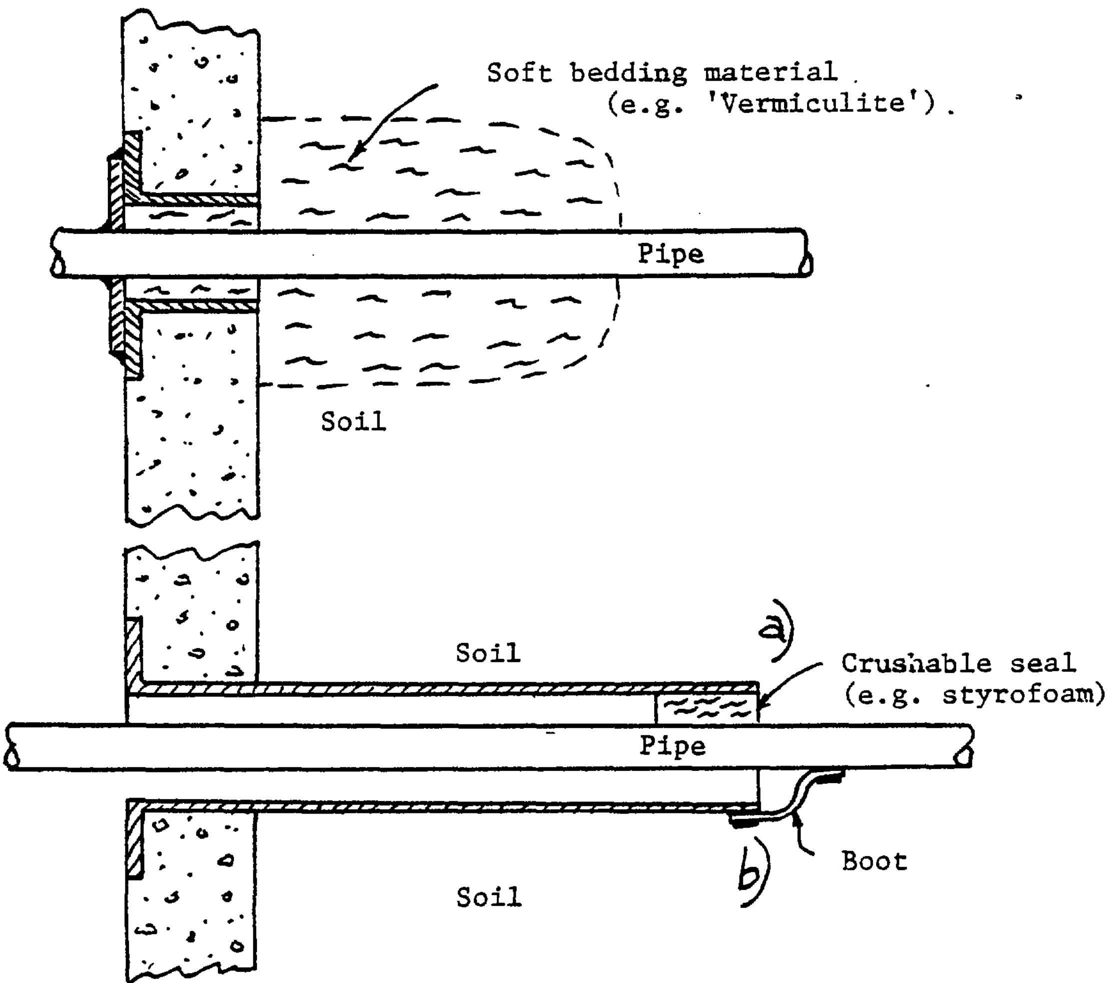


Figure E.1 Connection Between Buildings

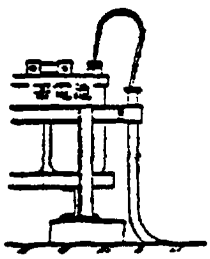


Underground Cable Penetrations

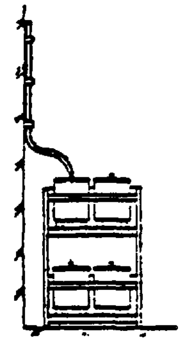


Underground Piping Connections

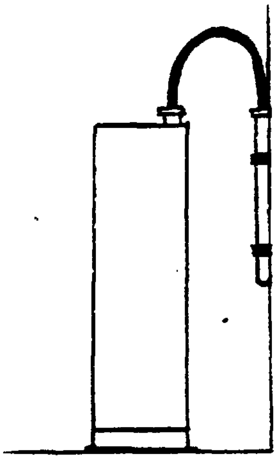
Fig. E-2 Connections for Buried Cables and Pipes



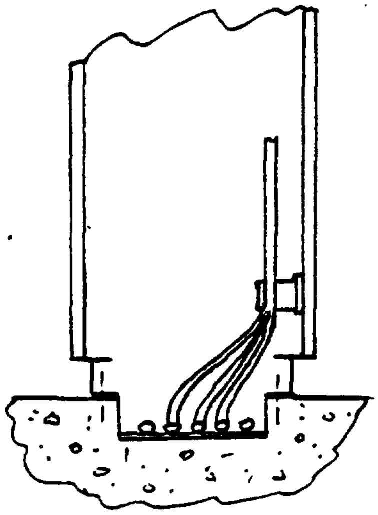
Battery connection
(side view)



Battery connection
(end view)



Cabinet
(top connected)

