PART II

APPENDIX A

SEISMOLOGICAL INVESTIGATION AND DESIGN CRITERIA

PREPARED BY CONVERSE CONSULTANTS
MAY 1983

NOTE: THIS APPENDIX IS FOR YOUR INFORMATION AND USE AS REQUIRED BY THE SUPPLEMENTARY CRITERIA FOR SEISMIC DESIGN OF UNDERGROUND STRUCTURES DATED JUNE 1984.

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Gentlemen:

Subject: Structural Seismic Design Criteria for Metro Rail Project

Enclosed is structural seismic design criteria for the proposed Metro Rail Project.

It is understood that the seismic design criteria will be attached as an appendix to the Seismological Investigation and Design Criteria report prepared by Converse Consultants.

In addition to Lindvall, Richter & Associates the following persons also participated in the preparation of the structural seismic design criteria:

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Very truly yours,

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Enclosure

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APPENDIX A

STRUCTURAL SEISMIC DESIGN CRITERIA
METRO RAIL PROJECT

1.0 INTRODUCTION

1.1 Purpose

This Appendix provides structural-seismic design criteria for the Southern California Rapid Transit Metro Rail Project. Geotechnical information has been synthesized from the Geotechnical Investigation Report so that the project structural engineers can utilize this document as an independent source of seismic criteria.

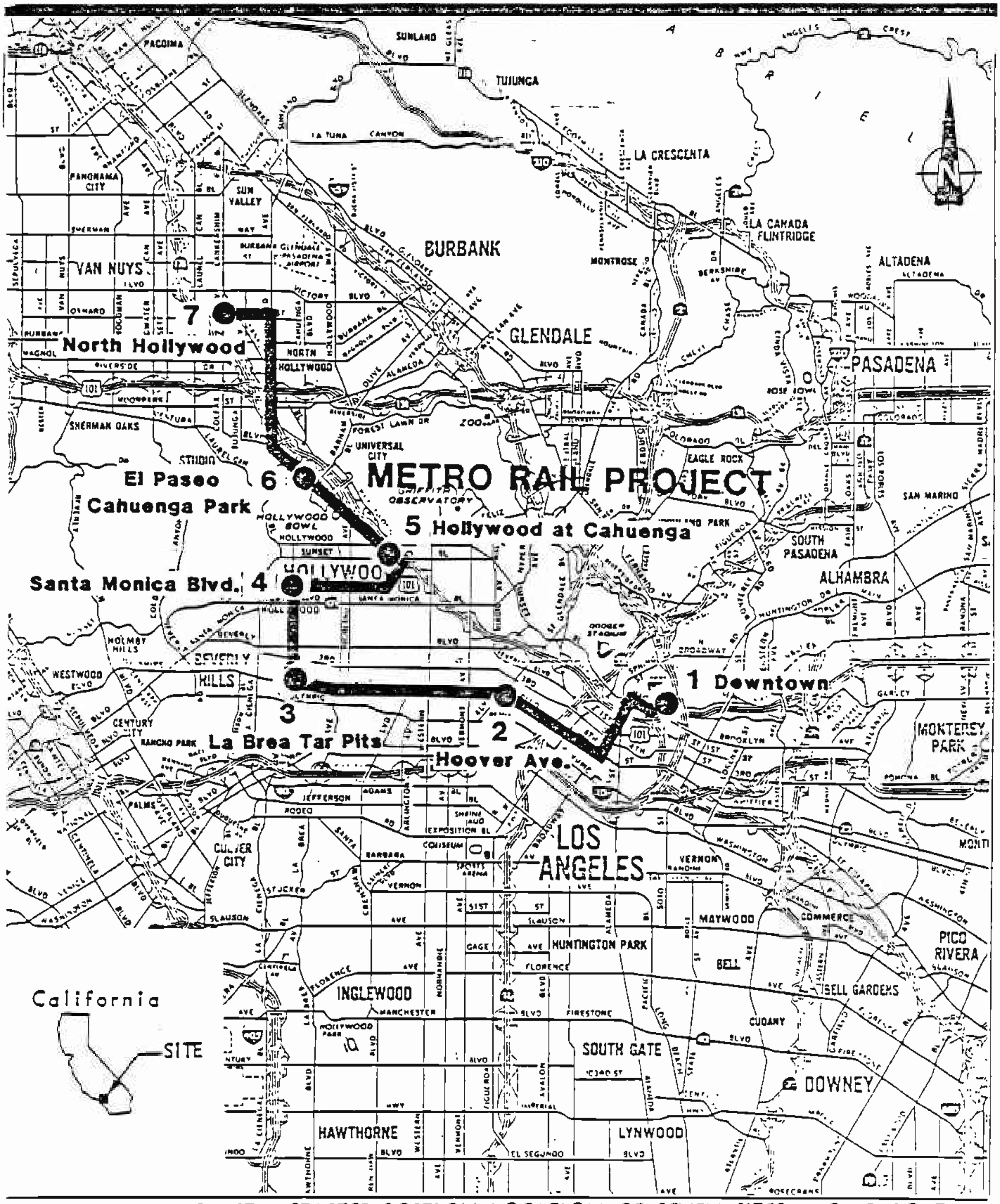
The basic structural design criteria for the project are provided in the SCRTD Criteria Document, (Reference A.1). This Appendix, however, provides special seismic design criteria for the project which supplement and supplant corresponding provisions of the criteria provided in the SCRTD document. In other words, the criteria provided herein take precedence for purposes of seismic design and qualification.

Special seismic criteria are required for the Metro Rail Project because of the relatively high exposure of the public to significant earthquake hazards. The hazards are manifested by the potential vulnerability of large numbers of passengers in this extensive system.

Three terms which are used throughout this appendix are "District," "Project" and "Engineer." "District" is used to mean the Southern California Rapid Transit District (SCRTD). "Project" is the Metro Rail Project. "Engineer" is used to mean the engineers contracted with by the District to perform the detailed structural design and to be in responsible charge of this work.

1.2 Scope and Coverage

The Metro Rail Project is a high-speed transit subway system planned for the greater Los Angeles Metropolitan Area. In particular, the starter portion is intended to connect downtown Los Angeles with Hollywood and then extend to the North Hollywood area of the San Fernando Valley. The latter leg would run in the direction of and generally parallel to the Cahuenga Pass. See Figure A-1 for Project location of the starter portion.



METRO RAIL PROJECT - STARTER PORTION LOCATION OF SEVEN SITES ALONG ROUTE

Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.

Figure No.
A-1

As stated above, this Appendix serves as a stand-alone structural-seismic design criteria document. Though not a completely detailed design specification, sufficient detail and guidelines are presented so that uniformity can be provided in the seismic design of the project. This, in turn, will provide for appropriate levels of seismic safety for the project facilities. The criteria herein are intended to supplement the general structural criteria provided in the SCRTD document (Reference A.1) and provide major changes in the seismic criteria area.

This Appendix includes four major chapters. Chapter 1 provides introductory and background information. Chapter 2 provides general discussion on the seismic design approach and philosophy. Also, seismic classes are defined in Chapter 2. Seismic classes are to be assigned to all project structures, systems, equipment and components. Seismic classification is based on item criticality with respect to public safety. Design requirements related to seismic class are then provided. Requirements for Seismic Class B and C items are given in Chapter 3; this chapter is quite brief as the seismic requirements cited are basically those of the referenced standard codes. Chapter 4 provides the detailed structural design requirements for Seismic Class A items. These requirements are divided into two basic types: seismic design and seismic qualification. Further, requirements for structures are separated with respect to type and location: buried line structures, partially buried and above-grade facilities, and aerial facilities. Additional special requirements are cited for structures at or adjacent to major fault crossings.

1.3 Alternate Criteria

Alternate criteria to that provided herein may be used by the Engineer for seismic design and qualification only on the following bases:

a. The Engineer submits the alternate criteria in writing to the District with appropriate substantiating data.

b. The District, upon review, agrees in writing to accept the alternate.

c. The circumstances and limits of use of the alternate are specified and approved.

2.0 DESIGN PHILOSOPHY

2.1 General Approach

The Metro Rail Project is a large-scale public project in an area highly susceptible to major earthquakes. Further, earthquake-initiated failures of selected structures and systems could lead to large scale loss of life. For this reason the District has developed special earthquake protection criteria for the project. These special criteria exceed minimum code provisions which would otherwise be utilized. Such code provisions are considered to provide only minimum though satisfactory levels of protection for normal usage buildings and facilities.

The guiding philosophy of earthquake design for the project is to provide a high level of assurance that the overall system will continue operating during and after an Operating Design Earthquake (ODE). Further, the system design will provide a high level of assurance that public safety will be maintained during and after a Maximum Design Earthquake (MDE). The definition of ODE and MDE levels is provided in Sections 2.2 and 4.3.

2.2 Risk Criteria

This section provides the basic philosophy related to defining levels of risk for the Metro Rail Project. Seismic classification is described in Section 2.3. However, for purposes of outlining risk criteria, Seismic Class A-1 and A-2 items of the Project are those with a primary function required to maintain public safety. Items in Seismic Classes B and C are those whose loss of function will not significantly impact public safety.

The level of seismic risk for the Project thus is established by assigning the level of earthquake to which critical items (Seismic Classes A-1 and A-2) are to be designed or qualified. Design and/or qualification indicates that a high degree of assurance is provided that such items will maintain their required function.

Two levels of earthquake are considered for design of critical items. The Operating Design Earthquake (ODE) defines, for any point on the subway system, the level of ground shaking at which critical items maintain function so that the overall system will continue to operate normally. The Maximum Design

Earthquake (MDE) defines the level of ground shaking, for any point in the subway system, at which critical items continue the function required to maintain public safety, preventing catastrophic failure and loss of life.

The ODE is defined as the earthquake event which has a return period of several hundred years. Such an event can reasonably be expected to occur during the 100-year facility design life. The probability of exceedance of this level of event is on the order of 40 percent during the facility life.

The MDE is defined as the earthquake event which has a return period of several thousand years. Such an event has a small probability of exceedance during the facility life. This probability is on the order of five percent or less.

The risk criteria as given above are only completely defined by assigning appropriate parameters relating the level of ground shaking to each earthquake event, ODE and MDE. These design criteria are given in Section 4.3. Also relevant to the actual level of risk is the design and analysis approach and performance requirements for items designed to the assigned levels of ground shaking. These criteria are provided in Section 4.5 and thereafter.

Part of the basic criteria regarding risk and overall system performance relates to general Seismic Categories and definitions of failure (Reference A.2). These are defined below:

Seismic Categories for System Performance

Category I - structures, components and systems which perform a vital safety-related function. Category I structures, components and systems shall be designed to avoid catastrophic failures and perform their vital safety-related function during and following the upper level design earthquake.

Category II - structures, components and systems (not in Category I) which are required to maintain safe and reliable system operation. Category II structures, components and systems shall be designed to avoid catastrophic and critical failures during and following the upper level design earthquake and remain operational during and after the lower level design earthquake.

Category III - structures, components and systems (not in Categories I or II) which are required for normal system operation. Category III structures, components and systems shall be designed according to appropriate code provisions.

Definitions of Failure

Catastrophic Failure - a failure that would result in loss of life and/or system. In this case the system is one that is required for reasons of safety to remain operational both during and following an earthquake.

Critical Failure - a failure that would result in severe injuries, severe occupational illness and/or major system damage. Major system damage resulting from an earthquake as defined should not cause loss of life.

Marginal Failure - a failure that would result in minor injury, minor occupational illness and/or minor system damage. Minor system damage resulting from an earthquake as defined should not significantly affect system operations, or induce injury.

Negligible Failure - a failure that would not result in injury, occupational illness and/or system damage.

2.3 Seismic Classes for Design

The definitions of seismic categories and failures given above are provided for overall hazard consideration and are consistent with the District's philosophy of minimizing risk to the public. For purposes of facilitating design, the above definitions are extended into seismic classes defined below. The definition of seismic class more directly relates to the seismic design process and will be used as a basis for assignment of corresponding requirements throughout the remainder of this document. Also, a definition of failure specifically related to Seismic Classes A-1 and A-2 is given.

During the design phase of the project, the Engineer shall prepare a detailed list providing seismic classes for all items of the project. This list shall be maintained, updated and submitted for review and approval to the District on a periodic basis.

2.3.1 Definition

In order to satisfy the seismic risk philosophy given above, all items shall be assigned to one of three seismic classes. Seismic class is a measure of criticality of each item as determined by the consequences of its failure. Refer to Table A-1. The terms "items" and "failure" are defined in Subsection 2.3.3.

TABLE A-1

DEFINITION OF SEISMIC CLASS

SEISMIC CLASS	CONSEQUENCES TO PUBLIC OF FAILURE
SC A	Major Injuries or Fatalities
SC B	Minor or Moderate Injuries
SC C	No Injuries

Further, SC A items are to be classified into two subgroups for convenience in assigning qualification requirements. SC A-1 items are those whose required performance under MDE conditions is primarily structural. See "function 1" in Subsection 2.3.3. Also, SC A-1 items, as appropriate, are required to remain operational during an ODE. This requirement primarily relates to equipment, for example, an elevator which is not required to remain operational after an MDE. SC A-2 items are those whose required performance involves continuing operation during and after an MDE. See "functions 2, 3 and 4" in Subsection 2.3.3.

It is noted that Seismic Class SC A includes structures, components and systems that would fall into Seismic Categories I and II. Seismic Category III items are to be incorporated into Seismic Classes B and C.

2.3.2 Design Requirements

The understanding of Seismic Class is broadened by inclusion of a brief summary of design and performance requirements associated with each Class.

SC B and SC C items are to be designed to meet Building or other applicable Codes. Importance factors I (Reference A.3), of 1.5 and 1.0 are included for SC B and SC C items, respectively. Requirements for SC B and C items are further amplified in Chapter 3.

SC A items are to meet earthquake design and qualification requirements given in Chapter 4. Requirements include consideration of three orthogonal components of earthquake ground motions which occur concurrently.

SC A-1 items are those whose critical function is structural. SC A-1 items shall be designed to perform elastically during and after an ODE. SC A-1 items shall be checked to assure adequate structural capability including acceptable damage levels and prevention of collapse when subjected to an MDE.

SC A-2 items are those whose critical function is operational. SC A-2 items shall be designed or shown by qualification to be capable of performing their required function or functions during and after an MDE level event. SC A-2 structural items, for example, critical equipment supports, shall be designed to perform elastically during the MDE level event.

2.3.3 Terms

For purposes of seismic design and qualification the terms "item" and "failure" are defined herein.

The use of "item" in conjunction with "Seismic Classes" or in the context of seismic design or qualification shall mean any system, subsystem, or component as applicable to the case in consideration. Items refer to systems, subsystems, or components whose function is structural, mechanical, electrical, controls, piping, vessels, architectural, heating, ventilating and air conditioning (HVAC), or any combination thereof.

"Failure" for purposes of this criteria is defined as the discontinuance of capability of an item to perform a required function. The required function may be one or more of the following:

1. A level of structural performance to maintain an item's position and/or to provide support for itself or other items in Seismic Class A-1.
2. A level of structural performance to maintain an item's position and/or to provide direct support for items in Seismic Class A-2.
3. A level of structural performance needed to maintain confinement of critical fluids or gases.
4. A level of operability such as continuing or minimally interrupted operation of an item of mechanical, electrical, hydraulic or other similar function.

Failure or failures of items and their consequences must be assessed on an item by item basis as part of the seismic classification process.

2.4 Summary of Seismological and Geotechnical Investigation

This section provides a brief summary of the seismological and geotechnical data presented in detail in the main body of this report. The purpose of the summary is to provide background for the correlation between the basic geotechnical data and the input design parameters given in Section 4.3. In order to avoid possible confusion, it should be noted that the basic seismological and geotechnical parameters described in this section for background purposes differ from the design earthquake parameters given in Section 2.2 and in Chapter 4. In particular, note that the return periods discussed in the main body and in this section differ from the period ranges which are defined in Section 2.2 for the design earthquakes.

2.4.1 Seismic Exposure

As part of the seismological investigation, 15 significant (regional) seismogenic faults were studied. Reference is made to Chapter 3 of the main body of this report. The 15 faults were considered major potential sources of strong ground motion that could affect the Metro Rail Project. Major nearby regional faults include the Malibu-Santa Monica, Hollywood, Raymond and Newport-Inglewood. The most significant distant structure is the San Andreas. Maximum Richter Magnitude was estimated for each fault. The estimates were based on postulated fault rupture length.

