MWDSLS SEISMIC DESIGN GUIDELINES

Last Updated: December 22, 2011

This policy document was the cooperative effort of many professionals and will result in designs which will substantially enhance the seismic design of the Metropolitan Water District of Salt Lake & Sandy facilities. A partial listing of key personnel includes: Dr. Larry Reaveley (University of Utah), Dr. Steven Bartlett (University of Utah), Mr. Ivan Wong (URS Corporation), Ms. Genevieve Atwood (MWDSLS), Mr. John Masek (ABS Consultants), and engineering staff members of MWDSLS. This policy was approved by the MWDSLS Engineering Committee on May 9, 2003.

POLICY

The MWDSLS policy for seismic design has been developed for use in design of new structures as well as retrofit design of existing structures.

1.1 ALL NEW CONSTRUCTION

- Design to Level 3 performance standard. (Conform to IBC 2003 Structural Design Criteria)

1.2 ADDITIONS TO AND ALTERATIONS (REMODELING) OF EXISTING STRUCTURES

For existing structures, which are being considered for alteration or remodel, a seismic risk evaluation shall be performed. The purpose of this evaluation shall be to determine the ramifications of seismic upgrades from both a facility impact and a cost perspective. Seismic evaluations shall be performed using FEMA 356 Criteria and using site-specific response spectra as specified in Tables 1-1 and 1-2 (Section 1.7). Evaluations shall be done for both Level 2 and Level 3 seismic design loads. The final design level choice shall be determined by MWDSLS on a structure-by-structure basis.

1.3 DESIGN OF EQUIPMENT, PIPING AND NON-STRUCTURAL COMPONENTS

For equipment, piping and non-structural components in new structures, seismic design shall be done using Level 3 design loads and IBC 2003 design criteria. For seismic retrofit studies of existing facilities, evaluations shall be done using Level 2 and 3 load criteria and IBC 2003 design standards. The final design level shall be determined by MWDSLS. Furthermore, specific design detailing requirements (currently under development by the District) shall be utilized in cases where they exceed IBC 2003 requirements. In no case shall equipment and piping be retrofitted to a lower level than the building retrofit level (Section 1.2).

For existing structures, equipment, piping and non-structural elements shall also conform to the following FEMA 356 rehabilitation requirements, as follows:

MWDSLS Level 2

Larger of :

a. 10% in 50 year site-specific spectra (Horizontal only) (Immediate Occupancy – NB performance level in FEMA 356)

or

b. 2% in 50 year site-specific spectra (Horizontal only) (Life Safety – NC performance level in FEMA 356)

MWDSLS Level 3

Larger of:

a. 10% in 50 year site-specific spectra (Horizontal only) (Operational Level – NA performance level in FEMA 356)

or

b. 2% in 50 year site-specific spectra (Horizontal only) (Immediate Occupancy – NB performance level in FEMA 356)

NA, NB, and NC levels are further defined in FEMA 356 (previously referenced).

1.4 GEOTECHNICAL SEISMIC DESIGN OF FOUNDATION SYSTEMS AND EARTHEN STRUCTURES

1.4.1 General Types of Required Geotechnical Analyses

Three general types of analyses may be used to evaluate the seismic stability of a foundation or earthen structure: (1) pseudo-static analysis, (2) dynamic analysis and (3) deformation analysis.

In pseudo-static analyses, the earthquake-induced loads on the wall system are represented as static forces and moments. Typically, these design loads and moments are calculated by applying a horizontal inertial force at the center of gravity of the structure or component that is equal to its mass times a seismic coefficient. The seismic coefficient is some fraction of the peak ground acceleration and its value depends upon the response characteristics of the structure, soil and foundation behavior, and the ability of the structure to accommodate displacement.

In a similar manner, the peak inertial loads of a structure, system or component (SSC) may be obtained from a dynamic structural response analysis. These dynamic loads are generally reduced by a load reduction factor and then applied in the same manner as the pseudo-static analysis. Like the seismic coefficient, the load reduction factor depends upon the behavior of the foundation soil and the ability of the SSC to tolerate seismic and permanent displacement.

An alternative to force-based design is deformation-based design where the amount of displacement of the SSC is estimated from analytical, empirical or numerical methods. These methods are usually applied to earthen structures (embankments, dams, earth retaining structures) that have reasonable deformation tolerance. If used, the merits and limitations of the approach should be discussed with MWDSLS. Also, care should be taken to ensure that such analyses have been conservatively performed and provide estimates that are within the displacement tolerance of the particular SSC. The level of conservatism required shall be addressed on a project by project basis with MWDSLS.

1.4.2 Acceptable Factors of Safety

Because seismic loads represent extreme events, relatively low factors of safety are applied to pseudo-static and dynamic analyses. Factors of safety as low as 1.1 are acceptable; however, the choice of the factor of safety is linked to the selection of the seismic coefficient or load reduction factor. Typically, these factors vary from 0.5 for displacement tolerant systems to 1.0 for displacement intolerant systems. Unless otherwise stated, a seismic coefficient or load reduction factor of 1.0 should be used for the evaluations.

1.4.3 Calculation of Peak Horizontal Ground Acceleration

Unless otherwise stated, the peak horizontal ground acceleration (PHGA) shall be calculated as 0.4 times the 0.2-second pseudo-spectral acceleration value for pseudo-static analyses. Also, unless otherwise stated, the peak vertical ground acceleration shall be assumed to be zero.

1.4.4 Analyses for Shallow Foundations

Shallow foundations (e.g., spread, spot or continuous footings) shall be evaluated using pseudo-static or dynamic analyses. The required analyses are: bearing capacity, overturning, sliding, settlement and structural failure due to seismic loading. In addition to these analyses, excessive settlement and lateral spread resulting from potential liquefaction should be evaluated for foundations overlying saturated, cohesionless soils.

There is no general agreement upon the proper seismic coefficient for bearing capacity analysis (FHWA, 1997a,). For SSCs that can tolerate deformations as large as a few to several inches, a seismic coefficient of 0.5 may be used. However, for SSCs that cannot tolerate more than two inches of displacement, then a seismic coefficient of 1.0 is appropriate. No seismic coefficient should be applied for sliding and overturning stability analyses of shallow foundations.

In addition, if pseudo-static analyses are used, potential amplification of horizontal and vertical inertial forces resulting from structural response shall be considered for all analyses.

In addition, both horizontal components (i.e., north-south and east-west) and the vertical component of the inertial forces and/or structural loads should be included when evaluating seismic stability of shallow foundations. However, because it is uncommon for the peak horizontal and vertical loads to occur simultaneously, it is appropriate to use the 100-40-40 rule to combine the maximum responses from each earthquake component (ASCE 4-98). The responses may be combined directly, using the assumption that, when the maximum response from one component occurs, the other responses from the other two components are 40 percent of their maximum value. In this method, all possible combinations of the three components, including variation in the sign or direction, shall be evaluated and the most critical case (i.e., case with the lowest factor of safety) is the design basis case.

Bearing capacity of shallow foundations may be analyzed using pseudo-static or dynamic methods and the general bearing capacity equation with appropriate calculation of the effective footing width resulting from eccentric loading (FHWA, 1997a). Also, to prevent uplifting of the footing edge, a limit is usually set on the allowable eccentricity, e, of the dynamic load. For eccentricities that are less than B/4, where B is the footing width, then the amount of uplift is usually in tolerable limits.

For sliding resistance of shallow foundations, the cohesion and/or frictional resistance at the base of the foundation shall be used to calculate the sliding resistance. For shallowly embedded foundations, the passive earth pressure resistance resulting from embedment is generally neglected. Also, for eccentrically loaded foundations, the effective base area should be used when evaluating sliding resistance.

1.4.5 Analyses for Deep Foundations

Deep foundations are any foundation systems that transfer structural and dynamic loads to considerable depth below the ground surface. These systems are typically drilled, driven or cast-in-place piles, piers or shafts. Deep foundations shall be analyzed for axial, lateral and uplift capacity and settlement. For piles, piers, or shafts that are placed in groups, the potential for capacity reduction resulting from group effects shall be considered. Liquefaction-induced settlement and lateral spread shall also be evaluated for such systems.

Similar to shallow foundations, the dynamic response of a deep foundation can be done using pseudo-static analysis or dynamic response analysis. Peak uplift and compressive forces should be compared with their respective capacities.