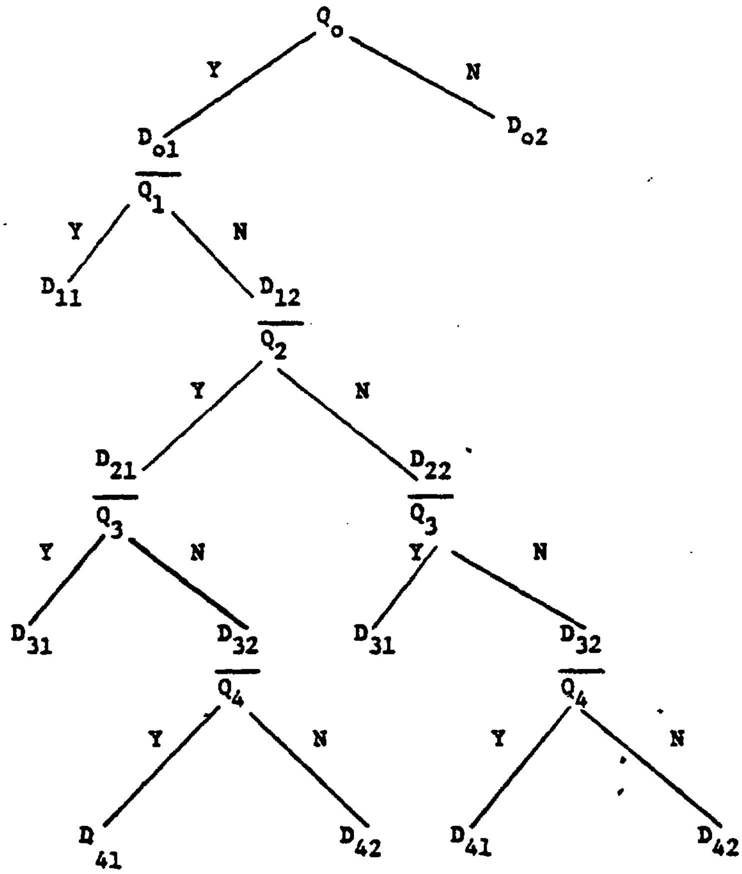


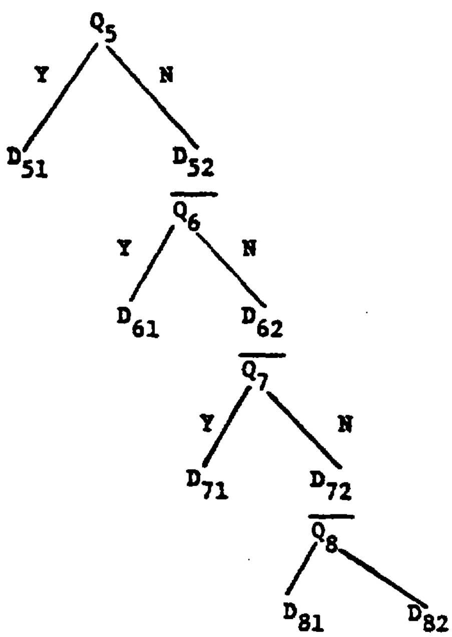
Cabinent
bottom connected
(preferred)

Fig. E-3 Connections to Equipment

APPENDIX F

DESIGN TREE DIAGRAM





Q: Question

Y: Yes

N: No

D: Decision

LEGEND TO DESIGN TREE DIAGRAM

Q_o: Is the facility one with limited radioactive inventory?

D_{01}: The document may be applicable

D_{02}: The document is not applicable, refer to IAEA 50-SG-S1, 50-SG-S2, 50-SG-S8.

Q_1: Are the Basic Criteria still maintained in case of collapse of buildings loss of normal leak tightness of the pool or other containment structures and large disruption of core or fuel?

D_{11}: Facility may be classified in Class C. Normal building and other industrial codes may be used.

D_{12}: Facility should be classified in Class A or Class B. The document is applicable.

Q_2: For the Basic Criteria to be satisfied is it sufficient that the building does not collapse, the pool or other containment structures do not lose normal leak lightness, and no big debris falls on the fuel or the core in case of earthquake?

D_{21}: Facility may be classed in Class B. Equivalent Static Approach may be used.

D_{22}: Facility should be classified in Class A. Simplified Dynamic Approach should be used.

Q_3: (to be posed for each C.E. structure). Is it necessary for the building (or the pool) to preserve normal leak tightness?

D_{31}: Structure should be in Category I, $\mu = 1$

D_{32}: Structure may be in a different category.

Q_4: Is it necessary for building to preserve basic stability?

D_{41}: Structure should be in Category II, $\mu = \mu_d$

D_{42}: Structure may be non-categorized, i.e. no additional seismic design required.

Q_5: Is it necessary for equipment to maintain active functioning?

D_{51}: Equipment should be in Category I, $\mu = 1$.

D_{52}: Equipment may be in a different category.

Q_6: In case of failure can equipment endanger or impair a Category I item?

D_{61}: Equipment should be redesigned or relocated such that the danger is alleviated or put in Category I.

D_{62}: Equipment may be in Category II or non-categorized.

Q_7: Is it necessary for equipment to preserve basic stability (i.e. no collapse)?

D_{71}: Equipment should be in Category II, $\mu = \mu_d$.

D_{72}: Equipment may be non-categorized.

Q_8: If not seismically qualified, can equipment become a missile under earthquake loading?

D_{81}: Equipment should be in Category II, $\mu = \mu_d$.

D_{82}: Equipment may be non-categorized.

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