In addition to seismic shaking, fault rupture is also a potential hazard. The proposed starter portion of the Metro Rail alignment crosses at least 12 faults. Of the 12 faults, only the Hollywood and Santa Monica are considered to have the potential for this hazard.

In order to estimate potential strong ground motion along the route, a statistical analysis was conducted taking into consideration regional seismogenic faults, geologic evidence for fault activity and historic seismic activity. Results of the analysis indicate that any one of nine regional faults within 30 miles of the proposed alignment is considered capable of generating an average 100-year peak horizontal acceleration of 0.22g. Major faults that are considered of prime importance in developing probable 100-year ground motions are the Newport-Inglewood, Sierra Madre and San Andreas.

Upper or limiting ground motion parameters were also estimated. Limiting values are generally considered independent of time and are based on estimates of Maximum Credible Earthquakes (MCE) for each of the 15 regional faults. The limiting peak horizontal acceleration of 0.70g from an MCE of magnitude 7 is related to the Malibu-Santa Monica fault zone.

2.4.2 Geotechnical Considerations

As part of the geotechnical investigation (report main body), data were collected and are presented on the performance of tunnels during earthquakes. Information is also provided on a method of calculating shearing, axial, bending and hoop stresses. Potential liquefaction zones were investigated, and methods are outlined for mitigation of this hazard. In addition, lateral earth pressures and dynamic bearing capacities are discussed.

Past performance of tunnels during seismic events indicates that damage may result from primary or secondary effects of earthquakes which include: (1) strong ground motions, (2) fault rupture; (3), regional tectonic movements, (4) landslides, (5) liquefaction and (6) differential compaction or consolidation of sediments. Instances of complete tunnel closure were associated with combined primary and secondary effects of earthquakes such a fault rupture and slope failure. However, in general, tunnels are safer than above-ground structures for a given level of shaking.

A correlation is noted between free field or ground surface acceleration levels and tunnel damage. Specifically, little damage is noted in rock tunnels for surface accelerations less than 0.4g, and no tunnel collapse has occurred for surface accelerations less than 0.5g. Other important factors which contribute to tunnel damage are: (1) increase in the lateral forces from the surrounding soil backfill in cut-and-cover structures, and (2) the duration of strong ground motion.

A major contribution to deformations and corresponding stresses in long linear structures such as tunnels is traveling seismic waves. In a simplified manner, traveling wave effects can be accounted for by assuming that the tunnel and surrounding soil move together as the wave passes, and that motion from point to point along the route follows the wave pattern and differs from point to point only due to a time lag.

Liquefaction is the transformation of a solid (saturated cohesionless soil) into a liquid state as a result of strong ground motion. The vibratory motion results in build-up of pore water pressures with resulting soil failure which can have significant effects on engineered structures. These effects can include loss of bearing capacity, increased active pressures and decreased passive pressure, differential settlements. significant lateral displacements, and increased uplift forces. Soils in a liquefied state do not conform to standard solid soil mechanical behavior and, therefore, require special design considerations. Refer to Section 4.5.

3.0 DESIGN REQUIREMENTS FOR SEISMIC CLASS B AND C ITEMS

3.1 Applicable Documents

Except as otherwise provided in Chapter 3 herein, structural design of Seismic Class B and C items of the Metro Rail Project shall be governed by the standard SCRTD criteria (Reference A.1) and all specifications, codes, and documents incorporated thereto. The criteria given in this chapter, in fact, are essentially the same for seismic design as those given in the standard SCRTD criteria and supporting documents. The few changes are relatively simple in application.

For this project any Seismic Class B or C buildings or building structural components shall be governed by the provisions of the City of Los Angeles Building Code, Reference A.3, except as modified in Section 3.2 and 3.3 below. The Los Angeles County Code shall not be utlilized.

3.2 Requirements for SC B Items

All Seismic Class B items shall be designed according to the seismic requirements of the applicable documents cited in Section 3.1 except that an Importance Factor, I = 1.5 shall be used in all cases for determining lateral forces.

3.3 Requirements for SC C Items

All Seismic Class C items shall be designed according to the seismic requirements of the applicable documents cited in Section 3.1 above without exception.

4.0 REQUIREMENTS FOR SC A STRUCTURES AND EQUIPMENT

4.1 General

This chapter provides criteria for structural/seismic design as well as seismic qualification of SC A items.

The design requirements are applicable to the usual design process. In this process a structural system is first sized and a configuration selected. This trial system is checked in the design process, and modified until a satisfactory design is achieved. The design process consists of determining structural response under loads. For a successful design the response must be limited to prescribed levels of structural resistance or capacity.

The criteria necessary to fulfill the design process are given in subsections below. Included are definitions of seismic input and loading conditions, acceptable analysis procedures for determining seismic response, and acceptance criteria in the form of structural response limits for loading combinations to be considered. Additional criteria in the form of structural material, system, and detail requirements are also provided.

The seismic qualification process is one in which an existing item, previously engineered or designed, is reviewed to determine if it fulfills the appropriate acceptance criteria. If not, the item must be retrofitted to bring it up to appropriate standards.

The qualification process may be fulfilled by response analysis in some cases. Often, however, testing is required to assess adequate performance. This is especially true for items whose operation must be assured during and/or after a prescribed seismic event.

The qualification process is generally for application to pre-engineered hardware and components as contrasted to application of the design process to structural systems. The criteria for seismic qualification are similar to the design criteria. The system to be qualified must be shown to meet the appropriate response limits under seismic loading conditions. Thus much of the criteria for seismic qualification are the same as for design.

4.2 Applicable Documents

Except as otherwise provided in this Appendix, structural design of Seismic Class A items of the Metro Rail Project shall be governed by the standard SCRTD criteria (Reference A.1) and all specifications, codes, and documents incorporated thereto. Structural design unrelated to earthquake effects is not covered herein. The criteria given in these chapters, however, provide almost total revision to the code-like earthquake design provisions which are specified in the standard SCRTD criteria and its supporting documents.

4.3 Seismic Environment

This section provides criteria for earthquake-related environment including design ground motion input and special loading considerations. The criteria are either directly defined herein as for design ground motions or included by reference to other sections or other documents, as for some special loading considerations.

4.3.1 Design Ground Motion

4.3.1.1 Design Ground Motion Parameters - The design ground motion values given in Table A-2 are to be used for all locations of the Project. The design ground motion parameters are to be used as input or to define input for the following cases:

(a) Traveling wave effects on line structures (tunnels) or other relatively long buried structures of foundations -

The design particle motions are to be used for determining ground motion wave-induced stresses as specified in Subsection 4.4.6.

(b) Vibratory motion response of buried structures -

The design ground accelerations (dga's) given are to be used for determining peak inertial forces in buried structures or for defining dynamic analysis of items supported on buried structures.

(c) Vibratory motion response of partially buried, above-grade, and aerial structures -

The design ground accelerations (dga's) given shall be utilized to define input for calculation of amplified dynamic response; in such cases dga's are to be used (1) to define coefficients for simplified analysis, (2) to set roll-off (high frequency "anchor" value) acceleration levels for criteria ground motion design spectra, and (3) to establish appropriate values for scaling time histories of ground motion.

TABLE A-2

DESIGN EARTHQUAKE PARAMETERS

DESIGN EARTHQUAKE	FOUNDATION CONDITION	DESIGN GROUND MOTION PARAMETERS					
		ACCELERATION (g)		VELOCITY (ft/sec)		DISPLACEMENT (ft)	
		Hor.	Vert.	Hor.	Vert.	Hor.	Vert.
ODE	Soil	0.30	0.20	1.4	1.0	1.6	1.0
	Rock	0.30	0.20	0.8	0.6	0.5	0.3
MDE	Soil	0.60	0.40	3.2	2.1	3.3	2.2
	Rock	0.60	0.40	1.9	1.3	1.0	0.7

2.4 by [illegible]

Duration of strong motion

ODE = 15 - 20 sec.
MDE = 25 sec (nearby faults); 30+ sec (San Andreas Fault).

4.3.1.2 Design Spectra - Elastic free field design spectra for use as input in seismic analysis of structural response are given in Figures A-2 and A-3 for a horizontal direction. These spectra apply to both soil and rock foundation conditions. Several curves are provided corresponding to selected levels of structural damping. Appropriate damping levels for various analysis conditions are given in Paragraph 4.4.3.7. Horizontal design spectra for damping values not shown can be constructed. The two high frequency break points between which straight lines are drawn connecting the design level acceleration with the spectral accelerations are 8 Hz and 33.3 Hz. (Refer to Figures A-2 and A-3). Design spectral bounds S_a, S_v, and S_d can be computed for the ODE using the following equations, respectively:

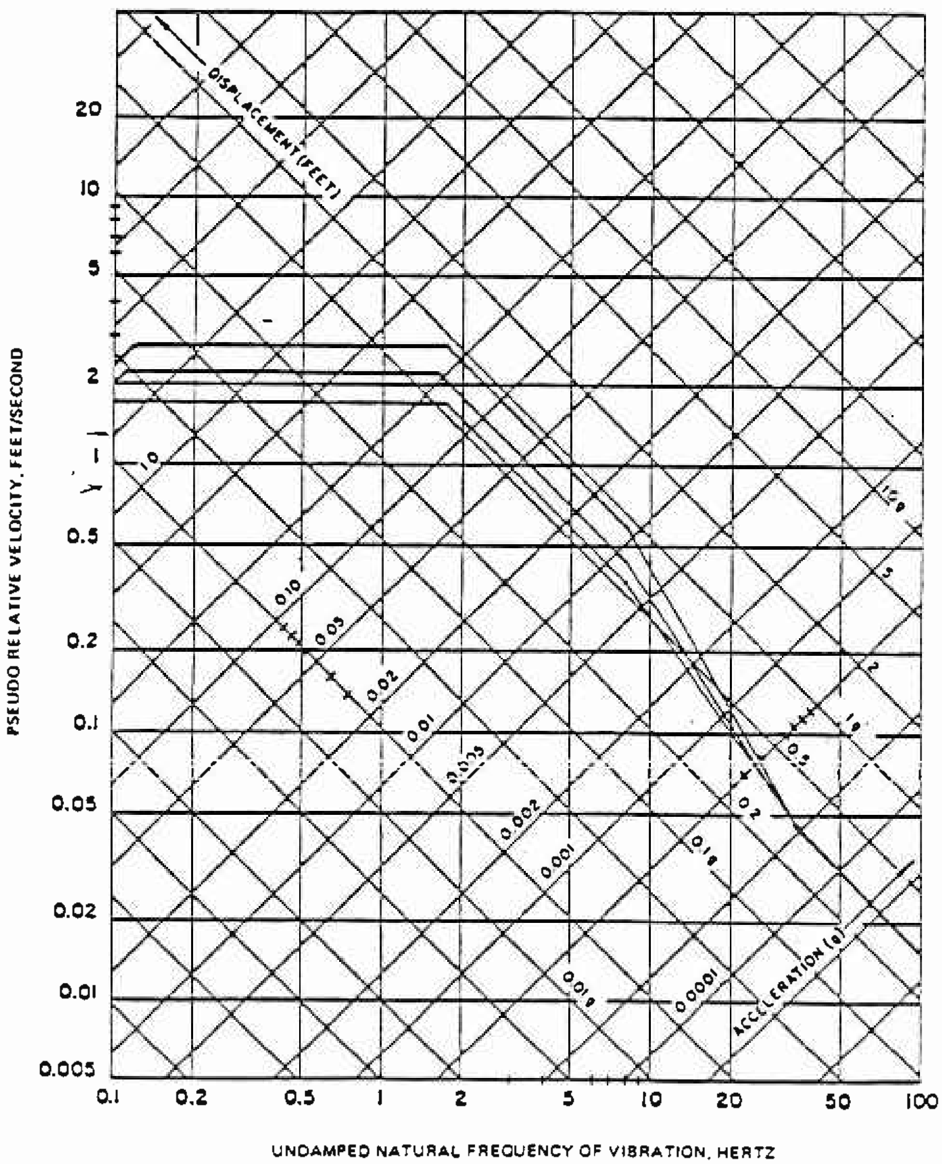
$$1.04 - 0.22 \ln D$$
$$3.14 - 0.62 \ln D$$
$$4.29 - 0.56 \ln D$$

Similarly for the MDE:

$$2.11 - 0.45 \ln D$$
$$6.98 - 1.38 \ln D$$
$$9.29 - 1.21 \ln D$$

In the above equations "D" is the selected fraction of critical damping. S_a and S_d will plot on logarithmic coordinates as 45° straight lines. Connecting S_a and S_d will be the horizontal line representing S_v.

For any given value of damping, the vertical design spectra shall equal two-thirds the horizontal design spectra for frequencies of engineering interest. Vertical design accelerations and response spectra are to be adjusted for near-fault locations. Refer to Paragraph 4.5.4.9



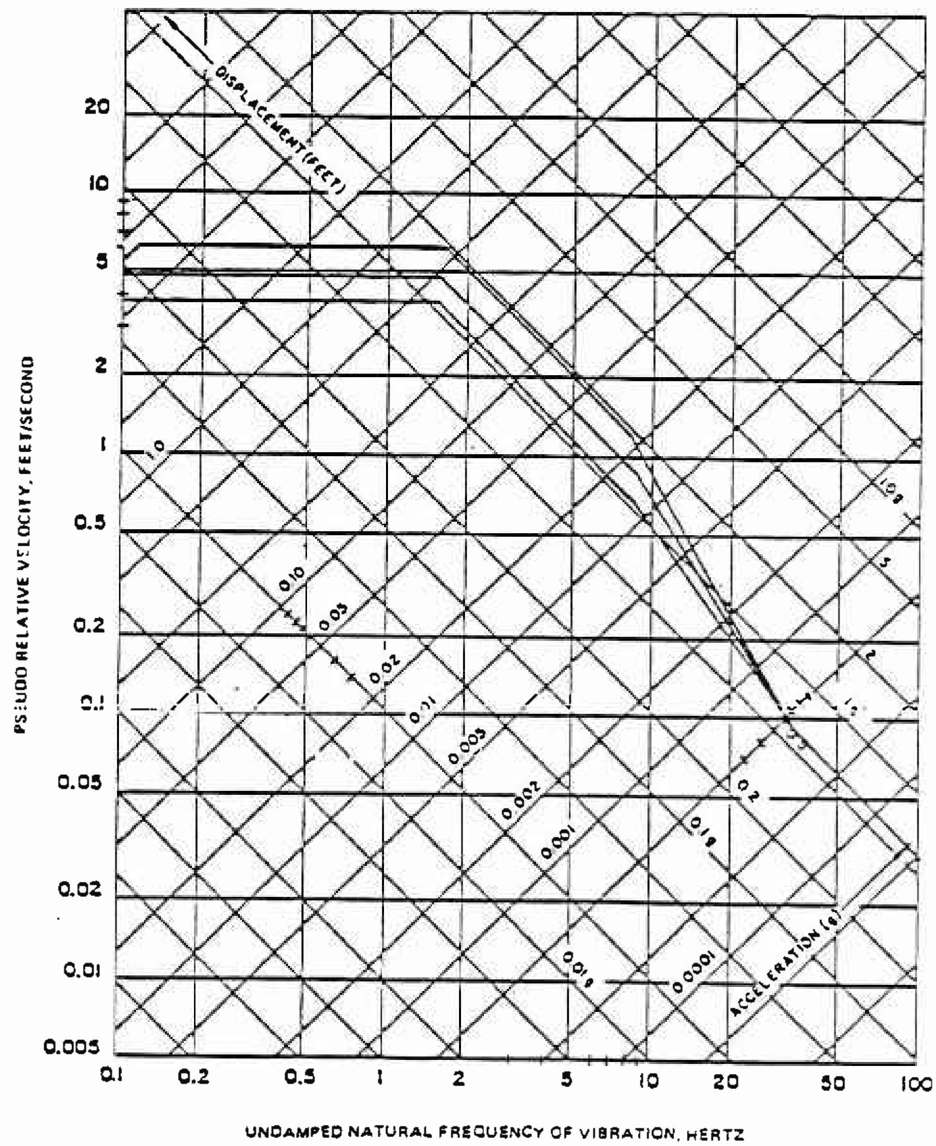
(2, 5, & 10% Critical Damping)

ODE HORIZONTAL DESIGN SPECTRA

Project No

Southern California Rapid Transit District
METRO RAIL PROJECT

Figure No
A-2



(2,5, & 10% Critical Damping)

MDE HORIZONTAL DESIGN SPECTRA

Project No.