For lateral loads, the lateral displacement of the deep foundation group is compared to acceptable levels of lateral displacement. For seismic performance Levels 1 and 2, it is acceptable to allow the deep foundation system to reach the yield state, as long as the resulting deformation is deemed tolerable to the SSC. For seismic performance Level 3,

it is recommended that the deep foundation system be designed not to reach the yield state.
However, the merits of reaching or not reaching the yield state should be discussed by the design team with MWDSLS and should be decided upon on a case-by-case basis. In addition, the deep foundation element and its cap or connections must be designed to accommodate any bending moment and shear that develops.

1.4.6 Analyses for Earth Retaining Structures

In the context of this section, earth retaining structures are considered to be cast-in place concrete (CIPC) gravity, CIPC cantilever, or mechanically stabilized earth (MSE) walls which provide lateral support to an adjacent soil mass. Other types of wall systems (e.g., segment wall systems) shall be evaluated by applicable methods that have been approved by MWDSLS.

Earth retaining structures shall be evaluated for global slope stability, overturning, sliding, bearing capacity, settlement, structural failure, and potential liquefaction effects in the foundation or backfill, as appropriate. In addition, all evaluations shall consider both up and down slope profiles and dynamic water pressures. For MSE wall, both external and internal stability shall be evaluated.

Seismic earth pressures that develop on flexible (i.e., yielding) earth retaining structures shall be designed using seismic active earth pressure theory (e.g., Mononobe-Matsuo, 1926, Seed and Whitman, 1970, Steedman-Zeng, 1990) or other appropriate methods approved by the design team and MWDSLS.

Seismic earth pressures that develop on rigid (i.e., non-yielding) earth retaining structures shall be designed using the method discussed by Wood (1973), or other appropriate methods approved by MWDSLS.

In addition to designing earth retaining structures for loads described above, seiche and sloshing effects shall be evaluated as described in Section 1.9.5.

Seismic stability of MSE walls shall be evaluated using Mononobe-Okabe analysis (Mononobe-Matsuo, 1926) using guidance developed by the Federal Highways Administration (FHWA, 1997b), or other methods approved by the design team and MWDSLS. The active dynamic horizontal thrust calculated for the wall is added to the static forces acting on the wall. Guidance for calculating the horizontal wall acceleration is given in FHWA (1997a) and the vertical acceleration is set equal to zero. The minimum allowable dynamic safety factor is 75 percent of the static safety factors for sliding and overturning (FHWA, 1997b).

Earth retaining structures shall also be evaluated for global stability during the seismic event. In general, seismic performance is evaluated in terms of acceptable deformation instead of pseudo-static slope stability analyses. The analyses methods are similar to those discussed in the embankment section below. Acceptable levels of deformation are to be determined by the service state requirements of adjacent or overlying impacted facilities. However, in no case may the best-estimate displacement exceed 6 inches (horizontal or vertical) for the design ground motion.

1.4.7 Analyses for Buried Structures

Seismic earth pressures that develop on flexible (i.e., yielding) buried structures shall be designed using seismic active earth pressure theory (e.g., Mononobe-Matsuo, 1926; Seed and Whitman, 1970; Steedman-Zeng, 1990), or other appropriate methods approved by the design team and MWDSLS.

Seismic earth pressures that develop on rigid (i.e., non-yielding) buried structures shall be designed using the method discussed by Wood (1973), or other appropriate methods approved by MWDSLS.

1.4.8 Analyses for Embankments

Embankments are generally unreinforced earthen structures such as fill slopes, dams, levees, dikes and impoundments. Analyses required to assess seismic embankment stability include: 1D and 2D ground response analysis, deformation analysis, liquefaction and lateral spread analysis. Finn (1999) and Kramer (1996) give a good summary of these analyses and their progression. Pseudostatic stability analyses should generally be discouraged and should only be used on non-essential facilities. Deformation-based analyses are preferable as long as the analysis has been performed in a conservative manner and captures potential variations in soil properties, earthquake ground motion, and slope geometries. Deformations can be transient displacements caused by the vibratory motion, as well as permanent deformation of the embankment and foundation soils resulting from slope movement and/or foundation failure.

The allowable amount of embankment deformation depends on the type of embankment, its function and the type and proximity of nearby facilities and structures. Allowable deformations should be determined by MWDSLS on a case-by-case basis according to the function of the embankment.

1.5 SEISMIC ANALYSIS OF PIPELINES

1.5.1 Analysis for New Pipelines

In addition to internal pressures, thermal loads, and sustained external static loads, new pipeline design shall consider earthquake loadings including: (1) seismic wave passage, (2) inertia, (3) oscillatory seismic anchor motion, and (4) permanent seismic anchor motion. Wave passage effects are those caused by soil strains from wave propagation. Inertial effects are caused by the vibratory response of the pipe to the design basis earthquake. Oscillatory seismic anchor motion is displacement caused by transient differential movement of anchor points (e.g., building, attachment, supports) and the pipeline. Permanent seismic anchor motion is permanent differential movement resulting from active fault (Holocene age, unless noted otherwise) displacement, dynamic soil settlement, liquefaction, or other seismically induced ground failure (e.g., mass movement, landsliding, etc.).

For new pipelines, seismic design shall be evaluated according to seismic performance Level 3. Seismic design and detailing shall consider, but not be limited to: (1) site-specific active fault (Holocene age, unless noted otherwise) location studies, (2) site-specific active fault (Holocene age, unless noted otherwise) offset studies, (3) site-specific mass movement studies, (4) site-specific liquefaction-induced ground failure and settlement studies, and (5) site-specific response spectra which include near source effects, long-period effects, topographical effects, and basin effects, as appropriate.

Active fault means Holocene age, unless noted otherwise. Active fault studies shall include conceptual design of active fault crossing mitigative measures including: trench geometry and size, pipeline connection types, pipeline supports for above ground reaches, and specialized backfill design for buried reaches. Mass movement and landslide studies, as a minimum, shall include stereographic and topographic evaluations to estimate the landslide hazard. Liquefaction-induced ground failure studies, as a minimum, shall include review of liquefaction hazard maps, surficial geology maps and groundwater depth maps to assess the potential for liquefaction, liquefaction settlement, and lateral spread.

More detailed evaluations, analyses, and mitigative design may be required for potential hazard areas or vulnerable reaches identified by the preliminary assessments. Final design measures to reduce seismic hazards to pipelines shall be determined by MWDSLS upon completion of concept evaluation and cost estimates.

1.5.2 Seismic Analysis of Existing Large Diameter Pipelines (36" and greater)

For existing pipelines, seismic evaluations shall be done according to seismic performance Levels 2 and 3. Active fault means Holocene age, unless noted otherwise.

Seismic evaluations and retrofit designs shall incorporate site-specific seismic conditions including:

a. Site-specific active fault locations (for location of active fault crossing design areas).
b. Site-specific active fault offsets (for piping retrofit and trench design).
c. Site-specific landslides and ground failures.
d. Site-specific permanent ground offsets due to causes other than active faulting.
e. Site-specific response spectra (where appropriate for use in above ground segment design).
f. Active fault crossing design measures shall address as a minimum: trench geometry and size, pipeline and connection types, and specialized backfill design.
g. Liquefaction.

These analysis results, and recommended retrofit design measures, shall be submitted to MWDSLS for review. Final design measures to reduce seismic hazards for pipelines shall be determined by MWDSLS upon completion of concept evaluation and cost estimates.

1.6 EMERGENCY RESPONSE PLAN

Keep the emergency response plan (ERP) updated. Coordinate with Salt Lake City, Sandy City, and FEMA, now Department of Homeland Security (DHS). The ERP shall conform to DHS requirements, now in development.

1.7 PERFORMANCE CRITERIA TABLES

Table 1-1: Seismic Performance Standards and Associated Design Ground Motions

MWDSLS Seismic Performance Standards	Design Ground Motions	Design Standard for New Design	Design Standard for Retrofit Design
Level 1	Larger of: (a) $^{2}/_{3}$ x [Site-Specific Spectra (2% in 50 yrs)] x 1.00 importance factor(c) or (b) [IBC 2003(b) (2% in 50 yrs)](a) x 1.00 importance factor(c) x 80%(e)	IBC 2003(c)	▨
	10% in 50 yrs site-specific horizontal only	▨	Larger of FEMA 356 Level k or
	2% in 50 yrs site-specific horizontal only	▨	FEMA 356 Level p
Level 2	Larger of: (a) $^{2}/_{3}$ x [Site-Specific Spectra (2% in 50 yrs)] x 1.25 importance factor(c) or (b) [IBC 2003(b) (2% in 50 yrs)](a) x 1.25 importance factor(c) x 80%(e)	IBC 2003(c)	▨
	10 % in 50 yrs site-specific horizontal and vertical(d), (f)	▨	Larger of FEMA 356 Level j or
	2% in 50 yrs site-specific horizontal and vertical(d), (f)	▨	FEMA 356 Level o
Level 3	Larger of: (a) $^{2}/_{3}$ x [Site-Specific Spectra (2% in 50 yrs)] x 1.5 importance factor(c) or (b) [IBC 2003(b) (2% in 50 yrs)](a) x 1.5 importance factor(c) x 80%(e)	IBC 2003(c)	▨
	10 % in 50 yrs site-specific horizontal and vertical(d), (f)	▨	Larger of FEMA 356 Level i or
	2% in 50 yrs site-specific horizontal and vertical(d), (f)	▨	FEMA 356 Level n

(a) Contains a reduction factor of 2/3

(b) International Building Code 2003 (IBC 2003). Use "Seismic Design Parameters" by Leyendecker, et al, USGS Open File Report 01-437 . Do <u>not</u> use USGS national hazard map values for 2% in 50 years listed on the USGS web-site.

(c) Table 1604.5 of the IBC 2003 applies an Importance Factor of 1.25 for all facilities for the treatment of potable water, and an Importance Factor of 1.50 for the portion of a system that is required to maintain water pressure for fire suppression purposes. Level 2 and Level 3 performance standards (above) incorporate these Importance Factors.

(d) For use of site-specific vertical response spectra, refer to Section 1.8 of this criteria document.

(e) If site-specific spectrum is not available for comparison to IBC 2003 spectrum, do not use 80% factor x IBC 2003 spectrum.

(f) No 2/3 reduction is allowed on vertical spectra (retrofit design only, at the discretion of design engineer for new design).

For sites other than Little Cottonwood Water Treatment Plant and the Point of the Mountain Water Treatment Facility, where site specific response spectra has been developed (see Sections 1.10 and 1.11), the IBC 2003 mapped spectral accelerations shall be employed. The IBC 2003 maps are based on the National Earthquake Hazard Reduction Program (NEHRP) Maps 1997, by Leyendecker, et al 1997. In addition, the mapped spectral accelerations based on NEHRP Maps 2000, by Leyendecker, et. al 2000, shall be determined. The controlling spectral values shall be used. If the controlling spectral value is determined from the NEHRP Maps 2000, MWDSLS shall be notified, as well as a written variance from the jurisdiction issuing building permits shall be obtained.

Note the NEHRP Maps 2000 are the updated/refined 1997 Maps. The NEHRP Maps 2000 are to be published in the forthcoming (Fall 2004) draft version of the IBC 2006.

Table 1-2 FEMA 356 Seismic Retrofit Criteria

		Target Building Performance Levels			
		Operational Performance Level (1-A)	*Immediate Occupancy Performance Level (1-B)*	*Life Safety Performance Level (3-C)*	*Collapse Prevention Performance Level (5-E)*
Earthquake Hazard Level	*50%/50 year*	*a*	*b*	*c*	*d*
	20%/50 year	*e*	*f*	*g*	*h*
	BSE-1(3) *(≅10%/50 year)*	*i*	*j*	*k*	*l*
	BSE-2(3) *(≅2%/50 year)*	*m*	*n*	*o*	*p*

Notes:

1. *Each cell in the above matrix represents a discrete Rehabilitation Objective.*
2. *The Rehabilitation Objectives in the matrix above may be used to represent the three specific Rehabilitation Objectives defined in sections 1.3.1, 1.4.2, and 1.4.3 of FEMA 356.*
3. *BSE = Base Seismic Event.*

k + p = Basic Safety Objective (BSO)
k + p + any of a, e, i, b, f, j, or n = Enhanced Objectives
o alone or n alone or m alone = Enhanced Objective
k alone or p alone = Limited Objective
c, g, d, h, l = Limited Objective

1.8 USE OF VERTICAL RESPONSE SPECTRA

Vertical response spectra shall be utilized as described in Tables 1-1-1 and 1-1-2.

1.8.1 Level 2 Analysis

For a “Level 2” performance standard, vertical response spectra shall be used. An independent vertical analysis shall be performed only for structural elements particularly vulnerable to vertical excitations. These items shall include, but not be limited to:

a. Cantilever elements
b. Floor spans exceeding 40’ in any direction
c. Connections for precast concrete elements
d. Precast or prestressed concrete floor or roof elements with or without a topping slab
e. Other elements as identified by MWDSLS or the reviewing structural engineer.

1.8.2 Level 3 Analysis

For a “Level 3” performance standard, vertical response spectra shall be used as follows:

1.8.2.1 Analysis of Particularly Vulnerable Elements

Analysis of particularly vulnerable elements listed in 1.8.1 a. – e. shall be performed.

1.8.2.2 Overall Vertical Analysis

In very limited cases, a complete vertical analysis shall be performed and combined with lateral analysis results. Results shall be combined using a rational procedure such as SRSS (Square Root of the Sum of the Square) on vertical and lateral loads or CQC (Complete Quadratic Combination). This type of analysis shall only be required on very limited highly critical structures identified by MWDSLS.

1.9 BASINS AND SPECIALIZED STRUCTURES

1.9.1 Accelerations

Accelerations shall be utilized as in Tables 1--1 and 1--2 for liquid containing structures in conjunction with their impulsive and convective hydrodynamics properties.

1.9.2 New Construction Shall Conform to:

1.9.2.1 ACI 350-01, ACI 350.3.01 for Environmental Engineered Concrete Structures

1.9.2.2 AWWA D100, D110, D115, and API 650 for Steel and Concrete Tank Structures

1.9.3 Existing Structures

Existing Structures shall be evaluated using FEMA 356 methodology and the loading obtained from either the ACI 350.03-01 or AWWA standards.

1.9.3.1 Steel Tanks

Steel Tanks shall be compared to the acceptance criteria for new construction in AWWA D100 or API 650.

1.9.3.2 Concrete Basins and other Liquid Containing Concrete Structures

Concrete Basins shall be evaluated using the FEMA criteria with the following criteria for concrete basin walls with reinforcing each way on each face (see Table 1--3).

Table 1-3 FEMA 356 Seismic Retrofit Criteria

	Maximum Basin "m" Values *(3),(4)*		
	Operational Performance Level (1-A)	*Immediate Occupancy Performance Level (1-B)*	*Life Safety Performance Level (3-C)*
Basin Dividing Walls (1)	*3*	*5*	*7*
Basin Perimeter Wall (2)	*2*	*3*	*5*

Notes:

1. *Dividing wall separates basins.*
2. *Perimeter walls separate basins from earth or other process areas where leakage could be more critical.*
3. *Testing must be done for all existing basins to verify size, spacing and condition of existing reinforcement.*
4. *"m" is a FEMA 356 effective ductility factor (see FEMA 356).*

1.9.4 Lateral Force Analysis of Basins and Specialized Structures (Earth and Concrete)

In the context of this section, basin structures are considered to be earthen embankments, which impound water. Evaluation of impoundment structures shall be conducted in accordance with the Section 1.4.6 Analyses for Earth Retaining Structures. In addition to the provisions herein, basin structures shall be designed to comply with dam safety regulations, as may be required by the State Engineer.

Unless otherwise stated, the peak horizontal ground acceleration (PHGA) shall be assumed to be 0.4 times the 0.2-second spectral acceleration including site effects, and the peak vertical ground acceleration (PVGA) for evaluating basin structures shall be assumed to be zero.

Evaluations of basin structures shall consider effects of hydrodynamic loading and soil strength loss due to the effects of earthquake shaking (e.g., liquefaction) or seismically induced foundation loading. Additionally, all evaluations shall consider surrounding slope profile geometries.

Basin structures shall be evaluated with respect to global stability due to seismic loading. In order to more accurately quantify seismic performance, deformation-based stability analyses shall be performed in lieu of pseudo-static slope stability analyses. Acceptable levels of deformation are to be determined by the service state requirements of adjacent or otherwise impacted facilities (particularly liner systems), but in no case greater than a best-estimate value of 12 inches for the design ground motion level. For Level 1 and 2 seismic design loads (and preliminary assessments involving Level 3 seismic design loads), correlations between Newmark-type displacements and the ratio of yield acceleration to maximum acceleration may be used. For Level 3 seismic design loads, more rigorous analysis methods (e.g., dynamic finite element) shall be used. Deformation evaluations shall also consider potential displacements due to active fault (Holocene age, unless noted otherwise) rupture, liquefaction-induced settlement, and lateral spreading.