Southern California Rapid Transit District
METRO RAIL PROJECT

Figure No.
A-3

Under some circumstances inelastic response may be calculated by the inelastic response spectra approach (Reference A.4). See Subsection 4.4.3. In such cases inelastic response spectra may be developed from corresponding elastic response spectra as illustrated in Figure A-4.

4.3.1.3 Time-History Input Motions - Where time-history type of analysis is to be used the District will provide appropriate digitized records in the form of computer tapes or decks for ODE and MDE level events. If a time-history for a magnitude 8 event on the San Andreas fault is required, a special record will be developed by the District. Development of this record will be based on the approach outlined in Reference A.5.

Time-history input motions may be utilized for generating in-structure (floor) response spectra to be used in modal spectral analysis of structure-supported items, or such accelograms may be used for analysis of complex above-grade or aerial structures.

4.3.1.4 Depth Dependence - Analysis utilizing simple one dimensional shear beam models usually predicts attenuation of peak accelerations with depth, especially if there are no abrupt variations in soil stiffness with depth. For a layered profile, some of the layers may be excited by certain frequencies of ground motion. As such, the general trend of attenuation may no longer be valid, and motions may vary from one depth to the other. Based on this and other observations, the design values in 4.3.1 shall be used for all depths of interest for the Metro Rail Project.

4.3.2 In-Structure Response Spectra

In-Structure Response Spectra (ISRS) may be required to define seismic environment for items supported on major structural systems or components. In such cases the Engineer shall develop ISRS as specified in Subsection 4.4.5. Where ground motions are required as input for generation of ISRS, design response spectra or time-histories defined in Subsection 4.3.1 shall be used as input.

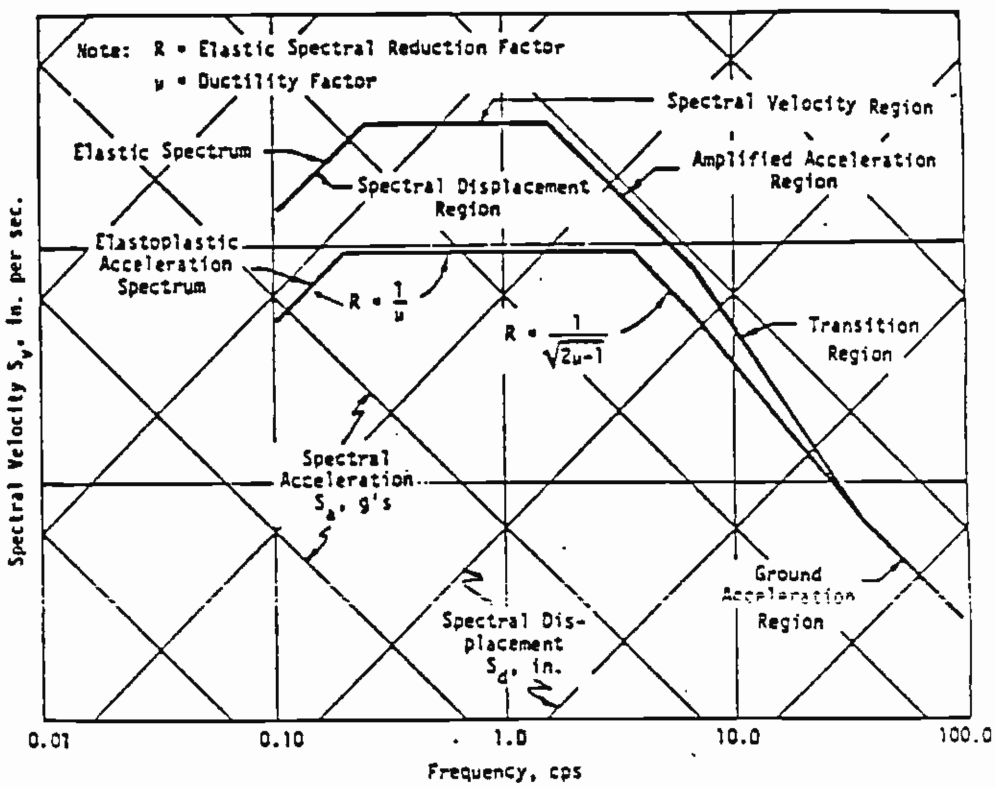
4.3.3 Special Loading Considerations

There are several special primary and secondary seismic loading conditions which occur and are to be defined for structural design purposes. Such conditions include, but are not limited to, the following:

(a) Traveling wave related effects.

(b) Relative displacements which may occur at fault crossings or those related to seismically induced slope failures.

(c) Increased soil pressure due to soil-structure interaction or stability related effects such as liquefaction.



(Reference. A.4)

RESPONSE SPECTRUM MODIFICATION FOR DUCTILITY

Project No.

Southern California Rapid Transit District
METRO RAIL PROJECT

Figure No.
A-4

(d) Abrupt changes in soil stiffness which may induce forces or stresses into below-ground structures.

With regard to traveling wave effects, the design ground motions given in Table A-2 are to be used for calculating applied loadings and induced stresses. The use of design motions for such calculations is defined in Subsection 4.4.6 on analysis of below-ground structures.

Relative displacements due to fault slip which must be accounted for in design at fault crossings are provided in Table A-3. Design for fault displacement is required only for MDE conditions.

The design fault parameters (Table A-3) are based on current information on the geologic characteristics of the two faults, the seismological character of a magnitude 6.5 and 7 event, and fault slip information obtained from the Kern County earthquake of 1952 and the San Fernando earthquake of 1971. However, as additional data is collected on the Hollywood and Malibu-Santa Monica Fault Systems during the later phases of this project, the above values will be reviewed and revised as appropriate.

Design approaches which may be used to account for special loading conditions, for example, conditions "a" through "d" cited above, are discussed in Sections 4.4 and 4.5.

TABLE A-3

DESIGN FAULT PARAMETERS

PARAMETER	SANTA MONICA	HOLLYWOOD
MDE MAGNITUDE	7	6.5
Total slip (average)	2.0 m	1.5 m
dip slip (vertical)	1.5 m	1.12 m
strike slip (horizontal)	1.3 m	1.0 m
Dip of fault plane (assumed)	60° N	45° N
Angle of intersection of fault plane and route alignment	65°	90°
Horizontal crustal shortening normal to strike	1.5 m	1.0 m
Width of zone of faulting or disruption	20 - 600 m	20 - 300 m

4.4 Determination of Seismic Response

This section covers the requirements for determining the seismic response for SC A items under criteria earthquake loading (input) conditions. As previously noted, Section 4.3 provides input criteria, while Section 4.5 covers response limits and acceptance criteria under various loading conditions and combinations including seismic.

4.4.1 Selection of Analysis Type

This subsection provides criteria for selection of the method for determination of dynamic response of structural systems. In this case analysis as described in Subsections 4.4.2 through 4.4.5 is for earthquake-generated ground or support structure motions. Criteria for analysis of traveling wave effects and other special earthquake loading conditions are covered in Subsection 4.4.6.

Three general dynamic analysis procedures are described:

(1) Simplified Dynamic Analysis (SDA) - Subsection 4.4.2
(2) Modal Spectral Analysis (MSA) - Subsection 4.4.3
(3) Time History Response Analysis (TRA) - Subsection 4.4.4.

The following paragraphs provide conditions to be fulfilled for selection of one of the three dynamic analysis procedures under specific circumstances. These conditions are summarized in Table A-4.

(a) A Simplified Dynamic Analysis (SDA) may be utilized when the dynamic response of the structure is mainly in one dynamic mode for the given direction of ground motion considered, and effectively no coupling occurs between responses in each of the three ground motion input directions. In order to fulfill this criteria it must be effectively demonstrated that at least 90% of the structure mass participates in the primary response mode for a given direction of excitation. Also, in order to qualify for SDA, the structure must have a relatively simple framing system and be regular. See "Regularity Class No. 1", Table A-5.

(b) Regardless of other factors, dynamic response of a structure may be analyzed by SDA procedures if the structure is rigid. A structure is considered to be rigid if its first mode natural period is equal to or less than 0.05 seconds.

(c) The dynamic response of SC A structures that do not qualify for SDA, as described above, must be analyzed by MSA or TRA methods. MSA is acceptable for all cases in which TRA is not required. Non-rigid structures which are irregular, as determined by Table A-5, require TRA analysis.

4.4.2 The Simplified Dynamic Analysis Procedure (SDA) - Peak dynamic force (F) on the item or structure shall be determined by the following formula considering a given orthogonal direction of input motion:

$$F = 1.5\ CW \qquad (a)$$

In this case C is the peak response acceleration (in units of "g") determined from the design response spectra defining earthquake motions for the given input direction. The first or dominant mode natural period of the structure shall be calculated and used to obtain the peak acceleration value from the design response spectra. W is the total weight of the structure which may participate in the structural response for the direction considered.

Where the first or dominant mode of the structure has a natural frequency of 20 hertz or more, the force (F) may be estimated on the basis of the following formula:

$$F = 1.2\ CW \qquad (b)$$

In this case the meaning of symbols is the same for formula (a) above.

TABLE A-4

SELECTION OF DYNAMIC ANALYSIS PROCEDURE

DYNAMIC ANALYSIS PROCEDURE	PROCEDURE SELECTION REQUIREMENTS FOR ANALYSIS IN PARTICULAR RESPONSE DIRECTION
SIMPLIFIED DYNAMIC (SDA)*	1. Structural framing system must be relatively simple. 2. Structure must be regular (see Table A-5). 3. At least 90% of mass must participate in the dynamic response mode considered. 4. No significant coupling between response in the three orthogonal input directions occurs.
MODAL SPECTRAL DYNAMIC (MSA)	May use in all cases where time-history response analysis (TRA) is not required
TIME-HISTORY RESPONSE (TRA)	Must use if structure is irregular and not rigid

*SDA is applicable in all cases where the structure is rigid, and, in this case, the four requirements tabulated are waived. See text for definition.

TABLE A-5

STRUCTURAL REGULARITY CLASSIFICATION

REGULARITY		IRREGULARITY FEATURES[a]				
No.	Class	Mass Ratio[b] Level to Level (floor to floor)	Stiffness Ratio[b] Between Vertical Sections (story to story)	Continuity of[c] Lateral Force Resisting System	Horizontal[de] Effective Eccentricity	Projection[f] Beyond Vertical Resisting System
1	Regular	Within 20%	Within 20%	Continuous	Within 10%	Within D/5
2	Slightly Irregular	Within 50%	Within 25%	Continuous	Within 15%	Within D/4
3	Irregular	Over 50%	Over 25%	Non-continuous	Over 15%	Over D/4

[a] A structure shall be assigned the highest Regularity Classification number for which it has one or more qualifying irregular features.

[b] The mass and stiffness ratios refer to the presence of a decrease or increase in one of these quantities in a story of a structure relative to a story immediately above.

[c] A continuous Lateral Force Resisting System is defined as one with no changes of basic material or framing system, without offsets or changes in the earthquake load path, and with no change in basic geometry.

[d] The effective eccentricity at any level is the total torsional moment divided by the total shear at that level. For structures to have Regularity Class Nos. 1 and 2, the major lateral load resisting elements must be parallel to the major orthogonal axes and the horizontal eccentricity between the center of rigidity and the geometric center at any level shall be no greater than 15%.

[e] Percentage of eccentricity shall be based on the lateral force resisting system dimension perpendicular to the direction of applied force.

[f] "D" is defined as the minimum horizontal building or structure dimension at the level under consideration. Adequate diaphragm stiffness shall be provided by design for each level including that for projecting portions.

In either of the above cases, the structure may be made up of a number of masses at different locations. When this occurs the total force F shall be distributed proportionately at each mass location for purposes of static analysis to determine responses such as member forces, stresses, deformation or deflection.

Damping values, and ductility factors if appropriate, shall be selected for defining the design response spectra as specified for modal spectral analysis in Subsection 4.4.3 below.

Input motions and corresponding structural response values for the structure or any component shall be considered to occur non-concurrently for each major input direction. Structural response on the structure as a whole and on each component as determined from the procedure described herein shall not be combined with response to other input directions. Design of each component shall be on a worst case basis considering all three orthogonal input directions and resulting response.

The factor of 1.5 in the force formula is provided to conservatively account for effects not otherwise included in the response. These effects include directional coupling, higher mode response and the possible unconservatisms relative to period computations.

4.4.3 The Modal Spectral Response Analysis Procedure (MSA)

This section describes criteria for performing seismic structural response analysis by the modal spectral method. As seen from the selection rules given in Subsection 4.4.1, Modal Spectral Analysis (MSA) is the norm or standard for determining seismic response of SC A items. Simplified Dynamic Analysis (SDA) is an exception for determining response of relatively simple structures, and Time-History Response Analysis (TRA) is an exception for highly complex and irregular structures.

The general method of Modal Spectral Analysis required is described in detail in standard texts on structural dynamics; for example, see Reference A.6. Generally, any computer approach used shall incorporate finite element methods utilizing the matrix-displacement method of structural mechanics. Examples of acceptable computer codes are: SAP IV, NASTRAN, ANSYS, STRUDL, STARDYNE and EASE 2. See References A.7 through A.12, respectively. These and other similar computer codes, meeting the requirements given in this section, may be utilized for performing MSA.

Design response spectra representing seismic ground motions shall be as described in Section 4.3. Input for uncoupled structure supported items to be analyzed shall be developed as described in Subsection 4.4.5. Also, see Paragraphs 4.4.3.2 and 4.4.3.3 below.

4.4.3.1 Mathematical Modeling - The extent and detail of mathematical models shall be consistent with obtaining realistic structural response of items to be analyzed within an engineering degree of accuracy.

Mathematical modeling of items shall be conducted to the detail required to assure obtaining the actual response and consistent with the method of analysis being used. For dynamic analysis the mathematical model shall be, as a minimum, a lumped-mass system interconnected by elastic elements. Modal damping may be assumed in the case of damped structural systems and/or components where the damping level does not exceed 10% of critical.

The models must adequately represent the physical characteristics of structures, systems, and components and their corresponding response to seismic excitations. Where it is difficult to model various structures, systems, and components, parametric studies may be required to determine sensitivity of the model to various parameter changes; e.g., mass, stiffness, material properties, etc. Upgrading must then be made to reflect the more accurate parameter representation as determined by the studies.

All physically connected structures, systems and components shall be represented as a combined single mathematical model unless such connected structures, systems and components are permitted to be uncoupled according to Paragraph 4.4.3.3. When uncoupling is justified, the subdivided structures, systems and components shall be modeled in a consistent manner. When structures, systems, and components are subdivided and, as a result, become supported structures, care must be taken in providing the input motion that is representative of the seismic response of the supporting structure. Refer to Subsection 4.4.5 and also Paragraph 4.4.3.2.

For efficient modeling, geometric, mass and reflective symmetry may be utilized to reduce the number of degrees-of-freedom; however, care must be taken to assure that significant translational and rotational degrees-of-freedom are considered at mass points. In addition, consideration must be given to the coupling effects which may occur between the translational and rotational degrees-of-freedom where the center of mass and center of resistance (for either torsional and bending effects) do not coincide.

At foundations or points of support, the rocking degrees-of-freedom should be considered in the mathematical model. Although, in some cases such degrees-of-freedom are insignificant, care should be utilized to justify their elimination.