1.9.5 Seiche and Sloshing Analysis of Sloped Wall Basins (Earth and Concrete) – Design Criteria to be Developed

Specifically, this methodology shall address seiche and sloshing effects at POMWTP. This is a problem that is not specifically defined in current code documents and therefore must be investigated on a site-by-site basis. The engineering committee shall retain a consultant to develop technical criteria for this type of design.

1.10 SITE-SPECIFIC RESPONSE SPECTRA FOR LCWTP SITE, DESIGN LEVELS 1, 2 AND 3

Site-specific response spectra for the LCWTP site are given in Figure 1--1 and Figures/Tables 1--4 through 1--7. A horizontal spectrum only is given for Level 1. Horizontal and vertical spectra are given for Levels 2 and 3. These spectra have been determined based upon site-specific data available as of the date of the publication of this criteria document.

Factors evaluated in development of these spectra were:

a. Shear-wave velocity profile
b. Depth to rock
c. Non-linear dynamic properties (shear modulus reduction and damping curves)
d. Active fault (Holocene age, unless noted otherwise) rupture directivity
e. Salt Lake Valley “basin – effects” for long-period ground motions
f. Vertical to horizontal (v/h) ratios.

Figure 1-1: Site-Specific Horizontal Spectra Level 1 (IBC 2003)

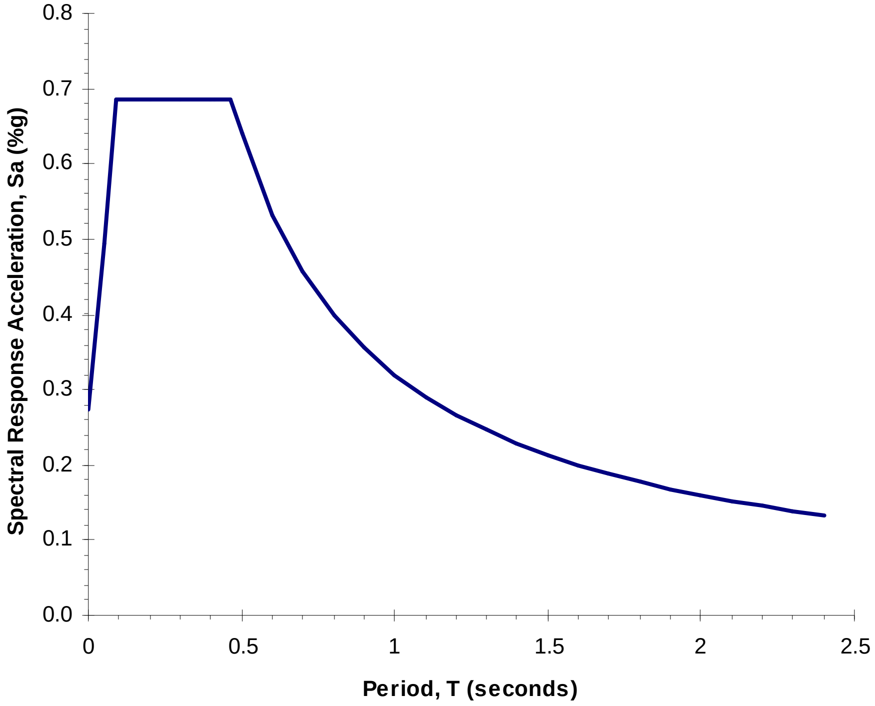

Figure 1-2
Figure 1-3

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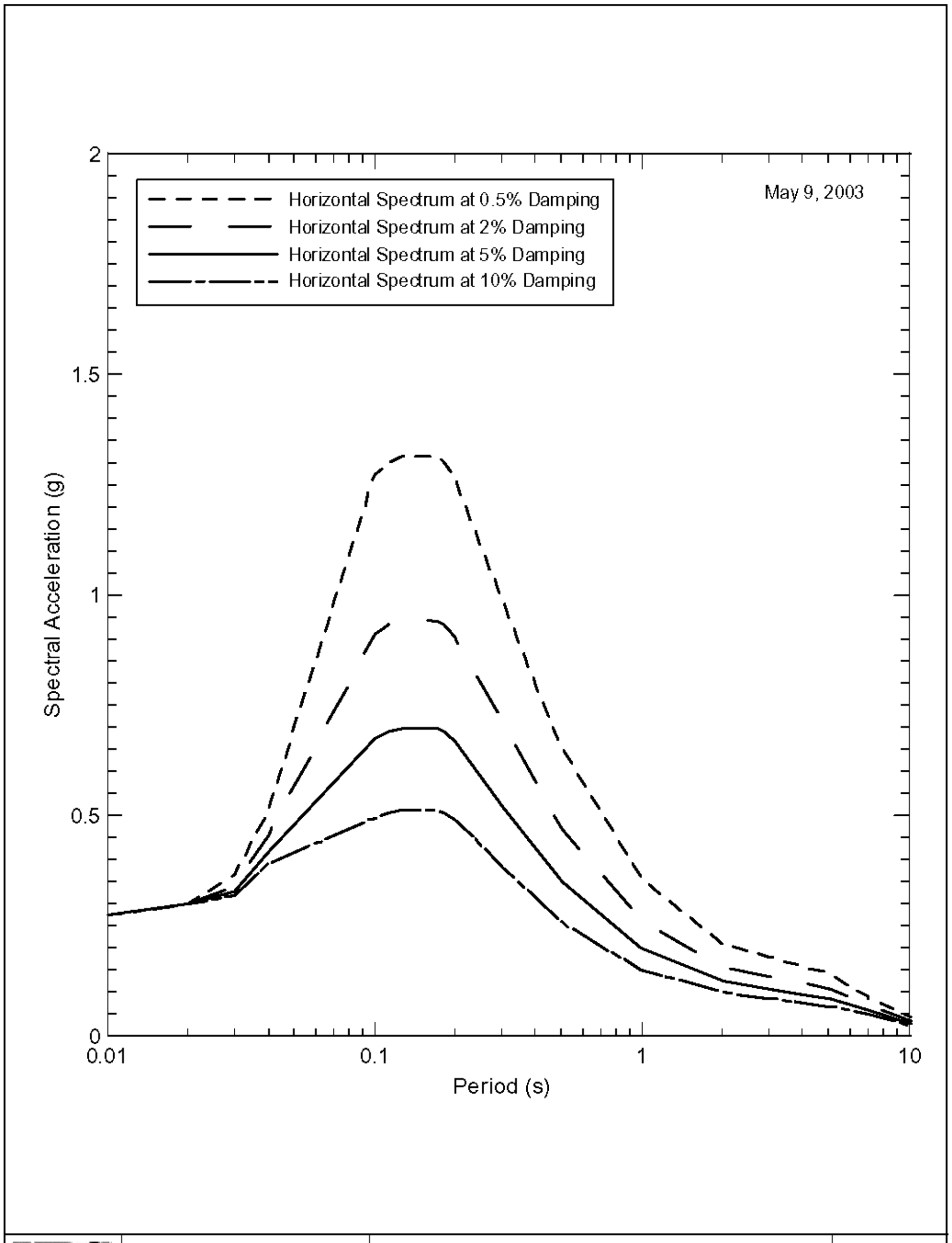
Figure 1-4: LCWTP Site-Specific Horizontal Spectra (10% in 50 years)



URS

Little Cottonwood
Water Treatment Plant

Project No. 26814172.01564

LITTLE COTTONWOOD SITE-SPECIFIC
HORIZONTAL SPECTRA
10% In 50 Years Exceedance Probability

Figure
1-13-4

Table 1-4
Recommended LCWTP Site-Specific Horizontal Spectra
(10% in 50-Year Exceedance Probability)
(May 9, 2003)

	Dampings			
Period (sec)	**0.5%**	**2.0%**	**5.0%**	**10.0%**
0.01	0.275	0.275	0.275	0.275
0.03	0.367	0.345	0.330	0.319
0.1	1.272	0.913	0.675	0.495
0.2	1.263	0.906	0.670	0.492
0.3	0.980	0.703	0.520	0.381
0.5	0.651	0.470	0.350	0.259
1.0	0.360	0.264	0.200	0.152
2.0	0.209	0.158	0.125	0.100
3.0	0.180	0.136	0.108	0.086
4.0	0.159	0.120	0.095	0.076
5.0	0.142	0.108	0.085	0.068
10.0	0.045	0.036	0.030	0.025

Figure 1-5: LCWTP Site-Specific Vertical Spectra (10% in 50 years)

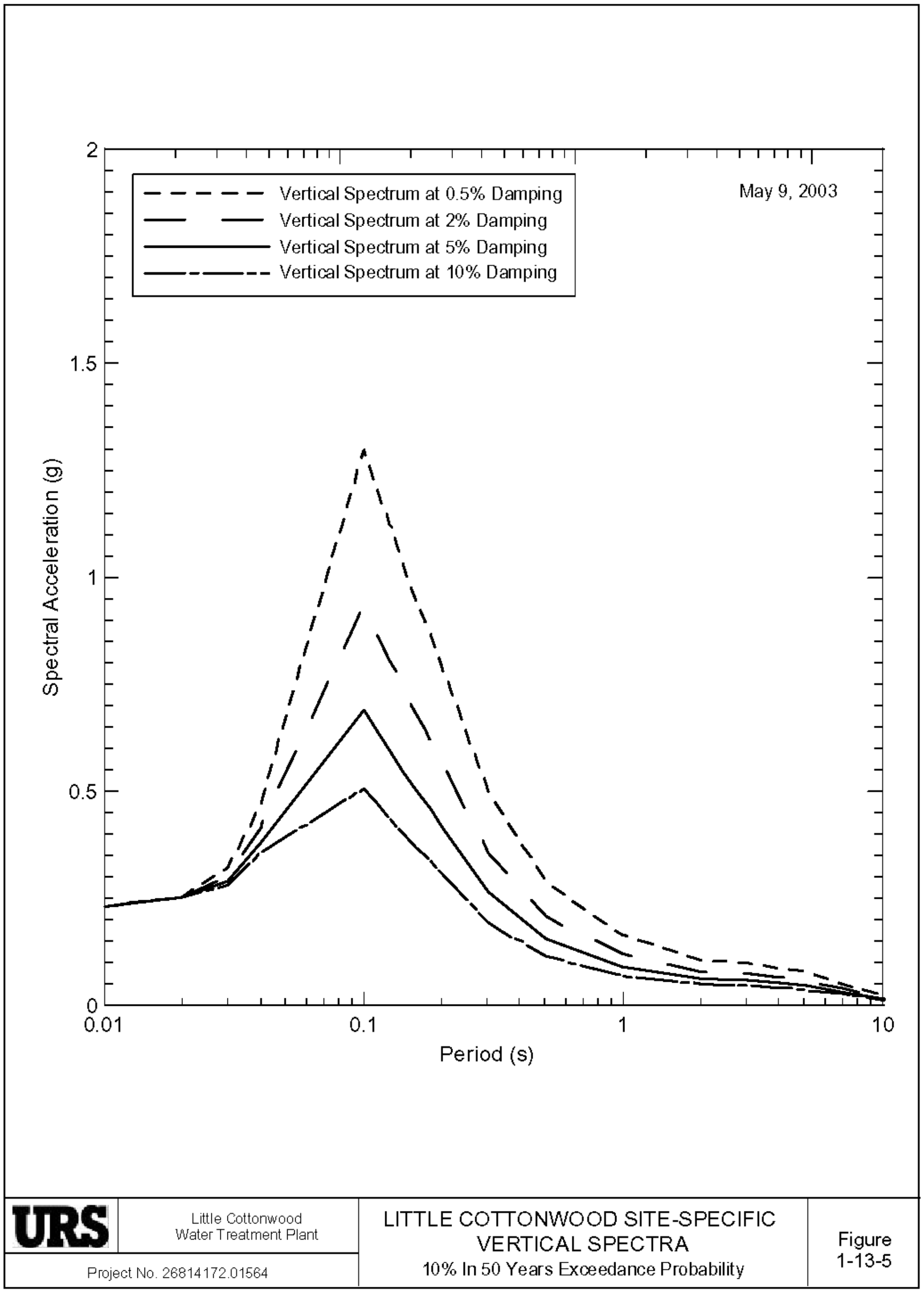


Table 1-5
Recommended LCWTP Site-Specific Vertical Spectra
(10% in 50-Year Exceedance Probability)
(May 9, 2003)

	Dampings			
Period (sec)	**0.5%**	**2.0%**	**5.0%**	**10.0%**
0.01	0.233	0.233	0.233	0.233
0.03	0.323	0.304	0.291	0.282
0.1	1.298	0.931	0.689	0.505
0.2	0.792	0.568	0.420	0.308
0.3	0.499	0.358	0.265	0.194
0.5	0.291	0.210	0.156	0.116
1.0	0.166	0.121	0.092	0.070
2.0	0.106	0.081	0.064	0.051
3.0	0.100	0.076	0.060	0.048
4.0	0.089	0.067	0.053	0.042
5.0	0.079	0.060	0.047	0.038
10.0	0.025	0.020	0.017	0.014

Figure 1-6: LCWTP Site-Specific Horizontal Spectra (2% in 50 years)

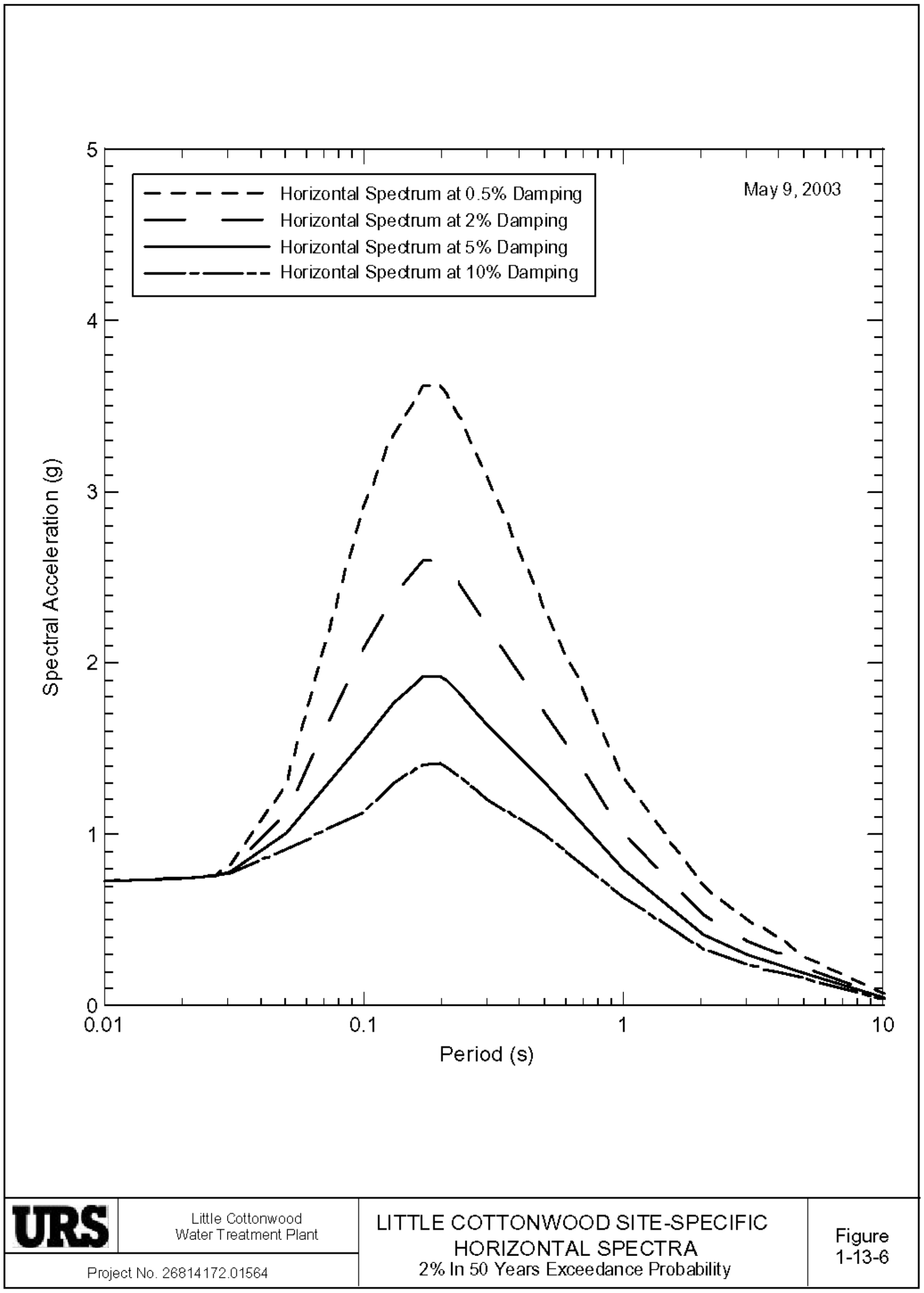


Table 1-6
Recommended LCWTP Site-Specific Horizontal Spectra
(2% in 50-Year Exceedance Probability)
(May 9, 2003)