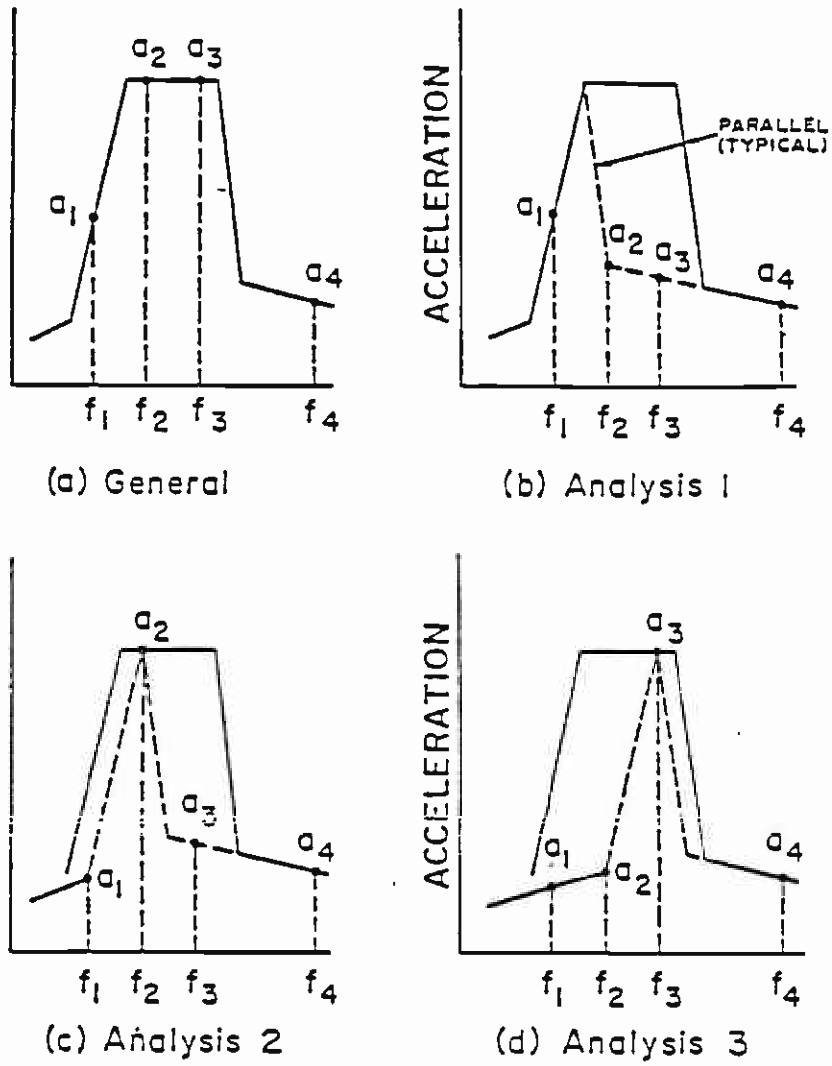
Discontinuities that may exist in a structure, system, or component (e.g., drastic changes in stiffness, gaps, or clearances) that become part of the mathematical model, may require special consideration. It may be necessary to treat gap or clearance discontinuities as non-linearities, and such discontinuities may be subject to impact forces. An appropriate mathematical procedure for representing the response of such gaps or clearances shall be used in determining the impact forces for design purposes. In addition, at points of rapid changes in stiffness, attention should be focused on stress risers.

When modeling equipment, the mathematical model should represent the equipment in its operational mode if it must remain in operation to maintain its required function.

4.4.3.2 Special Considerations in Modeling Supported Structures - The following special considerations are applicable to items supported by other structures rather than being directly supported on ground.

Where a supported item has two or more response frequencies that exist within the broadened resonant frequency band of the supporting point response spectrum (Subsection 4.4.5), the spectrum may be modified in the analysis to prevent unnecessary conservatism. Since the supporting structure, system, or component can have only one resonant frequency, the broadened spectrum is modified such that its peak corresponds to one of the supported substructures, subsystem, or subcomponent frequencies within the broadened range as depicted in Figure A-5 (Reference A.13). The supported substructure, subsystem, or subcomponent is analyzed using the supporting spectrum modified as shown once for each frequency in the broadened band. For example: if three frequencies of the supported substructure, subsystem, or subcomponent were in the broadened band, there would be three analyses, and the analysis producing the largest total response would be used for the design.

For the condition where a substructure, subsystem, or subcomponent is supported by more than one supporting structure, system, or component there will be differing response spectra at various support points. These support point spectra shall



Reference A.13)

USE OF IN - STRUCTURE RESPONSE SPECTRA

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be superimposed on each other, and the design spectrum for the substructure, subsystem, or subcomponent taken as the upper bound envelope of the support point spectra considered.

4.4.3.3 Conditions for Uncoupling of Structural Models - All physically connected structures, systems, and components are coupled to some degree and should be modeled accordingly. However, for purposes of simplicity and economy in many cases it is desirable and sufficiently accurate to separate models of structural systems into two or more individual parts. Models of structural systems may be uncoupled according to the following general guidelines.

There are two distinct types of coupling conditions:

(1) where structures, systems, or components are coupled together but supported independently, the coupled point may be considered as additional support point, and

(2) where the structures, systems, or components are physically coupled and physically support one or another through the coupling point, one being the primary support.

All other coupling conditions are combinations of these two.

Basically, uncoupling can be justified where the model of each subsystem or subcomponent is developed to account for interaction effects at interfaces; or where it is shown that the dynamic response of subsystems and subcomponents is independent as modeled.

Table A-6 provides conservative guidelines which are to be used as a basis for uncoupling models of structural systems. This table was developed on the basis of past studies which have been conducted to examine the effects of mass and stiffness relationships between various systems and their resulting interaction. See Reference A.14.

4.4.3.4 Number and Combination of Modes - In performing dynamic analysis using modal spectral methods, sufficient modes shall be included to accurately represent the response of the structure. To assure this accurate response determination, in general, the number of natural modes included in calculating the reponse shall be such that at least 90% of the modal mass is accounted for within the modes considered for a given orthogonal response direction.

In order to calculate the maximum of any response quantity, the responses from each normal mode shall be combined using "the square root of the sum of the squares" approach. For closely spaced modes - modes whose frequencies are within 10% of each other - the response shall be combined using "the sum of absolute values" criterion. The combined effects of the closely spaced modes shall then be combined with all other modes using the "the square root of the sum of the square" approach.

TABLE A-6

GUIDELINES FOR UNCOUPLING[a]

CASE No.	f_e/f_s[b]	M_e/M_s[c]	SUPPORT[d] CONDITION
1A	≤ 0.5	≤ 0.20	U
2A	0.5 to 2.0	<0.001[e]	U
3A	≥ 2.0	≤ 0.20	U
1B	≤ 0.5	>0.20	C
2B	0.5 to 2.0	>0.001	C
3B	≥ 2.0	>0.20	C

[a] Based on Reference A.14.

[b] f_e and f_s = natural frequencies of significant modes of the supported and supporting items, respectively. f_e is the particular significant modal frequency which is closest in value to f_s.

[c] M_e and M_s = total mass of the supported and supporting items, respectively.

[d] Modeling Condition U (Uncoupled):

Supporting Role - Model as uncoupled, neglecting effects to supporting item, but including mass of supported item.

Supported Role - Model as uncoupled, using as input at support points the dynamic response calculated for supporting item.

Modeling Condition C (Coupled):

Model as coupled (total) system or use acceptable procedure to account for interaction effects.

[e] Mass of supported item may be neglected.

The above approach for combining of modes is described in additional detail in Regulatory Guide 1.92 (Reference A.15). Two acceptable alternate methods for combining closely spaced modes are also described in Reference A.15.

4.4.3.5 Directional Considerations - Dynamic analysis by the MSA shall be performed considering the seismic excitation in the two orthogonal horizontal axes and the vertical axis that coincide most nearly with the principal axes of the structural model.

If the analyses are performed either concurrently or independently in each of the two horizontal directions and the vertical direction, the resulting maximum response (for example: maximum displacement, acceleration, moment, stress, etc.) for any element or at any point in any direction obtained for each of the three directions shall be combined individually by the square root of the sum of the squares. When using the independent direction input analysis approach, care must be exercised to assure that significant coupling effects are not unjustly neglected.

As an alternate method of combining responses induced by three orthogonal components of earthquake motion acting concurrently on a structure or element, the responses to seismic input consisting of 100% of the maximum in any given principal direction shall be directly superimposed with corresponding responses induced by 40% of the maximum input for each of the other two orthogonal directions. In this method, the peak response considered must be examined to assure worst case from all three possible (100-40-40) input conditions. Response components which are contributed by input from a given direction and demonstrated to be less than 5% of the total combined response may be neglected.

4.4.3.6 Torsional Effects - Models shall be developed to account for torsional effects due to significant eccentricity between centers of mass and rigidity in analyzing seismic response of the structure. Analysis shall account for traveling wave effects (i.e., accidental torsion), for structures with relatively large extent and plan dimension aspect ratios.

For buildings, building-like structures, towers, and platforms, an equivalent static accidental torsional moment shall be added to torsional moments related to actual eccentricity for inclusion in design forces. Torsional moments shall be distributed to structural resisting elements at each level of the structure by standard analytical methods. The accidental torsion added at each level shall be the product of the design seismic acceleration at the given level and an assumed eccentricity of the story mass and actual center of rigidity at the level. The assumed eccentricity shall be taken as 5% of the largest building plan dimension at the given level.

4.4.3.7 Selection of Damping Values - Damping values for use in MSA analysis are provided in Table A-7. These conservative values are cited by material, type of construction, and stress

DAMPING VALUES

STRUCTURE, SYSTEM OR COMPONENT	DAMPING VALUES[a,b] Operating Design Earthquake Stresses Ranging From 1/2 Yield To Working Stress (2/3 Yield) (percent)	DAMPING VALUES[a,b] Maximum Design Earthquake Stresses At or Near (4/5) Yield To Full Yield Stress (percent)
Equipment and Piping Systems	1 to 2	2 to 3
Steel Structures - Welded	2 to 3	4 to 7
- Bolted	4 to 7	7 to 10
Reinforced Concrete Structures	3 to 5	7 to 10
Prestressed Concrete Structures	2 to 3	5 to 10
Wood Structures - Nailed or Bolted	5 to 7	10 to 15
Foundation Systems for Structures[c] on rock - $C_s \geq 6000$ fps	2	5
on firm soil - $C_s \geq 2000$ fps	5	7
on soft soil - $C_s < 2000$ fps	7	10
Tanks Sloshing Response (water or oil)	0.1	0.1
Overall Tank Response[d]	3	5

[a] Damping values represent percent of critical damping.

[b] Where a range of values is specified, the engineer should select the most appropriate value within that range for the item(s) being analyzed. If deemed necessary, damping values less than those specified may be used. Refer to discussion in text on relative conservatism.

[c] Where C_s = Shear Wave velocity in the material. Values listed are rock/soil material damping values.

[d] For tank structure and liquid moving with tank.

levels, and are based on References A.16, A.17 and A.18. In particular, the lower and upper values of damping cited for each design earthquake level in Table A-7 correspond respectively to the lower and upper stress levels cited in each heading. The lower damping levels of each pair are highly conservative design values, nearly lower bounds. The upper levels of each pair are average or moderately conservative design values.

In lieu of the values given, values based on the results of rigorous testing or analysis may be used where proper documentation is provided. Similarly, higher elastic response damping ratios may be used, provided justification is shown that such higher values are appropriate. For example, consideration of the effects of radiation damping may allow large damping values to be used for foundation materials. For those structures, systems, and components which are subjected only to low stress levels, appropriately reduced damping values should be used when computing displacements. For inelastic analysis, the higher damping values of Table A-7 should be used. Further, energy losses should be considered due to inelastic deformation. The values shown in Table A-7 are equivalent viscous damping ratios and are given as percent of critical damping. For special structures, systems, and components that have been designed to have high structural damping values, testing should be conducted in the frequency range of the earthquake to verify such damping. Soil material damping values shown are conservative guidelines for the overall structural rocking mode. Soil radiation damping values to be used may be calculated by standard procedures considering the mass, stiffness and geometry of the particular foundation to be designed. See Paragraph 4.4.3.9.

4.4.3.8 Analysis Considerations for Nonlinear Response - The design criteria given permits structures in Seismic Class A-1 to perform in the inelastic range under MDE loading conditions. Special design considerations must be followed in this case as specified in Section 4.5

The general approach in the Metro Rail System for evaluating nonlinear structural/seismic response under MDE input motions is by the inelastic modal spectral method developed by Newmark and Hall (Reference A.19).

This approach is applicable to simple and moderately complex structures. For irregular and highly complex structures, inelastic response must be determined by direct integration Time-History Response Analysis (TRA) as delineated in Subsection 4.4.4.

The inelastic MSA is an approximate method whose use should be applied judiciously. Guidelines for use of this method and appropriate limitations for its use are treated in References A.20 and A.21, respectively.

Nonlinear design spectra for inelastic analysis are provided by criteria given in Section 4.3. Appropriate damping values are cited in Paragraph 4.4.3.7. Limiting values of ductility which may be used are given in Table A-8. Where values of ductility above 2 are used, analytical studies demonstrating that the ductility capacity of the system is at least as great as that assumed shall be performed and provided to the District for review and approval.

4.4.3.9 Soil-Structure Interaction (SSI) - For the analysis and design of Seismic Class A items supported by ground, the effects of soil-structure interaction (foundation compliance effects) shall be evaluated. The evaluation may be made solely on the basis of engineering judgement depending on the type of structure and foundation system; in more complex cases, the evaluation shall be based on the guidelines or analysis procedures discussed or referenced in this paragraph.

Where the structures, systems, or components are relatively lightweight and their foundations (concrete supports) are shallow, soil-structure interaction may be judged to be negligible. Where soil-structure interaction is negligible, the design response spectra specified in Section 4.3 shall be used directly as input.

More detailed guidelines for determining the necessity of including soil-structure interaction in the seismic analysis of a structure are given in Reference A.22. In addition, the methods outlined below may be used expeditiously to assess the need for further detailed consideration of SSI effects in analytical models.

Where soil-structure interaction effects must be included in the analysis, the basic approach utilizes lumped parameter analysis models based on elastic half-space theory. This approach is described in detail in Chapters 7 and 10 of Reference A.23. In this method, the foundation is considered to be essentially rigid, and equivalent soil springs and dash-pots are developed for each mode of rigid body response to model the effective soil compliance and energy dissipation. The footings are considered to be resting on the surface of a half space. The effects of embedment may be assessed in an approximate manner. See, for example, References A.24, A.25, and A.26. Finite element analysis may be required to assess SSI effects in special cases.

A general model for dynamic analysis of structural framing systems, to include soil-structure interaction effects, consists of a planar frame model which includes one principal horizontal and the vertical direction of motion. Each column footing location will include up to three soil springs: one vertical, one horizontal, and one rotational spring. Parametric studies of dynamic response with a range of soil spring and damping values are to be made, and where the response is insensitive to variations in soil properties, the foundation

TABLE A-8

MAXIMUM SYSTEM DUCTILITY VALUES*
FOR SC A-1 ITEMS

ITEM	MAXIMUM DUCTILITY VALUE
Equipment	2.0
Piping	3.0
Steel Structural Systems Rigid Frames or Individual Elements Loaded Primarily in Tension or Flexure	5.0
Braced Frames or Individual Elements Loaded Primarily in Compression	2.0
Concrete Structural Systems Ductile Rigid Frames or Individual Elements Loaded Primarily in Flexure	4.0
Ductile Detailed Shear Walls	2.0

* The ductility values specified are the maximum values that may be used under any condition. It may be more appropriate to use values less than those specified, depending on the details of the item under consideration. See discussion in text.

Ductility values of 1.0 only are permitted for SC A-2 items.

may be considered fixed in the given direction or directions of response; that is to say, SSI effects for this direction are negligible. Models shall include all contributing mass and inertia of the footing and supported structure or equipment. No equivalent soil mass is to be included in the analysis (Reference A.23).

Where soil-structure interaction is shown to have a significant impact on the structure or equipment design, verification analyses may be performed by finite element methods. In this method, a two-dimensional (one horizontal and vertical) analysis representation which includes a finite element model of the equipment or structure and foundation and the adjacent soil shall be developed. Similar models may be used to assess special effects, such as those from wave propagation on long foundations or buried structures and local soil effects on free field earthquake motions.

For tunnel and basement floors and walls, interaction effects are to be included generally by adding a seismic surcharge, that is an increment of dynamic soil pressure, when designing walls and floors for lateral and vertical loadings. The dynamic surcharge loading is discussed in Paragraph 4.5.4.5.

Figure A-6 indicates the spring constants which correspond to various rigid body modes of response for footings resting on half-spaces. These constants are independent of footing-soil system frequency and depend only on footing dimensions, the soil shear modulus, G, and Poisson's ratio, υ. Representative values for the upper soil strata are given in Table A-9.

TABLE A-9

MATERIAL PROPERTIES

TUNNELING CONDITION	COMPRESSION WAVE VELOCITY (ft/sec)	POISSON'S RATIO	SHEAR MODULUS-G (psi)	CONSTRAINED MODULUS - E_c (psi)
Soil	*	.3	*	*
Soft Rock	*	.25	*	*
Hard Rock	*	.2	800,000	2,400,000

*To be determined and provided by the District.