	Dampings			
Period (sec)	**0.5%**	**2.0%**	**5.0%**	**10.0%**
0.01	0.731	0.731	0.731	0.731
0.03	0.818	0.797	0.783	0.773
0.1	2.896	2.078	1.536	1.127
0.2	3.623	2.599	1.922	1.410
0.3	3.099	2.223	1.644	1.206
0.5	2.321	1.709	1.305	1.000
1.0	1.331	1.010	0.797	0.636
2.0	0.702	0.532	0.420	0.335
3.0	0.501	0.380	0.300	0.239
4.0	0.407	0.309	0.244	0.194
5.0	0.284	0.226	0.187	0.158
10.0	0.073	0.058	0.048	0.041

Figure 1-7: LCWTP Site-Specific Vertical Spectra (2% in 50 years)

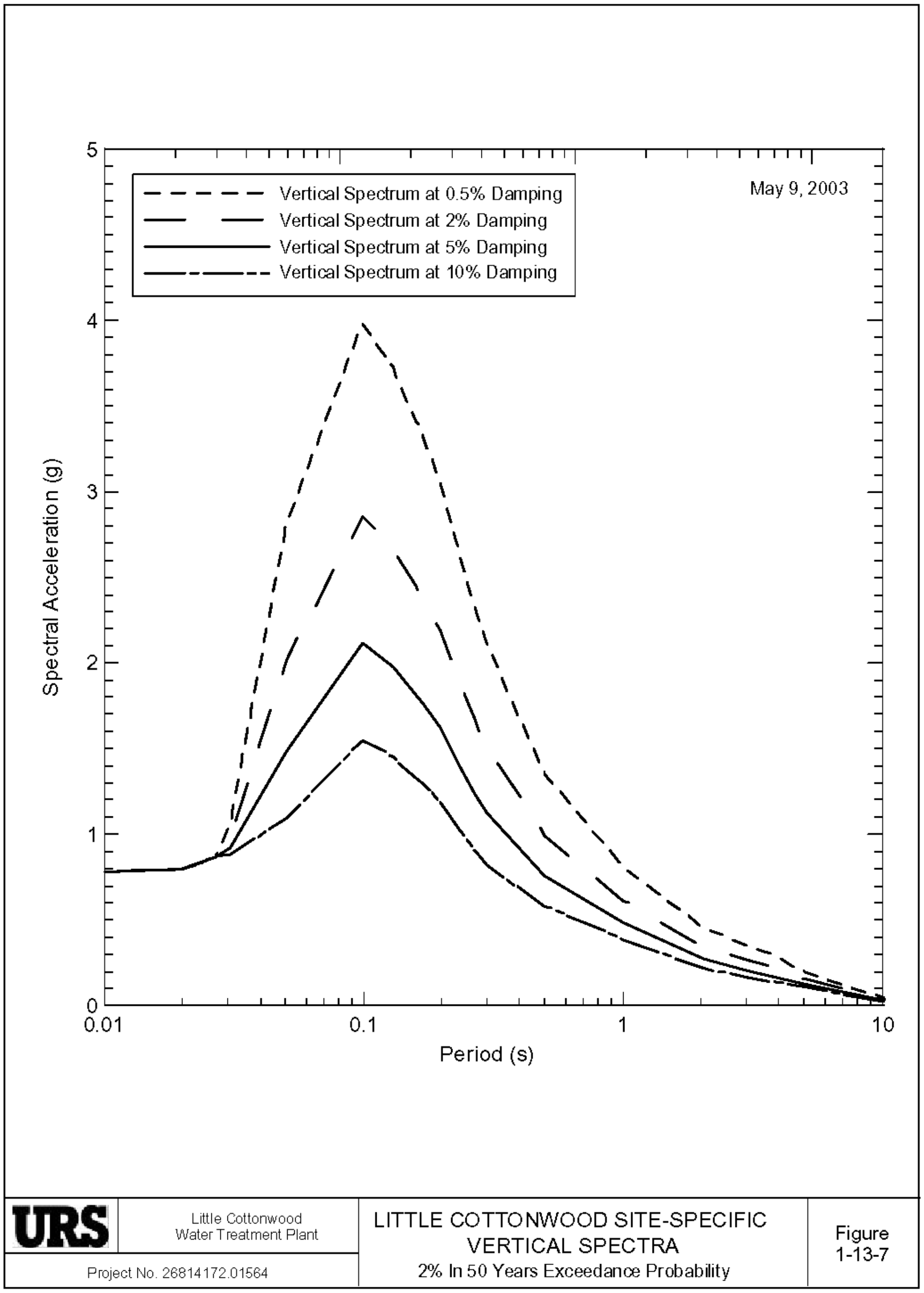


Table 1-7
Recommended LCWTP Site-Specific Vertical Spectra
(2% in 50-Year Exceedance Probability)
(May 9, 2003)

	Dampings			
Period (sec)	**0.5%**	**2.0%**	**5.0%**	**10.0%**
0.01	0.782	0.782	0.782	0.782
0.03	1.048	0.973	0.923	0.885
0.1	3.982	2.857	2.113	1.550
0.2	2.909	2.087	1.543	1.132
0.3	2.115	1.517	1.122	0.823
0.5	1.353	0.997	0.761	0.583
1.0	0.810	0.614	0.485	0.387
2.0	0.458	0.347	0.274	0.219
3.0	0.349	0.264	0.209	0.167
4.0	0.283	0.215	0.170	0.135
5.0	0.198	0.157	0.130	0.110
10.0	0.051	0.040	0.033	0.028

1.11 POMWTP FACILITY SITE-SPECIFIC RESPONSE SPECTRA, DESIGN LEVELS 1, 2 AND 3

Site-specific response spectra for the POMWTP site (see Figures/Tables 1-8 through 1-11) were developed in a manner similar to that described in Section 1.10. Specifically, spectra were developed based on a site-specific probabilistic seismic hazard analysis for an exceedance probability for 2% in 50 year and 10% in 50 year events. Both horizontal and vertical spectra were developed.

As in Section 1.10, site-specific spectra for the POMWTP site included:

a. Shear-wave velocity profile
b. Depth to rock
c. Non-linear dynamic properties (shear modulus reduction and damping curves)
d. Active fault (Holocene age, unless noted otherwise) rupture directivity
e. Salt Lake Valley “basin – effects” for long-period ground motions
f. Vertical to horizontal (v/h) ratios.

Figure 1--8: POMWTP Site-Specific Horizontal Spectra (10% in 50 years)

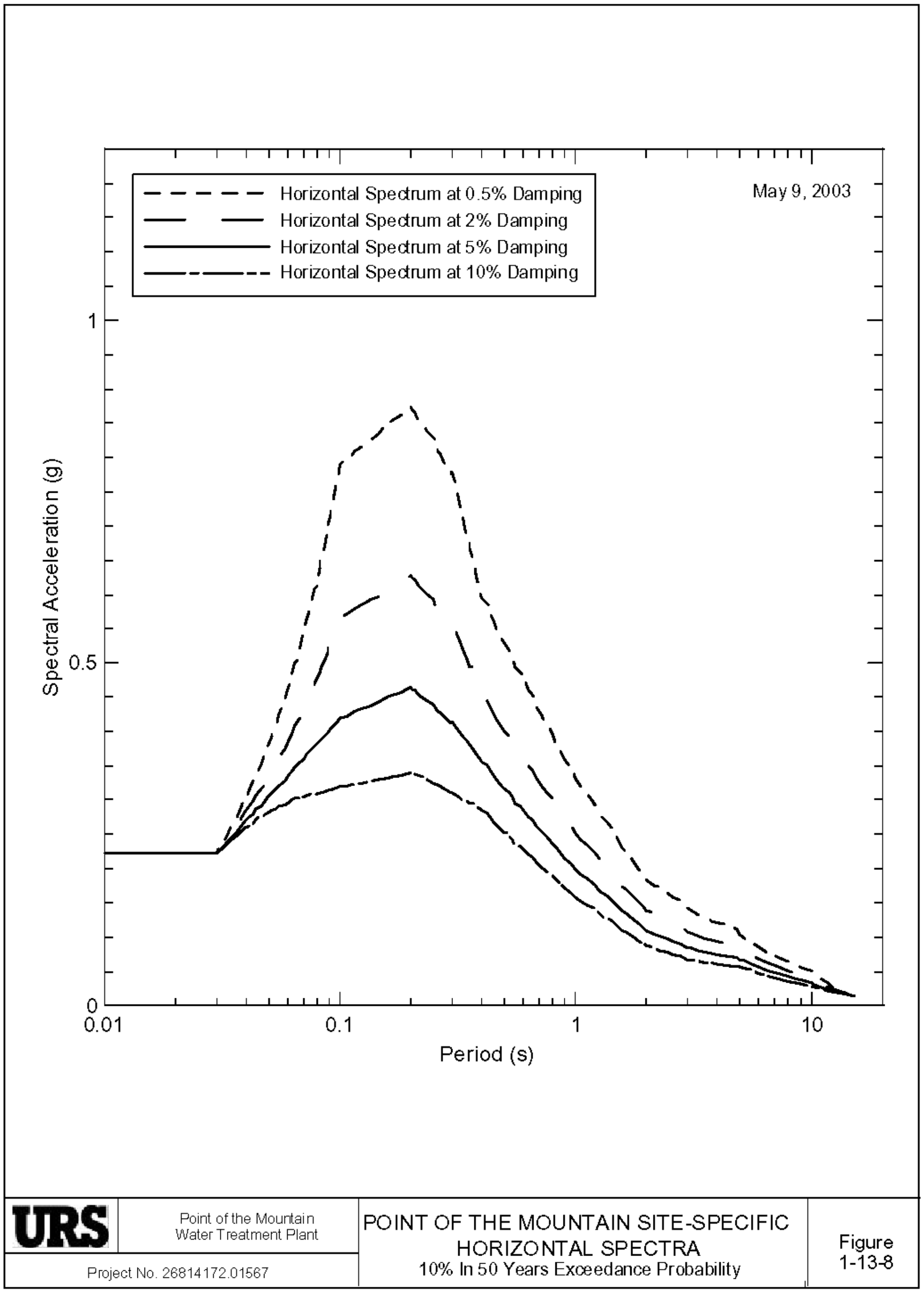


Table 1-8
Recommended POMWTP Site-Specific Horizontal Spectra
(10% in 50-Year Exceedance Probability)
(May 9, 2003)

Period (sec)	**Dampings**			
	0.5%	**2.0%**	**5.0%**	**10.0%**
0.01	0.222	0.222	0.222	0.222
0.03	0.222	0.222	0.222	0.222
0.1	0.791	0.567	0.420	0.320
0.2	0.875	0.627	0.464	0.340
0.3	0.779	0.559	0.413	0.310
0.5	0.528	0.401	0.316	0.252
1.0	0.332	0.252	0.199	0.159
2.0	0.183	0.139	0.110	0.088
3.0	0.142	0.108	0.085	0.068
4.0	0.121	0.093	0.075	0.061
5.0	0.103	0.082	0.068	0.057
10.0	0.052	0.041	0.034	0.029
15.0	0.015	0.015	0.015	0.015

Figure 1-9: POMWTP Site-Specific Vertical Spectra (10% in 50 years)

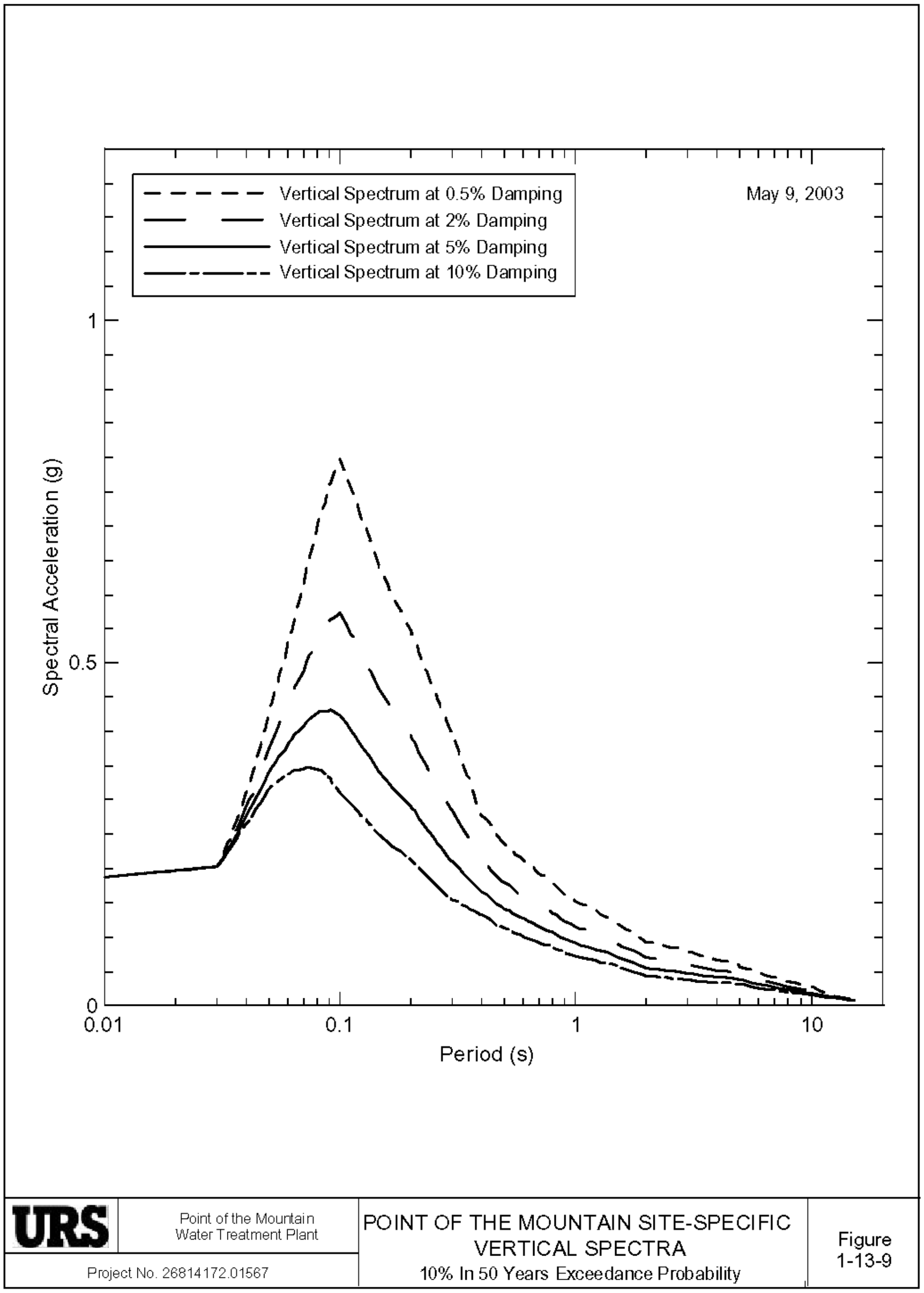


Table 1-9
Recommended POMWTP Site-Specific Vertical Spectra
(10% in 50-Year Exceedance Probability)
(May 9, 2003)

	Dampings			
Period (sec)	**0.5%**	**2.0%**	**5.0%**	**10.0%**
0.01	0.188	0.188	0.188	0.188
0.03	0.203	0.203	0.203	0.203
0.1	0.799	0.573	0.424	0.311
0.2	0.549	0.394	0.291	0.214
0.3	0.398	0.286	0.211	0.155
0.5	0.236	0.179	0.141	0.133
1.0	0.153	0.116	0.092	0.073
2.0	0.093	0.071	0.056	0.045
3.0	0.079	0.060	0.047	0.038
4.0	0.068	0.052	0.042	0.034
5.0	0.058	0.046	0.038	0.032
10.0	0.029	0.023	0.019	0.016
15.0	0.008	0.008	0.008	0.008

Figure 1-10: POMWTP Site-Specific Horizontal Spectra (2% in 50 years)