Radiation damping for the analysis models may be estimated using formulas developed for the half-space theory (Reference A.23) given in Table A-10. In these formulas, ρ is the soil mass density and m is the supported mass of foundation and

SPRING CONSTANTS FOR RIGID CIRCULAR FOOTING RESTING ON ELASTIC HALF-SPACE

MOTION	SPRING CONSTANT	REFERENCE *
Vertical	$k_z = \frac{4Gr_o}{1-\nu}$	Timoshenko and Goodier (1951)
Horizontal	$k_x = \frac{32(1-\nu)Gr_o}{7-8\nu}$	Bycroft (1956)
Rocking	$k_\psi = \frac{8Gr_o^3}{3(1-\nu)}$	Borowicka (1943)
Torsion	$k_\theta = \frac{16}{3}Gr_o^3$	Reissner and Sagoci (1944)

(NOTE. $G = \frac{E}{2(1+\nu)}$ = Shear Modulus)

ν = Poisson's Ratio

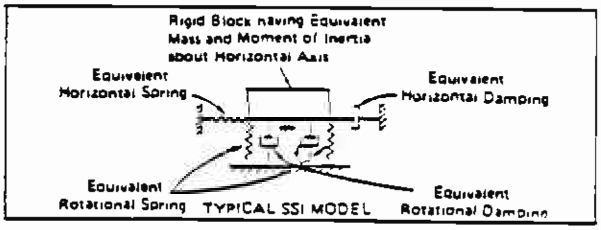
*See Table 10-13 Ref. A. 23

SPRING CONSTANTS FOR RIGID RECTANGULAR FOOTING RESTING ON ELASTIC HALF-SPACE

MOTION	SPRING CONSTANT	REFERENCE *
Vertical	$k_z = \frac{G}{1-\nu}\beta_z\sqrt{4cd}$	Barkan (1962)
Horizontal	$K_x = 4(1+\nu)G\beta_x\sqrt{cd}$	Barkan (1962)
Rocking	$k_\psi = \frac{G}{1-\nu}\beta_\psi 8cd^2$	Gorbunov-Possadov (1961)

NOTE. Values for β_z, β_x, and β_ψ are given in the figure for various values of d/c. d is always perpendicular to axis of rocking.

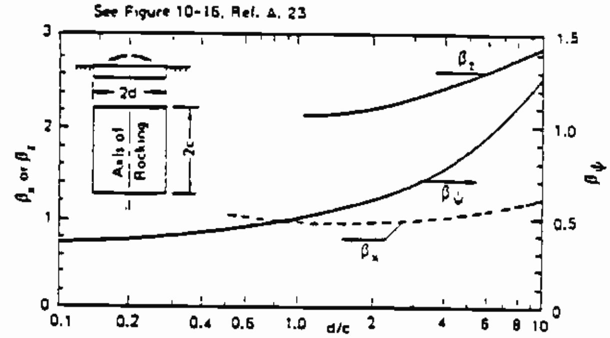
*See Table 12-14 Ref. A. 23



TYPICAL SSI MODEL

COEFFICIENTS β_z, β_x, AND β_ψ FOR RECTANGULAR FOOTINGS (After Whitman and Richart, 1967)

See Figure 10-16, Ref. A. 23



(From Vibrations of Soil and Foundations by Richart, Hall and Woods, Ref. A.23)

SSI SPRING CONSTANTS

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TABLE A-10

SOIL DAMPING
(After Richart, Hall, and Woods; Ref. A.23)

EQUIVALENT RADIATION DAMPING FOR RIGID CIRCULAR FOOTINGS*		
MODE OF VIBRATION	MASS (or INERTIA) RATIO	DAMPING RATIO D
Vertical	$B_z = \frac{(1-\nu)}{4} \frac{m}{\rho r_o^3}$	$D_z = \frac{0.425}{\sqrt{B_z}}$
Sliding	$B_x = \frac{(7-8\nu)}{32(1-\nu)} \frac{m}{\rho r_o^3}$	$D_x = \frac{0.288}{\sqrt{B_x}}$
Rocking	$B_\psi = \frac{3(1-\nu)}{8} \frac{I_\psi}{\rho r_o^5}$	$D_\psi = \frac{0.15}{(1 + B_\psi)\sqrt{B_\psi}}$
Torsional	$B_\Theta = \frac{I_\Theta}{\rho r_o{}^5}$	$D_\Theta = \frac{0.50}{1 + 2B_\Theta}$

*or for rectangular footings with the equivalent radius, r_o as follows:

For translation: $r_o = \sqrt{\frac{4cd}{\pi}}$

For rocking: $r_o = \sqrt[4]{\frac{16cd^3}{3\pi}}$

For torsion: $r_o = \sqrt[4]{\frac{16cd(c^2 + d^2)}{6\pi}}$

in which

2c = width of the foundation (along axis of rotation for the case of rocking)
2d = length of the foundation (in the plane of rotation for rocking).

<u>NOTE:</u> ν = Poisson's Ratio

m = contributing mass of foundation and structure or equipment

I = mass moment of inertia about appropriate axis of foundation and structure or equipment

ρ = mass density of soil

structure or equipment. The B factors are so-called mass ratios for each mode of response. The parameter r_o is the radius of a circular footing. Equivalent radii (r_o) for rectangular footings are developed based on the formulas also provided in Table A-10. Note that the equivalent radii are different for each mode of response.

The soil radiation damping values calculated as described above are based on half-space models considering motions (energy source) generated at or adjacent to the foundation under consideration. For seismic disturbances where energy is supplied somewhat continuously from an external source, the effect of radiation damping must be included in the model.

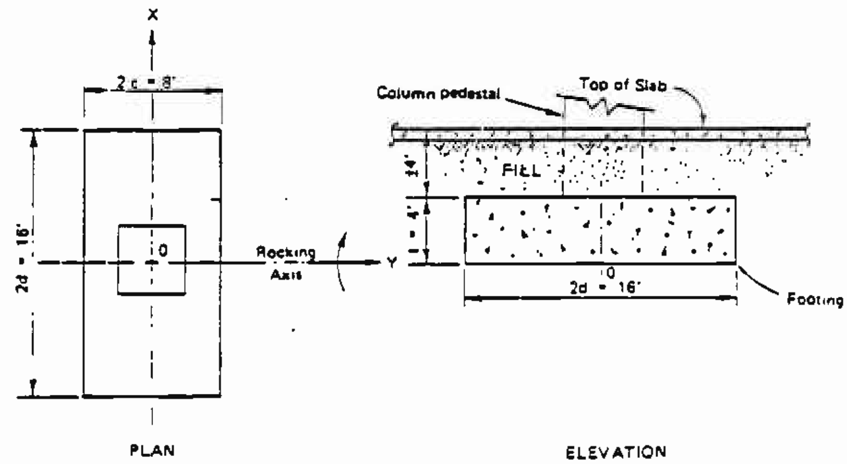
Damping related to SSI should also include the effect of soil material damping. Conservative or lower bound material damping values for SSI effects are listed in Table A-7. Overall damping values for seismic analysis of project structures and systems will be conservatively estimated between the values estimated for SSI effects and the system structural damping. It is noted that structural damping in itself has the lowest value, as SSI effects are not included therein.

A typical structure footing is illustrated in Figure A-7. In seismic analysis models including SSI springs and dashpots, the vertical and horizontal springs generally provide for SSI coupled horizontal and rocking response. Rotational springs can be included to provide a measure of column base fixity under dynamic loading conditions.

In considering SSI effects the springs obtained from the methods outlined above may be introduced directly into the dynamic analysis model. Where the stiffness of the soil spring is high relative to the structure (five or more times as stiff) for a given mode or direction, SSI effects can be neglected and the spring omitted from the model.

Damping can be modeled by a discrete dashpot element or modification to the overall modal damping in the structural model. The values given relate to discrete damping elements. Soil damping effects generally will not induce large damping into the overall structural system. Large soil damping values also indicate when SSI effects can be neglected and structures assumed fixed at soil interfaces. Figure A-8 provides correction factors for soil spring and damping values as a function of foundation embedment. Also, suggested upper bounds to calculated values of soil damping are tabulated.

4.4.3.10 <u>Consideration of Relative Displacements</u> - Structure supported items, especially distributive systems such as piping and conduit, may be subjected to relative support displacements. Relative support displacements are determined in the MSA of the supporting structure. Determination of response effects on items to relative support displacements shall be made by



FOOTING ARRANGEMENT

PROPERTIES

Unit Weight of Soil	$\gamma = \rho g = 120$ pcf
Unit Weight of Concrete	$\gamma_c = \rho_c g = 150$ pcf
Mass Density = ρ	
Acceleration of Gravity	$g = 32.2\ Ft/_{Sec}2$
Total Weight of Foundation	$gm_{st} + gm_s + gm_f = gm = 238240$ lb.
Weight of Structure	$gm_{st} = 100000$ lb.
Weight of Soil Above Footing	$gm_s = 61440$ lb.
Weight of Footing	$gm_f = 75800$ lb.
Length of Footing (Direction of Rocking)	$2d = 16$ Ft
Width of Footing	$2c = 8$Ft
Footing Thickness	$t = 4$ Ft
Mass Moment of Inertia of Foundation System About Point 0 = $I_{FTG} + t^2 (m_s + m_{st})$	$I_{\psi} = 143819$ lb.-Sec^2-Ft^{-1}
Mass Moment of Inertia of Footing About Point 0	$I_{FTG} = 63600$ lb.-Sec^2-Ft^{-1}
$I_{FTG} = \frac{1}{12} m_f \left[(2d)^2 + t^2\right] + m_f \left(\frac{t}{2}\right)^2$	
Mass Transfer to Point 0, Mass of Structure and Soil Considered Concentrated at Top of Footing, $I = t^2 (m_s + m_{st})$	$I = 80219$ lb-Sec^2-Ft^{-1}

SAMPLE STRUCTURE FOOTING - SSI PARAMETERS

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SPRING CONSTANT MULTIPLIER (N)

MOTION	MULTIPLIER "N" (embedment factor)
VERTICAL	$N_z = 1+0.6(1-\nu)\ (H/r_o)$
HORIZONTAL	$N_x = 1+0.55(2-\nu)\ (H/r_o)$
ROCKING	$N_\psi = 1+1.2(1-\nu)\ [(H/r_o)+0.2(2-\nu)]\ (H/r_o)^3$
TORSIONAL	NONE AVAILABLE

where

H = depth of embedment (use 2/3 of actual)
r_o = equivalent radius
ν = Poisson ratio

DAMPING COEFFICIENT MULTIPLIER (S)

MOTION	MULTIPLIER "S" (embedment factor)
VERTICAL	$S_z = [1+1.9(1-\nu)\ ({}^H/r_o)] \big/ \sqrt{N_z}$
HORIZONTAL	$S_x = [1+1.9(2-\nu)\ ({}^H/r_o)] \big/ \sqrt{N_x}$
ROCKING	$S_\psi = [1+0.7(1-\nu)\ ({}^H/r_o)+0.6(2-\nu)\ ({}^H/r_o)^3] \big/ \sqrt{N_\psi}$

LIMITING VALUES OF TOTAL DAMPING

Mode	Maximum Total Damping (1) for Analysis
Vertical	0.95
Translation	0.60
Rocking	0.40
Torsion	0.20

(1) Damping expressed as fraction of critical

(Reference A.24)

CORRECTIONS TO SSI PARAMETERS FOR EMBEDMENT

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standard static structural analysis methods with peak relative displacements as input. The inclusion of this effect with other loading conditions, including dynamic vibratory response, shall be as covered under Section 4.5.

4.4.3.11 Documentation Requirement for Analysis - All design calculations including manual and computer analysis for determining seismic response and additional calculations to verify that response levels are within acceptable limits must be performed and independently checked by responsible design professionals appropriately experienced in structural and earthquake engineering.

The calculations shall document all key results, formulas utilized, related assumptions, and criteria used. Sketches shall be provided to depict key dimensions and details for each structure. Calculations shall be sufficiently complete and detailed to be clear when subjected to an independent review by SCRTD engineers or consultants. A complete reproducible copy of engineering calculations shall be submitted to the District for each structure when its design is finalized and ready for construction.

Where computer calculations are performed, copies of computer input and output, descriptions of computational methods, and verification of program accuracy shall be included in the analysis documentation. A dimensioned pictorial description of computer models shall also be included. Descripton of computer programs and verification of their accuracy shall be accomplished by reference to an unmodified published and verified program (e.g., SAP or STRUDL) where used, or a complete description and verification shall be provided for unpublished computer programs were used.

4.4 Time-History Response Analysis Procedure (TRA)

Except as otherwise provided herein, requirements for TRA shall be the same as those for MSA as given in Subsection 4.4.3.

TRA is only required for design or review of items having relatively complex response behavior which can only be adequately calculated in this manner. Such complex response behavior may result from one or more of the following characteristics of a given structure:

(a) complexity of structural geometry

(b) high degree of structural irregularity in geometry, load path, or eccentricity of mass and stiffness

(c) significant nonlinear geometric effects due to large deformation or displacements

(d) nonlinear material behavior which cannot be reasonably assessed by more approximate means.

Time-history modal response methods may be utilized for elastic analysis of irregular structures. Where detailed nonlinear response is required, the direct-integration-of-equations-of-motion approach must be utilized.

4.4.4.1 Approval - As TRA is generally costly and sensitive to various input, modeling, and material criteria, models and approaches for calculating response in this manner shall be submitted to the District or their representatives for approval prior to initiating the analyses.

4.4.4.2 Modeling and Other Analytical Considerations - In addition to the analysis considerations discussed in Subsection 4.4.3, engineering judgement and care must be applied to limit models to a size consistent with determining response of interest.

Also, judgement as to model geometry, realistic material characteristics and validity of results is more critical for nonlinear analysis. Special care must be taken in selecting appropriate material properties where soils or rock are included in the analytical model.

Time-steps must be selected with respect to model element type and size, input definition and pulse rise times, and response refinement required.

4.4.4.3 Computer Codes - Verified published versions of time-history and/or nonlinear response codes which may be applicable include SAP IV, NASTRAN, ANSYS, NONSAP, MARC, and DRAIN (References A.7, A.8, A.9, A.27, A.28 and A.29, respectively). Documentation and calculations shall conform to provisions cited in Paragraph 4.4.3.11.

4.4.5 Generation of In-Structure Response Spectra - ISRS

Design in-structure response spectra (floor spectra) are required as input for structure supported items such as substructures, equipment or piping. Such ISRS shall be developed by one of two methods: Time-History or Modified Singh. Requirements for the methods are described below.

Raw ISRS developed for locations shall be smoothed and peaks broadened according to procedures described in Regulatory Guide 1.122 (Reference A.30).

In order to limit the number of ISRS required for design or qualification of supported items, it is appropriate to envelope spectra generated from various response point locations to create design ISRS. See also Paragraph 4.4.3.2.

When generating ISRS, care must be taken to include response contribution from all input directions. Combination of components shall be in accordance with Paragraph 4.4.3.5.

In generating ISRS for MDE conditions commensurate adjustments of input or the analysis approach must be made to account for nonlinear support motions.

4.4.5.1 Time-History Generation of ISRS - This method is essentially a modal time-history response analysis with appropriate models to accurately develop time-history response at key support point locations. The analysis shall follow the requirements given in Subsections 4.4.3 and 4.4.4. The support motions are then to be used for generation of raw spectra which are then enveloped, smoothed and broadened to make ISRS (or floor spectra) suitable for design purposes. The generation of raw spectra shall utilize an acceptable response spectrum generation method similar to that used for generation of ground spectra; for example, the Nigam-Jennings method (Reference A.31). ISRS generation and development shall meet the general requirements of the Regulatory Guide (Reference A.30).

4.4.5.2 Generation of ISRS by the Modified Singh Approach - The general Singh approach has been modified and further developed for user convenience and cost effectiveness. See Reference A.32. In this method ISRS are generated directly from design spectra which provide input criteria for supporting structures. Required enveloping, smoothing and broadening must conform to criteria of the Regulatory Guide (Reference A.30).

4.4.6 Criteria for Determining Response of Buried Structures

4.4.6.1 General - Deep buried structures are defined as those with ground cover (above the top of structure) of at least one-half the structure's width or diameter.

Deep buried structures shall be designed for response due to both vibratory inertial loading effects and traveling wave effects. The response for inertial effects shall be determined on the basis of criteria given in Subsections 4.4.1 through 4.4.5. These criteria also apply to determination of inertial loading on shallow-buried and partially buried as well as above ground and aerial structures.

This Subsection covers the response which is related to traveling waves. Such response is applicable to "line" or tunnel-type deep buried structures. These responses are calculated as stresses in structures which conform to the ground seismic wave shapes. Formulas for such peak responses are given in Paragraph 4.4.6.2 below and are based on peak ground motion parameters. Such formulas are developed from one-dimensional wave theory. Stresses induced include those due to overall bending and shear about both horizontal and vertical axes of the line structure as well as longitudinal (axial) components. Design of below grade walls to resist localized lateral earth pressures caused by traveling waves is covered in Paragraph 4.5.4.5.

The formulas given in Paragraph 4.4.6.2 provide conservative estimates of stresses. . The inclusion of soil-structure-interaction (SSI) effects on deep-buried line structures due to traveling waves can, in many cases, reduce stresses calculated by these formulas. When stresses calculated by the formulas have a major impact on the design, parametric studies shall be conducted to assess the significance of SSI effects. Where significant, detailed finite element analyses shall be performed to calculate the traveling wave induced SSI stresses. However, the District may supply simplified formulas during the design phase of the project. These formulas, to approximate the effects of SSI, may be used in some cases in lieu of the detailed finite element analyses.

The combining of responses such as stresses induced by various input sources including vibratory motions and traveling waves are covered under Section 4.5. Material properties which are applicable for use in calculating wave propagation stresses in underground structures are the applicable values for the selected structural materials. In addition, an apparent wave propagation velocity (C) through ground or rock is required for such calculations. C shall be taken to be 3600, 4800, and 15,000 ft/sec for project site soil, soft rock, and hard rock, respectively.

4.4.6.2 <u>Traveling Wave Induced Stresses in Tunnel-Like Structures</u>

Stresses due to traveling seismic waves acting on deep-buried tunnels or other line structures shall be calculated from induced axial forces, shears and moments utilizing the appropriate classical equations of structural mechanics:

$$f_a = P/_A, \; f_v = VQ/It, \; f_b = MC/_I.$$

Refer to any standard text on mechanics of materials, for example Reference A.33.

The stresses induced by propagating waves in tunnels or other linear (line) structures are affected by the type of seismic wave, its direction of propagation relative to the structure, and interaction of these effects. Wave types include compression, Love, Rayleigh and shear. As earthquake-generated shear waves generally are associated with the largest ground particle accelerations and velocities, the stress calculations are based on shear waves. A conservative criterion for the design condition is that induced axial and bending stresses occur concurrently; and that the shear wave horizontal angle of incidence is 45 degrees and zero degrees with the tunnel longitudinal axis for calculating axial (longitudinal) and bending (transverse) stresses, respectively. Refer to the discussion in this regard in Reference A.34. The equations given in the following subparagraphs for determining induced axial forces, shears, and moments are based on the above cited criterion.

The maximum axial force, P, acting on straight sections taken normal to the longitudinal tunnel axis shall be calculated as follows:

$$P = \frac{V_{max}\ AD}{2\ C}$$

with an upper bound limit of $P = f\lambda/4$

where

V_{max} is the maximum horizontal ground particle velocity as given in Subsection 4.3.1

A is the net cross-sectional area of the tunnel

C is the apparent horizontal wave propagation velocity

f is the friction force per unit length between structure and soil

λ is the apparent wave length

D is the structural rigidity such that

$$D = \frac{E(1-\upsilon)}{(1+\upsilon)\ (1-2\upsilon)}$$

and

E is the elastic modulus of the tunnel structure

υ is Poisson's ratio for the tunnel structural material.

The axial force in the tunnel, as indicated by the upper bound, is limited to the value at which slip may occur between the tunnel and the surrounding soil (Reference A.34).

Shear forces, V (horizontal or vertical), acting on a cross section taken normal to the longitudinal tunnel axis shall be calculated as follows:

$$V = \frac{V_{max}\ A_v G}{C}$$

where

V_{max} is the maximum ground particle velocity as given in Subsection 4.3.1 (horizontal or vertical)

C is the apparent horizontal wave propagation velocity

A_v is the cross sectional "shear area" of the tunnel taken normal to the longitudinal axis (horizontal or vertical)

G is the shear modulus of the tunnel structural material.

Moments, M, (horizontal or vertical) acting on sections taken normal to the longitudinal tunnel axis shall be determined as follows:

$$M = \frac{A_{max} E I}{C^2}$$

where

A_{max} is the maximum ground particle acceleration as given in Subsection 4.3.1 (horizontal or vertical)

C is the apparent horizontal wave propagation velocity

E is the elastic modulus for the tunnel structural material (lining)

I is the moment of inertia of the structure about the principle axis (horizontal or vertical).

Shears or peak moments occur about both the horizontal (transverse) axis and about the vertical axis at any point along the tunnel. The shears and moments, horizontal or vertical, must be calculated utilizing corresponding ground motion directional components and structural cross-sectional areas or moments of inertia, horizontal or vertical, as indicated above. Units for use in each formula may be in any system but must be internally consistent within each formula.

4.5 Design Requirements

As noted in Chapter 2, the design process involves layout and sizing of systems so that response to selected input falls within acceptable limits based on system resistance characteristics.

This section provides such limits or so-called acceptance criteria for SC A items. Also provided in this section are miscellaneous design details and design criteria not included elsewhere in this document.

As provided in Subsection 2.3.2, the general approach for SC A-1 items involves detailed design for elastic performance under load combinations including ODE conditions. SC A-1 items shall be checked for satisfactory inelastic performance including damage limits, stability and drift limits as well as selected internal force and stress limits under loading combinations which include MDE conditions.

SC A-2 items generally must perform elastically under MDE conditions. Additional criteria with regard to performance levels are given in the subsections which follow.

4.5.1 Load Combinations

This subsection defines various design loads and required load combinations including seismic. The use of these combinations in the design process is also specified.

For purposes of consideration hereafter, loadings are divided into four possible broad categories: operating (OL), normal environmental (NEL), upset condition (UCL) and extreme environmental (EEL). Seismic ODE loadings fall into the category of "normal environmental" while MDE loadings fall into the category of "extreme environmental".

Operating loads on an item are normally defined as dead, live, thermal, hydraulic and other loads induced by or under facility operation. Normal environmental loads include those related to wind or seismic events as they affect the facility. Such loadings have a moderate to high possibility of occurrence during the design life of the facility. Extreme environmental loads include only events such as an MDE which have a small probability of occurrence during the facility life.

Upset conditions include loadings which happen due to accidental occurrences such as thermal transients, pipe rupture, explosions, failures of other items which induce loads, and other abnormal, non-operating events.

The following load combinations shall be included in the design process:

(1) OL
(2) OL + NEL
(3) OL + UCL
(4) OL + EEL

All possible operating loadings which may act concurrently on an item under consideration shall be placed in Load Combination 1 (LC 1). If alternate nonconcurrent operating loadings can occur which require evaluation, Load Combination 1 may be subdivided thus:

LC 1A, LC 1B, ... etc.

For purposes of design, the worst case (most critical) combination or combinations considering all loadings and combinations shall govern the design of all items and their components. Limits on application or concurrency for Load Combinations 2, 3 and 4 are discussed below.

With respect to Load Combination 2 all possible operating loads shall be considered as described above, but only one environmental load shall be considered at a time, for example:

LC 2A = OL + WL
LC 2B = OL + OEL

where

WL = Normal wind loading
OEL = Operating design earthquake loading.

When considering ODE loadings, effects from vibratory motion and traveling wave effects shall be presumed to occur concurrently on buried structures.

Support displacements resulting from ODE caused vibratory motions may induce stresses in items, especially distributive systems such as piping. ODE support displacements shall be included as applicable but shall not be considered to occur concurrently with other vibratory (inertial) ODE effects.

With respect to Load Combinations 3 and 4, all appropriate operating loads shall be considered to be applied concurrently or in groups as discussed above. However, only one UCL or one EEL loading shall be considered to occur at a time for purposes of design or item evaluation.

4.5.2 Allowable Stresses In design (or qualification of items), when considering Load Combinations 1 and 2, allowable stresses for working stress design, and load factors for ultimate strength design, shall be in accordance with the appropriate RTD codes and standards (Reference A.1). In particular for LC 1, normal allowable stresses and load factors, as applicable, shall be the acceptance criteria. For LC 2 the same applies, except increases in allowable stress values (generally 1/3) for combinations with environmental loads (wind and seismic) permitted by the codes and standards shall apply. Correspondingly, decreases in load factors for strength design also apply.

In evaluating the effects of loadings considering LC 3 and LC 4 on an item, full yield stresses for steel structures and ultimate stresses for reinforced concrete structures may be developed in conjunction with ductility factors as applicable. Note that for SC A-2 items, only a ductility factor of unity is permitted. For design of other types of structural systems and materials, 1.5 times normal allowable stresses or yield, whichever is less, may be developed with ductility factors as applicable.

Where the District standards do not include allowable stress or load factor criteria for a particular material or item, the District or its consultants will provide such criteria as required. Similarly, the District will provide yield criteria where not otherwise covered.

Anchor bolts shall be designed to develop their full capacity by methods indicated in Reference A.35.

Table A-11 provides a general summary of stress/force acceptance criteria for the defined load combinations. Additional criteria are given in subsections below for piping, vessels, equipment, machinery and other non-structural items.

TABLE A-11

ACCEPTANCE CRITERIA FOR REQUIRED LOAD COMBINATIONS

LOAD COMBINATION	STRESS/FORCE ACCEPTANCE CRITERIA[1]
1. OL	Normal Code Allowable Stress (F_a)
2. OL + NEL[2]	Code Allowable with Normal Increase[3]
3. OL + UCL or 4. OL + EEL[4]	Structural Steel[5] 1.67 F_a but less than yield Reinforced Concrete[5] All load factors and strength reduction factors shall be 1.0 Other Materials[5] Allowable Stress = 1.5 F_a

[1]The acceptance criteria given herein are general. Additional criteria are given in Subsections 4.5.2.1, 4.5.2.2, 4.5.2.3, and 4.5.3.

[2]NEL includes only ODE or code prescribed wind loading.

[3]For example: 1.33 F_a for steel or appropriate decrease in load factors for concrete.

[4]EEL indicates MDE or other extreme environmental loads.

[5]Utilization of ductility is permitted only for SC A-1 items.

4.5.2.1 Special Considerations for Piping Systems - The basic design criteria, including that for stress, from ANSI B31.3 (Reference A.36) are applicable to design of piping systems, except as modified/supplemented herein.

The stress under LC 1 shall be limited to S_h and those under LC 2 shall be limited to 1.33 S_h, where S_h is the basic (allowable) hot stress. Other allowables in application of this load shall be consistent with this approach.

For Load Combinations 3 and 4, the allowable stress shall be determined as follows:

$$S_a = f[1.25 (S_c + S_h) - S_L]$$

where

S_a is the allowable stress

S_c is the permittable cold stress

S_h is the permittable hot stress

S_L is the actual stress under structural load conditions

f is the stress range reduction factor.

4.5.2.2 Special Considerations for Vessels Systems - The basic design criteria, including that for stress, from ASME Code UC-23 (Reference A.37) is applicable to SC A vessels. Stress limits are given in Table A-12.

4.5.2.3 Allowable Stresses on Equipment, Machinery and Other Non-Structural Items - This paragraph covers general requirements for allowable stresses under seismic loading conditions for mechanical systems, equipment and components.

Permittable stress under Load Combinations 1 and 2 shall be based on the criteria for structural systems cited above and when not otherwise addressed shall be based on applicable recognized codes and standards for the material to be evaluated.

Consideration of stresses under Load Combinations 3 and 4 shall be based on appropriate allowables using a similar approach to that used for piping or vessels, as applicable. See Paragraphs 4.5.2.1 and 4.5.2.2.

Forces on equipment at flanges and nozzles due to piping stress and anchorage shall be considered in design of the equipment. Seismic forces shall be based on a piping seismic analysis or appropriate limiting values. Allowable flange and nozzle stresses shall be based on applicable codes or specifications of API, ASME, NEMA and similar codes for items of various materials as appropriate. Load combinations and stress allowables shall be considered in a similar manner to piping systems and piping components as specified herein.

TABLE A-12

STRESS LIMITS UNDER SEISMIC LOADING COMBINATIONS FOR PRESSURE VESSELS

	COMPONENT AND LOADING CONDITIONS	ALLOWABLE STRESSES FOR SEISMIC LOAD CONDITIONS: ODE Load Combinations	ALLOWABLE STRESSES FOR SEISMIC LOAD CONDITIONS: MDE Load Combinations
a.	Maximum general primary membrane stress*	S_a	$1.5S_a$
b.	Combined maximum general primary membrane stress plus primary bending stress across the thickness	$1.5S_a$	$2S_a$
c.	Localized discontinuity stresses and secondary stresses as defined in ASME Code Section VIII, Division 2, combined with maximum general primary membrane stress	$3S_a$	$3S_a$

*S_a differs for Tension (ASME Code UG-23a) and Longitudinal Compression (ASME Code UG-23b)

.5.3 Other Acceptance Criteria

Where performance of an item can be affected by relative displacement or drift, limiting criteria shall be established on a case by case basis except as provided below.

Building-like structures, stations, towers, and other structures designed with lateral force resisting systems comprised primarily of moment resisting frames shall be designed to limit story to story drift and total lateral displacements under seismic loading as follows:

$$\Delta ODE \leq 0.010\ H$$
$$\Delta MDE \leq 0.020\ D_F\ H$$

where

Δ is the relative lateral displacement or drift between any two locations a vertical distance, H, apart on the structure under consideration. Δ is caused by ODE and MDE motions, respectively.

is the ductility factor permitted and utilized in design for SC-A1 items. For SC-A2 items D_F is limited to 1.0.

In the above equations the left side represents the drift calculated on the basis of seismic-dynamic analysis of the structure which is to be limited to a maximum value prescribed by the right-hand side of the equipment for the ODE and MDE, respectively.

4.5.4 Additional Design Considerations

4.5.4.1 Connections - All structural connections (including attachments and anchorages) for SC A items shall be designed to withstand forces based on either of the two following limits:

(a) 1.25 times the computed force which would otherwise control the connection design

(b) the greatest capacity of all members which frame into or though the connection.

4.5.4.2 Stability and P-Delta Effects - Structures and other items shall be designed to remain stable under all loading conditions. When ductility is permitted in design, P-delta effects which account for the maximum inelastic deformation shall be evaluated and included in assessing stresses and structural stability.

4.5.4.3 Overturning Effects - Items designed to resist lateral force shall be positively anchored to supporting structures or foundations when considering earthquake loading conditions. This provision is applicable to items under ODE and MDE conditions except as follows:

At the interface of major structure foundations with soil, anchorage to prevent uplift need not be provided if the primary structure has a natural period greater than 0.05 seconds. In other cases stability ratios for overturning (and sliding) shall be no less than 1.5 and 1.0 for ODE and MDE conditions, respectively. The stability ratio for overturning is defined as the applicable total dead load resisting moment divided by the maximum value of earthquake induced overturning moment statically applied on the structure. The sliding stability ratio is defined as the applicable resisting forces divided by the maximum horizontal earthquake forces applied to the structure in a given direction.

4.5.4.4 Ductile Detailing - In designing SC A items, especially primary structures, detailing and arrangements shall provide for maximum structural ductility.

All steel and concrete structures shall be designed with ductile detailing as applicable to the highest seismic zone. Steel frames shall be designed to provide details, width to thickness ratios, bracing and lateral support as required by AISC Specification, Part 2 (Reference A.38) or the Los Angeles City Building Code, Division 26 (Reference A.39), whichever is more stringent.

Concrete frames and shear wall structures shall be designed to provide ductility according to the most stringent requirements of ACI, Appendix A, (Reference A.40) or the Los Angeles City Building Code, Division 26 (Reference A.41).

4.5.4.5 Static and Dynamic Soil Pressures - In the design of below grade walls which resist lateral earth pressures, the design shall provide for a seismic increment of pressure in addition to the normally considered dry or saturated lateral earth pressures. Such so-called dynamic pressures are included to allow for soil-structure interaction effects which may occur.

Because cohesionless soil is to be used to backfill foundation walls and tunnels, and because the assumption of cohesionless soil tends toward derivation of conservative lateral pressures for design, the assumption of cohesionless material with relatively low values of angle-of-internal friction is made. A representative angle for typical materials for the project (36°) is used. In lieu of this criteria, structural walls for natural soils and bedrock conditions may be designed for loadings determined from properly substantiated data upon approval by the District.