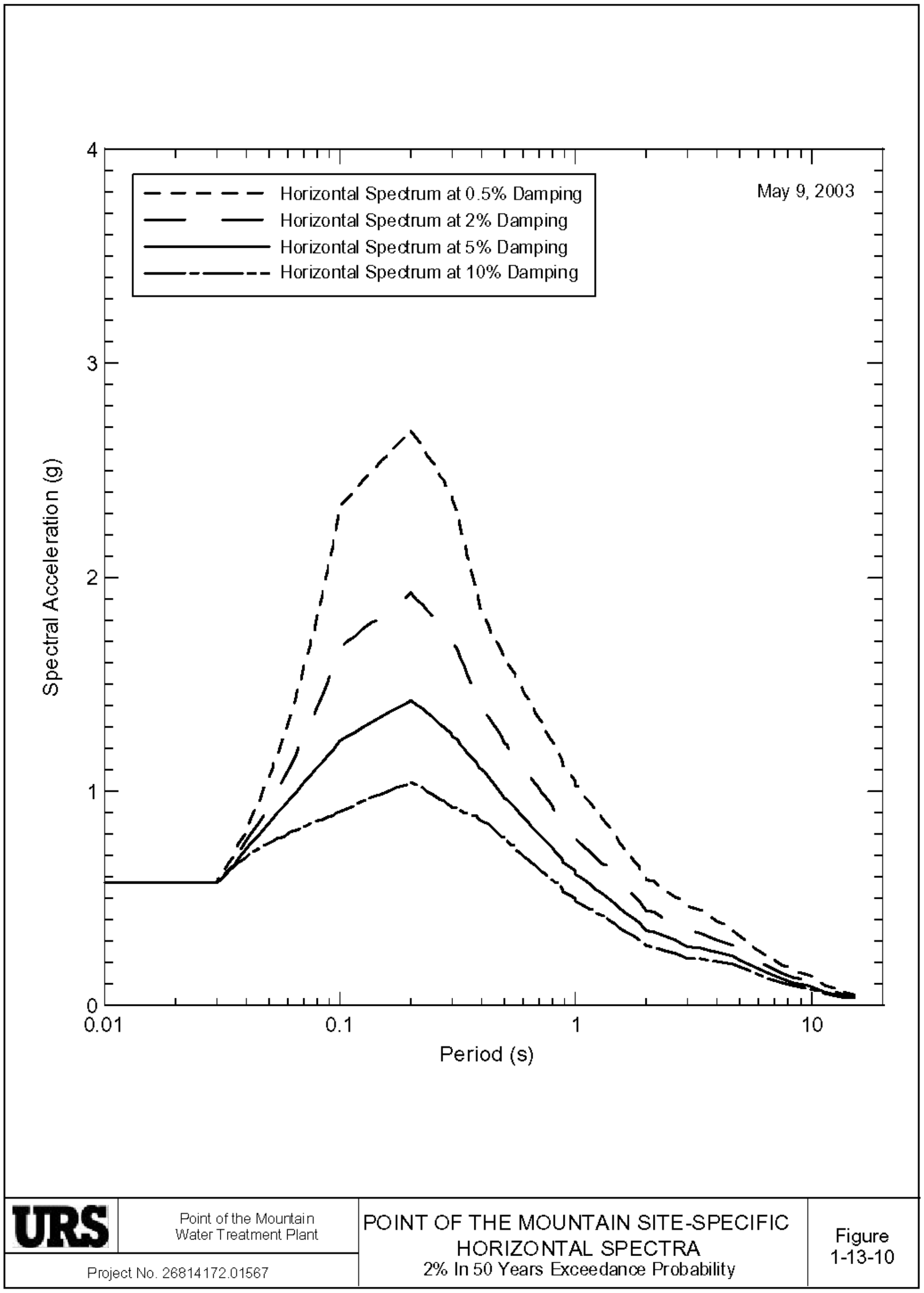
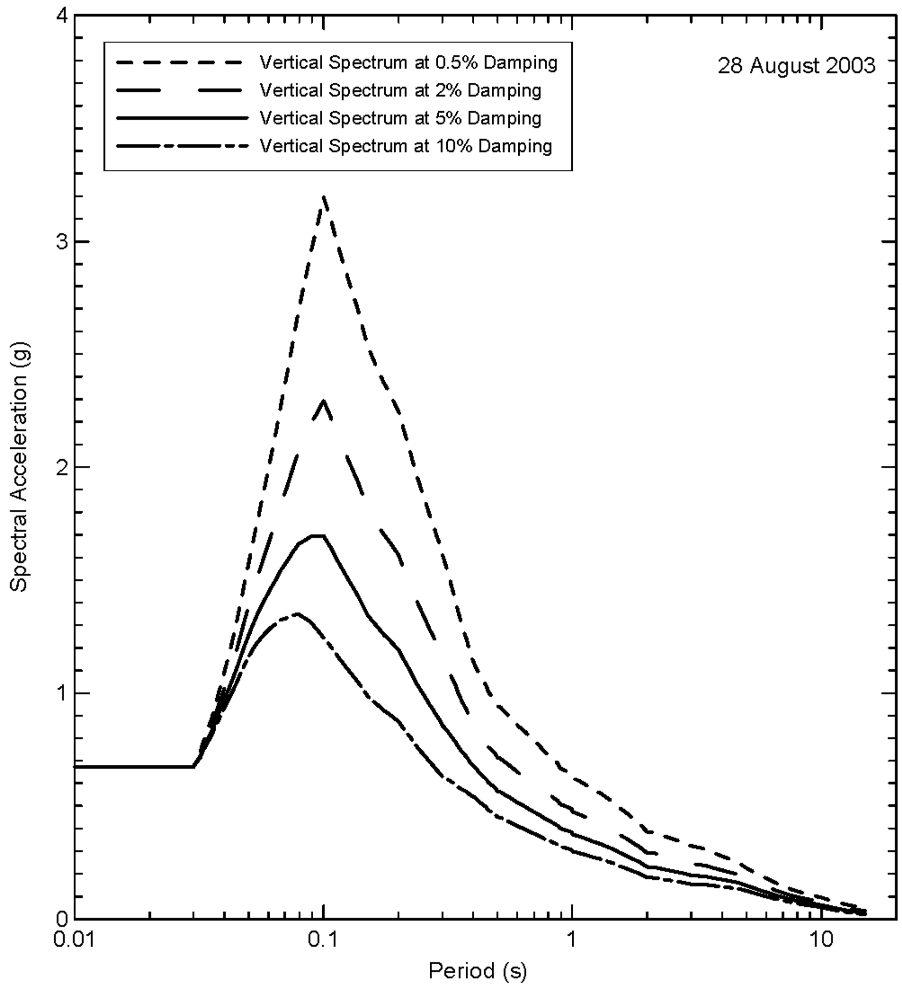


Table 1-10
Recommended POMWTP Site-Specific Horizontal Spectra
(2% in 50-Year Exceedance Probability)
(May 9, 2003)

	Dampings			
Period (sec)	**0.5%**	**2.0%**	**5.0%**	**10.0%**
0.01	0.575	0.575	0.575	0.575
0.03	0.575	0.575	0.575	0.575
0.1	2.333	1.674	1.238	0.908
0.2	2.687	1.928	1.426	1.046
0.3	2.369	1.699	1.257	0.922
0.5	1.615	1.225	0.967	0.772
1.0	1.026	0.778	0.614	0.490
2.0	0.590	0.448	0.353	0.282
3.0	0.464	0.352	0.278	0.222
4.0	0.398	0.309	0.250	0.205
5.0	0.324	0.257	0.213	0.180
10.0	0.137	0.109	0.090	0.076
15.0	0.050	0.044	0.040	0.037

Figure 1-11: POMWTP Site-Specific Vertical Spectra (2% in 50 years)



URS	Point of the Mountain Water Treatment Plant	POINT OF THE MOUNTAIN SITE-SPECIFIC VERTICAL SPECTRA 2% In 50 Years Exceedance Probability	Figure 1-13-11
Project No. 26814172.01567			

Table 1-11
Recommended POMWTP Site-Specific Vertical Spectra
(2% in 50-Year Exceedance Probability)
(May 9, 2003)

Period (sec)	Dampings			
	0.5%	**2.0%**	**5.0%**	**10.0%**
0.01	0.673	0.673	0.673	0.673
0.03	0.673	0.673	0.673	0.673
0.1	3.196	2.293	1.696	1.244
0.2	2.247	1.612	1.192	0.874
0.3	1.613	1.157	0.856	0.628
0.5	0.942	0.714	0.564	0.450
1.0	0.624	0.473	0.374	0.298
2.0	0.384	0.291	0.230	0.184
3.0	0.323	0.245	0.193	0.154
4.0	0.283	0.218	0.176	0.144
5.0	0.225	0.179	0.148	0.125
10.0	0.095	0.076	0.063	0.053
15.0	0.035	0.031	0.028	0.026

1.12 UTILIZATION OF APPROPRIATE STRUCTURAL DAMPING PERCENT FOR EXISTING FACILITIES ONLY

Site-specific response spectra have been provided for multiple damping levels. For new construction, selection of damping values shall be based upon the values allowed in appropriate building codes previously referenced. Utilization of alternate damping values shall be allowed if demonstrated by accepted engineering principles and approved by MWDSLS.

For existing structures, general guidelines for selection of appropriate damping shall be based on FEMA 356, Chapter 1. It is recognized however, that for higher damage states (Life Safety only), higher damping values may be appropriate. Damping values shall be based on established engineering principles and approved by MWDSLS.

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