The calculation of seismic loading on walls is based on the well known Monobe-Okabe formulation (Refèrences A.42 and A.43) which is further developed and explained by Kapilla (Reference A.44), Seed and Whitman (Reference A.45), and Dowrick (Reference A.46). Figure A-9 depicts the basis for determining the dynamic soil pressures, P_{AE}. The Monobe-Okabe formulation as shown in Figure A-9 represents the sum of the normal active static pressures (P_A) and the dynamic soil pressure increment (ΔP_{AE}). However, the point of application of the resultant load ($P_{AE} = P_A + \Delta P_{AE}$) must be determined from the individual components P_A and ΔP_{AE}. Thus both components of soil pressure must be calculated and superimposed for design purposes (See Figure A-10). Figure A-10 and Table A-13 are derived for frequently encountered design conditions, i.e., zero values of friction angle of wall δ, slope angle of back of wall β, and slope of backfill ω. As with static loadings, the dynamic effects of submergence and surcharge are included, and similar simplifying assumptions are made as to cohesion and wall friction. Note that the dynamic pressure increments under seismic conditions are not considered to be a function of wall stiffness, in contrast to the static case. The ground water seismic incremental pressure is assumed to be parabolically distributed with the coefficients as shown in Figure A-10. The basis of this assumption is given by Westergaard (Reference A.47). Soil material properties listed in Table A-13 are based on data from the main body of this report and the static/dynamic soil pressure values are based on the criteria, formulations and design conditions discussed above. For different design conditions, careful consideration should be given to the applicability of Figure A-10 and Table A-13. Modifications as necessary shall be made by referring to References A.42 - A.47 as applicable.

Under high values of ground acceleration, the Monobe-Okabe dynamic pressure increments become overly conservative (See Reference A.48). This conservatism occurs when the angle (αAE) defining the failure plane of the sliding soil wedge falls below 30 degrees. Therefore, for any design conditions, αAE shall not be taken less than 30 degrees when calculating dynamic pressures.

4.5.4.6 <u>Design for Special Hazards - Underground Structures</u> - This section provides guidelines regarding the following potential hazards:

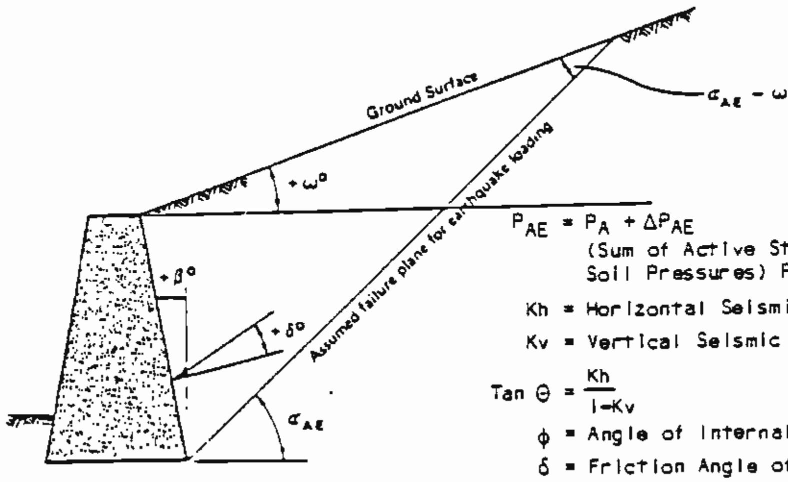
(a) Fault rupture;

(b) Abrupt changes in foundation material rigidity;

(c) Soil or rock failures and dynamic settlements;

(d) Liquefaction and related soil failures; and

(e) Intersection or joining points of relatively rigid buried structures.



$P_{AE} = P_A + \Delta P_{AE}$ (Sum of Active Static and Dynamic Increment Soil Pressures) Point of Application Varies

K_h = Horizontal Seismic Coefficient

K_v = Vertical Seismic Coefficient

$$\tan \Theta = \frac{K_h}{1-K_v}$$

ϕ = Angle of Internal Friction of Soil

δ = Friction Angle of Wall

P_A = Static Active Soil Pressure

ΔP_{AE} = Dynamic Soil Pressure Increment

$$\cot(\alpha_{AE} - \omega) = -\tan(\phi + \delta + \beta - \omega) + \left[\sec(\phi + \delta + \beta - \omega)\right]\sqrt{\frac{\cos(\beta + \delta + \theta)\sin(\phi + \delta)}{\cos(\omega - \beta)\sin(\phi - \theta - \omega)}}$$

NOTE. Failure Plane for sliding wedge to develop dynamic soil pressures is based on Mononobe – Okabe Relations. Coulomb conditions are assumed. In many design cases β, δ, and ω equal zero.

(Reference A.42 - A.46)

BASIS FOR DYNAMIC PRESSURES

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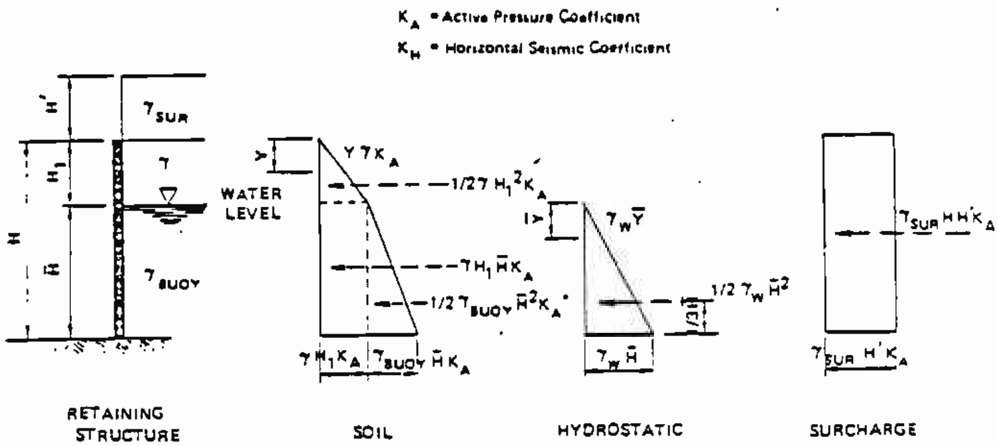
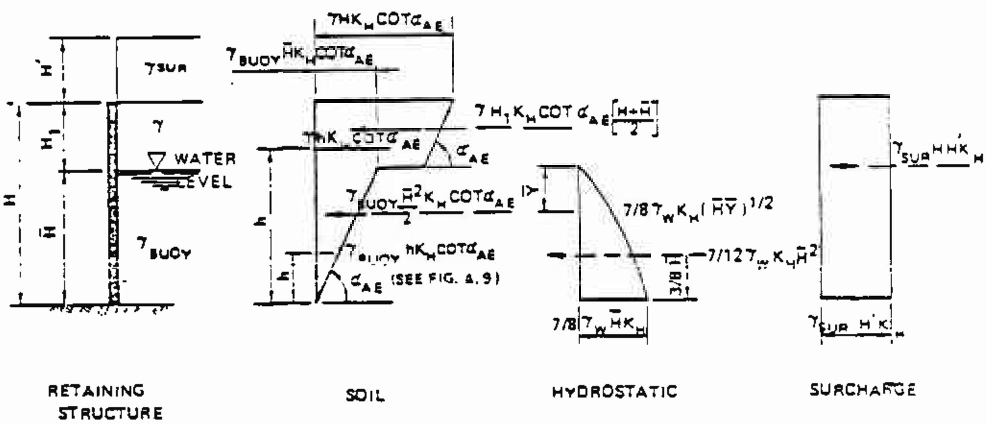


ACTIVE STATIC PRESSURES (P_A)



ACTIVE DYNAMIC PRESSURES (ΔP_{AE})

Note : Refer to Table A-13 for typical values and terms. Substitute K_O for K_A for static pressure on rigid walls.

See Figure A-9 for basis of Dynamic Pressure.

ACTIVE DYNAMIC PRESSURES ON RETAINING STRUCTURES

TABLE A-13

DESIGN VALUES FOR LATERAL SOIL PRESSURE PARAMETERS
(SEE PARAGRAPH 4.5.4.5 FOR APPLICATION)

PARAMETER	SYMBOL	UNITS	DESIGN VALUES Static	ODE	MDE
Seismic Coefficient					
- Horizontal	K_H	--	-	.30	.60
- Vertical	K_V	--	-	.20	.40
$\tan^{-1} \frac{K_H}{1-K_V}$	θ	Degrees	-	20.6	45.0
Angle of Internal Friction of Soil	ϕ	Degrees	36	36	36
Friction Angle of Wall	δ	Degrees	0	0	0
Slope Angle					
of Backfill	ω	Degrees	0	0	0
of Back of Wall	β	Degrees	0	0	0
Pressure Coefficient					
Active	K_A	-	.26	-	-
At Rest	K_o	-	.42	-	-
Angle of Assumed Failure Plane*	α_{AE}	Degrees	63	43.5	30
Unit Weight of Soil (Total)	γ	#/ft³	120	120	120
Buoyant Unit Weight of Soil	γ_{buoy}	#/ft³	66	66	66
Unit Weight					
of Water	γ_w	#/ft³	62.4	62.4	62.4
of Equivalent Surcharge	γ_{sur}	#/ft³	varies	varies	varies

*Refer to discussion in text on value of angle for MDE conditions.

TABLE A-13
(continued)

PARAMETER	SYMBOL	UNITS	DESIGN VALUES		
			Static	ODE	MDE
Height					
of Equivalent Surcharge	H'	ft	varies	varies	varies
of Soil Above Water Table	H_1	ft	varies	varies	varies
of Soil Below Water Table	$\bar{H}$	ft	varies	varies	varies
of the Wall	H	ft	varies	varies	varies
Distance Down					
from Top of Wall	y	ft	varies	varies	varies
from Water Level	$\bar{y}$	ft	varies	varies	varies
Distance Up					
from Bottom of Wall	h	ft	varies	varies	varies
Static Pressure - Flexible Wall	$\gamma H_1 K_A$	$\#/ft^2$	$31.2H_1$	-	-
Static Force - Flexible Wall	$1/2 \gamma H_1^2 K_A$	#	$15.6H_1^2$	-	-
Static Pressure - Rigid Wall	$\gamma H_1 K_o$	$\#/ft^2$	$50.4H_1$	-	-
Static Force - Rigid Wall	$1/2 \gamma H_1^2 K_o$	#	$25.2H_1^2$	-	-
Static Pressure - Flexible Wall**	$\gamma_{buoy} \bar{H} K_A$	$\#/ft^2$	$17.2\bar{H}$	-	-
Static Force - Flexible Wall**	$1/2 \gamma_{buoy} \bar{H}^2 K_A$	#	$8.6\bar{H}^{2\dagger}$	-	-

**Below ground water level.

†Plus 31.2 H, $\bar{H}$. See Figure 10.

TABLE A-13
(continued)

PARAMETER	SYMBOL	UNITS	DESIGN VALUES Static	ODE	MDE
Static Pressure - Rigid Wall**	$\gamma_{buoy}\bar{H}K_o$	#/ft^2	$27.7\bar{H}$	-	-
Static Force - Rigid Wall **	$1/2\gamma_{buoy}\bar{H}^2K_o$	#	$13.9\bar{H}^2$†	-	-
Hydrostatic Pressure**	$\gamma_w\bar{H}$	#/ft^2	$62.4\bar{H}$	-	-
Hydrostatic Force**	$1/2\gamma_w\bar{H}^2$	#	$31.2\bar{H}^2$	-	-
Static Surcharge Pressure - Flexible Wall	$\gamma_{sur}H'K_A$	#/ft^2	$.26\gamma_{sur}H'$	-	-
Static Surcharge Force - Flexible Wall	$\gamma_{sur}HH'K_A$	#	$.26\gamma_{sur}HH'$	-	-
Static Surcharge Pressure - Rigid Wall	$\gamma_{sur}H'K_o$	#/ft^2	$.42\gamma_{sur}H'$	-	-
Static Surcharge Force - Rigid Wall	$\gamma_{sur}HH'K_o$	#	$.42\gamma_{sur}HH'$	-	-
Dynamic Pressure	$\gamma HK_H COT\alpha_{AE}$	#/ft^2	-	$37.3H$	$124.7H$
Dynamic Force	$1/2\gamma H_1K_H COT\alpha_{AE}[H+\bar{H}]$	#		$18.7H_1[H+\bar{H}]$	$62.4H_1[H+\bar{H}]$
Dynamic Pressure**	$\gamma_{buoy}\bar{H}K_H COT\alpha_{AE}$	#/ft^2	-	$20.5\bar{H}$	$68.6\bar{H}$
Dynamic Force**	$1/2\gamma_{buoy}\bar{H}^2K_H COT\alpha_{AE}$	#	-	$10.3\bar{H}^2$	$34.3\bar{H}^2$
Hydrodynamic Pressure**	$7/8\gamma_w\bar{H}K_H$	#/ft^2	-	$16.4\bar{H}$	$32.8\bar{H}$

**Below ground water level.

†Plus 50.4 H, $\bar{H}$. See Figure 10.

TABLE A-13
(continued)

PARAMETER	SYMBOL	UNITS	DESIGN VALUES		
			Static	ODE	MDE
Hydrodynamic Force**	$7/12\gamma_w\bar{H}^2K_H$	#	-	$10.9\bar{H}^2$	$21.8\bar{H}^2$
Dynamic Surcharge Pressure	$\gamma_{sur}H'K_H$	#/ft^2	-	$.30\gamma_{sur}H'$	$.60\gamma_{sur}H'$
Dynamic Surcharge Force	$\gamma_{sur}HH'K_H$	#	-	$.30\gamma_{sur}HH'$	$.60\gamma_{sur}HH'$

**Below ground water level.

The purpose of this subsection is to note the existence of potential significant seismic hazards within the Metro Rail route. This highlights the need in selected circumstances for special and very detailed design solutions to mitigate the potential hazards.

In such cases general guidelines are provided herein. Detailed design solutions shall be developed on a case by case basis as additional information is obtained on each local area of the Metro Rail system and its surrounding geoseismic environment.

General locations where potential hazards a to c above could occur can be obtained by reviewing data in the main body of this report. Additional data on this subject is required for design and will be provided during the design phases of the project by the District's Geotechnical Consultant.

If a significant earthquake occurs on either the Hollywood fault or the Santa Monica-Malibu fault, structures and other items crossing the fault zones potentially will be subjected to oblique displacement (i.e. a significant component of displacement in both the vertical and horizontal directions)as well as possibly significant horizontal crustal shortening across the fault zone (Item a above). Refer to Table A-3. The potential for shearing and compression of structures at fault crossings will require detailed design solutions such as providing appropriate degrees of articulation and flexibility for tunnel structures, connections and systems within tunnels. Similar solutions may be required for hazard Item b above.

Potential zones of soil or rock failures, instability and/or excessive dynamic settlements may require relocation of critical items to more stable locations. If relocation is not practical, soil or slope stabilization or special foundation design measures may be required.

As stated in Subsection 2.4.2, liquefaction induced soil deformations, loss of soil strength and development of high excess pore water pressures can have a significant effect on engineered structures. Effects can include:

- loss of structure/foundation support
- increased uplift forces
- increased loadings
- significant and difficult to predict differential vertical and horizontal displacements.

Mitigation measures include but are not limited to:

- route realignment
- relocation of critical structures along the route
- removal and/or recompaction of liquefiable soils
- designing special drainage systems
- installing deep foundations
- providing structures designed to accommodate large differential settlements.

Large potential lateral displacement may require design measures similar to those at fault crossings.

4.5.4.7 Design for Special Hazards - Aerial Structures - Bridge-like structures of relatively long spans are susceptible to effects of relative displacements between supports due to earthquake motions. The Engineer shall include provisions for relative displacements in design of such aerial structures. Either appropriate restraints between supports and structural members shall be provided or bearing lengths shall be adequate to provide for relative displacements with a factor of safety. Relative displacements to be accounted for shall be determined by analysis of traveling wave effects on the structures and supports.

4.5.4.8 Non-Structural Elements - Provisions by design or by qualification of SC A-2 items to retain operability and function during seismic events are cited in other parts of these criteria. In addition, non-structural elements which are part of or attached to SC A items shall be designed and constructed to accommodate the structural seismic displacements which may occur. The design consideration shall mitigate dislocation and damage of such nonessential but related items by appropriate detailing of connections and by insuring adequate flexibility to allow for the support movements.

4.5.4.9 Vertical Design Motions Near Faults - This paragraph provides special vertical design input motion criteria for near-fault locations. These criteria are applicable only to SC A-2 items which are potentially sensitive to vertical acclerations. For purposes of limiting these criteria, "near-fault locations" shall be taken as Starter Portion locations within 2.5 miles of the Malibu-Santa Monica and Hollywood faults.

For purposes and at locations indicated above design vertical ground accelerations shall be taken to be equal to horizontal design accelerations for ODE and MDE, respectively. Similarly, vertical design response spectra shall be taken to be equal to horizontal.

4.6 SEISMIC QUALIFICATION OF SC A ITEMS

This section provides criteria for verifying performanace of previously designed SC A items. All of the criteria for design of SC A items given in Sections 4.1 through 4.5 are in force with regard to seismic qualification unless otherwise specified in this section. Any item which does not meet qualification requirements must be modified or redesigned until the required performance for qualification is obtained.

ualification may be obtained by analysis or test or combination of both. nalytical methods shall be in accord with Sections 4.3 through 4.5 herein. esting shall conform to Subsection 4.6.1. Qualification by historical pproach is a means whereby complete documentation of previous design or ualification efforts is provided to demonstrate meeting the requirements for nput, analysis, testing, and acceptance specifed herein. A qualification eport must include data from the original qualification or design plus all dditional information to demonstrate meeting criteria of this document, and hall conform to the requirements of Subsection 4.6.3.

alification may be performed by the Structural Engineers who are responsible r the structural-seismic design of the facility, or it may be performed by dependent manufacturers or suppliers of equipment with appropriate consul-nts. In the latter case, the Structural Engineer must develop a detailed alification specification to provide suppliers/consultants with the complete d specific qualification criteria on an item by item basis.

6.1 Testing Requirements

4.6.1.1 Input - Free field ground motion input for seismic qualification testing shall be based on the criteria given in Section 4.3 herein with modifications indicated in this subsection (4.6.1) as applicable. Directional considerations shall comply with Paragraph 4.4.3.5 except as modified herein.

Support motions to be used as input shall comply with Subsection 4.3.2 and 4.4.3 as applicable and except as modified herein.

Input in general shall consist of time history acceleration records or response spectra which are appropriately utilized by testing apparatus incorporating transducers to generate time dependent table motions of proper definition.

Qualification test input requirements are specified entirely or in part in the form of response spectra. It is required that each response spectrum of the test input motion be shown to envelope the specified spectrum, and that the peak zero period input acceleration level is equal to or greater than the zero period acceleration value of the required spectrum. Test motions shall be monitored and analyzed with a shock spectrum analyzer, or equivalent, to verify that input motions to the equipment being tested are actually above the specified level.

4.6.1.2 Test Plan - A complete test plan shall be developed for approval by the District prior to actual testing. The plan shall outline the approach to testing which is to be used in sufficient detail to demonstrate compliance with requirements given in this document.

4.6.1.3 General Testing Approach - Except as otherwise provided herein, testing required for mechanical, electrical and other equipment and hardware shall comply with the applicable provisions of Reference A.49, the IEEE Standard 344-1975. In addition, testing which involves assurance of the function of electrical relays or contacts shall be performed according to applicable provisions of Reference A.50, IEEE Standard 501-1978.

Seismic tests shall be performed by subjecting equipment to vibratory motion which conservatively simulates that required at the equipment mounting. The details of the test procedures given below constitute the more common ones presently in use, but do not preclude others if acceptable to the District. The test program may be based upon selectively testing a representative number of mechanical or electrical components according to type, load level, size, etc., on a prototype basis.

The equipment shall be tested in such a manner as to demonstrate its ability to perform its intended function, and sufficient monitoring equipment shall be used to evaluate performance before, during and following the test.

The external and internal operational loads and functions (including energized circuitry) to which the equipment is normally subjected shall be applied or simulated. The equipment to be tested shall be mounted in a manner that simulates the intended service mounting. Proof of correct operating performance may require recording and data logging of monitored parameters and events during and following the shake tests. For example: for mechancial equipment or mechanical components, typical monitored parameters may be pressure drop, valve closure, shaft rpm, etc. For electrical equipment or electrical components, typical monitored parameters may be contact closure, voltage and current levels, etc.

Four major procedures corresponding to four levels of testing are specified herein. In general, the complexity of the test item and the testing increases from Level 1 to Level 4.

The testing levels considered are as follows:

Level 1 Resonant Search
Level 2 Single Frequency Test
Level 3 Single Axis Multiple Frequency Test
Level 4 Multiple Axis Multiple Frequency Test.

4.6.1.4 Directional Considerations - The qualification tests should be performed with test input motion applied in the direction of each of the three orthogonal major axes of the equipment simultaneously. However, alternative procedures may be allowed under conditions outlined below.

Single axis tests may be allowed if the equipment being tested can be shown to respond independently in each of the three orthogonal axes. This is the case if the coupling is zero or very low. For example: if an item is normally mounted on a panel that amplifies motion in one direction, single axis testing of the item may be adequate. Similarly, if an item is restrained to motion in one direction, single axis testing is appropriate and may be used. Single axis testing may also be used for multiple axis dynamically coupled equipment if the input accleration level is increased to account for the coupling, and the effects of all possible axes of vibration are tested. If the above considerations do not apply, multiple axis testing shall be used. The minimum is biaxial testing with simultaneous inputs in a horizontal and vertical axis. Independent random inputs are preferred but if in-phase inputs are used (such as with single frequency tests) four tests shall be run as follows:

First - with the inputs in phase

Next - with one input 180° out of phase

Next - with the equipment rotated 90° horizontally and the inputs in phase

Finally - with the same equipment orientation but with one input 180° out of phase.

4.6.1.5 Selection of Multiple or Single Frequency Testing - Seismic excitation generally has a broad frequency content. Mutliple frequency vibration input motion should therefore be used for seismic qualification. However, single frequency input, such as sine beats, may be applicable provided one, or more, of the following three conditions is met:

(a) When the seismic ground motion has been filtered due to one predominate structural mode, the resulting floor motion may consist of one predominate frequency. This is characterized by a sharp, narrow-banded response spectra.

(b) When it can be demonstrated that the anticipated response of the equipment is adequately represented by one mode of vibration.

(c) The input has sufficient intensity and duration to excite all modes to the required magnitude, such that the testing response spectra will envelope the corresponding response spectra of the individual modes.

4.6.1.6 Resonant Search Test (Level 1) - This test shall be used to determine natural frequencies, mode shapes and internal damping of a particular system or equipment item or a selected component. The test may be utilized to provide information for analysis. This type test, even when not referred to by level, is to be automatically incorporated under the Level 2, 3, or 4 tests described below, as resonant search is always required prior to performance testing.

When performed to support analysis, the methods of testing may be either low amplitude shaker tests with small energy shakers mounted on the equipment, or shake table tests. In either case the test shall be utilized to determine damping, frequencies, and modes of vibration of the item. Pull back tests may be considered as an alternate method. The exploratory tests shall be in the form of a single axis continuous sweep frequency search using a sinusoidal steady-state input at the lowest possible amplitude at which the test facilities are capable of determining resonance. The search shall be performed in each principal axis direction and shall include a minimum of two continuous sweeps from 1 to 35 to 1 Hz at a frequency sweep rate of no greater than 1 octave per minute. All resonant frequencies of the equipment shall be recorded for the testing of flexible equipment as specified below. Structural coupling data may also be obtained to provide justification for deviation from the multiple axis input requirement.

4.6.1.7 Single Frequency Tests (Level 2) - This test generally requires utilization of a shake table with single axis input, one frequency at a time, at prescribed acceleration levels up to a given maximum.

If no resonances are located within the range of 1 to 35 Hz or if the criteria of Paragraph 4.6.1.5 are met, then single frequency testing may be acceptable.

Where called for or accepted as an alternate, single frequency testing shall be performed at appropriate frequencies, but as a minimum at the following frequencies: 5, 8, 13, 20 and 33 Hz. The tests shall also include resonant frequencies, if known, of the support structure as indicated by peaks in the applicable response spectra. In any case, single frequency tests shall be made at a minimum of five frequencies. The equipment shall be tested a minimum of two times at each frequency.

Single frequency tests shall consist of either sine beat tests or continuous sine tests as specified below:

(a) Sine Beat Test. A test at any frequency shall consist of the application of sine beats whose peak acceleration produces equipment response equal to the specified test spectrum at the test frequency. The duration of the beat shall be a minimum of 10 cycles unless it can be shown that a lower number of cycles is sufficient to exceed or duplicate the required response of the equipment. The time of the pause between beats shall be long enough to allow the equipment to come to rest but no longer than 5 minutes. A minimum of five beats is required.

(b) Continous Sine Test. A test at the frequency of interest shall consist of the application of a continuous sinusoidal motion corresponding to the acceleration at which the equipment is to be qualified for an appropriate length of time. A time duration shall be selected which is conservatively consistent with the uses for which the device is being qualifed. A time duration of 20 to 30 seconds is commonly used for the test of the equipment.

For single frequency testing, the test frequencies shall be at the five frequencies specified above. Any building frequencies indicated by peaks in the response spectrum, and any equipment resonances noted during exploratory tests within 1 to 30 Hz must also be included as test frequencies.

4.6.1.8 Single Axis Multiple Frequency Tests (Level 3) - Multiple frequency tests shall be required to verify performance of relatively complex equipment under a seismic event.

For the general case where equipment is found to have resonant frequencies in the range of 1 to 35 Hz and the seismic ground motion has not been strongly filtered, the support motion retains broad band characteristics, and multiple frequency testing is applicable. Though equipment is defined to be rigid at or above frequencies of 30 Hz, the range for testing is broadened to allow for test support interaction.

The testing procedure shall be that which most nearly simulates field situtations and clearly demonstrates the equipment's capability to perform during and after the specified earthquake events. The equipment shall be subjected to the full qualification test levels in each major direction for at least the number of times corresponding to the number of time-history input records to be utilized for testing.

Single axis tests are called for when a given item responds without significant directional coupling. At least three tests with different time-history inputs are required for each of the three major orthogonal axes of the item.

4.6.1.9 Multiple Axis Multiple Frequency Tests (Level 4) - This type of test may be specified to be two or three axis tests depending on the characteristics of the equipment or component to be qualified. For multiple axis testing, the specification of three axis testing will be cited in the particular item specification; otherwise two-axis testing is required, normally one major horizontal axis with a vertical axis at a time. Sufficient tests must be performed to consider response to concurrent input motion in the direction of the three principal axes.

4.6.1.10 Testing of Assembly Components - It is not always practical to monitor performance of an entire complex while in the operating mode. When this situation arises the overall assembly may be tested in a nonoperating status at the prescribed input level. Accelerations are then measured at the attachment points of key components. These levels are then used to perform seismic qualification tests of each key component individually with each component in the operating mode. The assembly is considered to be qualified when all of the key components are correspondingly qualified.

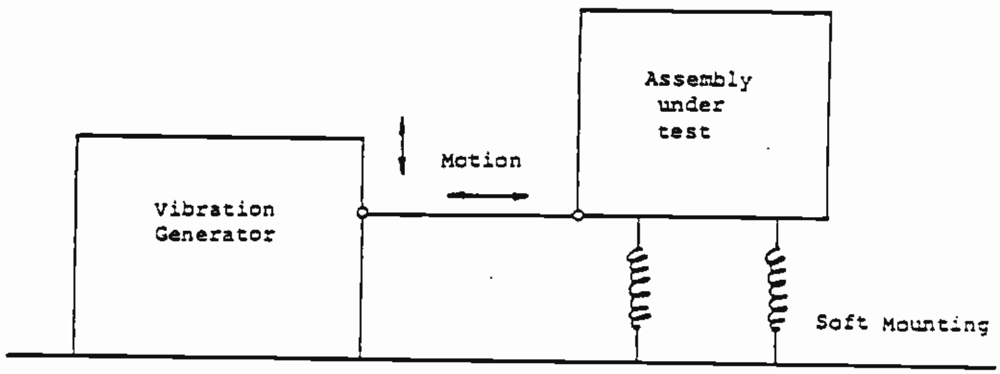
4.6.1.11 Assemblies and Support Structures - An assembly or equipment item and its supporting base or structure to be tested shall be mounted on a shake table or other test device in a manner that simulates the intended service mounting. If the equipment is too large to be so mounted, other means which simulate the service mounting shall be used. Possible alternatives involve the use of a "slip table", or "soft mounting" the equipment, using flexible supports with resonance outside the frequency band of the test and rigidly connecting the base of the equipment to the shake table or other input device (see Figure A-11). The vibratory motion shall be applied to the equipment as described above.

Where equipment consists of a large assembly with one or more supported components - such as air conditioning units, consoles, racks and large panels which support smaller components tested as specified above - the assembly or support may be vibration tested without the smaller components being in operation. However, the components shall be in their operational configuration; that is, charged with oil in crankcases, refrigerant in coils, water in heat exchangers, etc.

The goal is to qualify the support structure and to determine that, at the expected frequencies and accelerations of the specified earthquakes, the support structure does not amplify the forces beyond that level at which the components have already been proven to operate. Appropriate functional monitoring shall be performed during the testing.

2 Combined Analysis and Testing

In some instances, seismic qualification response determination is best accomplished by a combination of analysis and testing.



ALTERNATE TEST ARRANGEMENT FOR LARGE SYSTEMS & EQUIPMENT

Southern California Rapid Transit District
METRO RAIL PROJECT

Project No.

Figure No
A-11

Testing for mode shapes and frequencies (resonant search) may be required to support analysis for complex items. Selected sensitive or complex components of an item assembly may require testing as an extension to analysis of the assembly. Also, where an item is too large for testing as a unit, analysis of the assembly plus testing of selected components may be the required qualification procedure.

6.3 Documentation of Seismic Qualification

After completing the actual qualification for an item, a complete detailed report of the test and/or analysis results and supporting information which demonstrates seismic qualification shall be provided.

4.6.3.1 Analysis Report - The seismic qualification report for a qualification by analysis shall include the items listed below in addition to those required elsewhere.

(a) Brochure showing typical equipment to be analyzed.

(b) Outline, isometric, exploded view(s) or assembly drawing(s) of equipment showing locations of components.

(c) Dimensioned sketches or drawings detailed to show locations of applied loads and centers of gravity.

NOTE: The requirements of (b) and (c) above may be combined on the drawing series.

(d) Applied loads including equipment weight, seismic and operational loads.

(e) Designations and grades of material(s) to be used; i.e., SAE, ASTM, AISI, etc.

(f) All calculations shall be provided. The calculations shall include both computer and manual calculations and shall meet the requirements for design calculations specified in Paragraphs 4.4.3.11 and 4.4.4.3.

(g) Sources or references for design criteria where not specifically included in this document; (i.e., allowable stresses, flange leak criteria, etc.). These may be SAE standards, ASME Code, AISC Manual or other sources.

(h) If modifications are needed to satisfy seismic requirements, the final report shall include drawings showing details of these modifications.

(i) Certification that the equipment is seismically qualified. The certification shall be by a licensed professional engineer.

(j) Peak support reactions under load combinations including seismic and reaction locations.

4.6.3.2 Test Report - The report for a qualification by test shall include as a minimum the items listed below.

(a) Acceptance (pass/fail) criteria for testing.

(b) Type of test motion (single or biaxial).

(c) Design response spectra used.

(d) Labeled photographs of equipment mounted in test position, including views showing all accelerometers as mounted for test.

(e) Calibration procedures for shake table or other test devices including maximum support accelerations of each calibration run.

(f) Plots of test response spectra of shake table or other test devices including a listing of test method, test frequencies, acceleration and input wave forms.

(g) Seismic test results and observations including description of abnormalities encountered during testing.

(h) Detailed description and drawings showing recommended and installed modifications to the equipment, if any.

(i) Data on test set-ups shall be provided which demonstrates the accuracy of the input, measurement/monitoring system, and results as determined.

(j) The procedure for performance tests performed prior and after seismic testing.

(k) Performance test results and conclusions.

(l) Certification by Contractor and his consultants that the equipment is seismically qualified.

4.6.3.3 Inspection of Tests - The Engineer shall provide notice at least two weeks in advance of any test to be performed and advise the District so that the test set-up may be reviewed. The test shall not be performed prior to approval by the District. The Engineer shall make appropriate arrangements for the District to observe and monitor the entire test.

The testing program shall not be terminated until the District approves the test set-up and test as having complied with the test plan and criteria herein.

4.6.4 Verification

In conjunction with requirements for documentation given in other parts of Chapter 4, data provided shall include information which demonstrates the accuracy of the procedures and results, both for analysis and testing.

Final qualification plans and reports shall be signed by a licensed professional engineer indicating responsible charge of the qualification work and responsibility for the qualification of the subject item.

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