WATTS BAR NUCLEAR PLANT

SEISMIC ANALYSIS

CORRECTIVE ACTION PROGRAM PLAN

REVISION 1

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1	As a result of the NRC /TVA February 7-8, 1989 meeting on WBN CAPs, revision 1 was made to establish criteria for evaluation (Set B) and criteria for new design/modification (the envelope of B and C). (Affected pages - ii, 1-24, 26, and 27) Additional changes were made for clarification purposes. Changes made to this document in this revision are identified with a bar in the right margin of the text.	06/22/89

SEISMIC ANALYSIS

CORRECTIVE ACTION PROGRAM PLAN

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SEISMIC ANALYSIS

CORRECTIVE ACTION PROGRAM (CAP) PLAN

1.0 INTRODUCTION

The seismic design basis for Watts Bar Nuclear Plant (WBN) (Reference 1) is the Modified Newmark design spectrum anchored at 0.18 g horizontal and 0.12 g vertical for the Safe Shutdown Earthquake (SSE). The Operating Basis Earthquake (OBE) is equal to one-half the SSE. The design basis spectrum was confirmed to be an acceptable design basis by comparison with the Site Specific Response Spectra developed in 1979. The seismic design basis was documented in the WBN Final Safety Analysis Report (FSAR) and the NRC review and acceptance was documented in the WBN Safety Evaluation Report (SER). (SER is based on FSAR through Amendment 46).

An independent review of the seismic analysis calculations for Seismic Category I structures was initiated in September 1987 as part of the Civil Calculation Activity of the Design Baseline Verification Program. The Civil Calculation Activity is being performed to ensure that essential civil calculations exist, are retrievable, and are technically adequate. The seismic analysis calculations were selected for an early review to ensure that the analysis and the resulting Amplified Response Spectra (ARS) used for seismic design of structures, systems, and components are technically adequate and satisfy licensing requirements.

Based on this review, certain aspects of the structural seismic analysis were identified as requiring further evaluation and justifications.

An area of seismic analysis methodology was also identified from the WBN Employee Concern Program which required additional evaluation. The concern is related to the time interval of integration used for performing seismic analyses. Also, three CAQRs identified issues related to soil properties used in seismic analyses and consideration of soil and pile interaction effects. The employee concern, CAQRs, and their brief descriptions are provided in Attachment 1. The issues identified from the calculation review, employee concern, and CAQRs are tabulated in Table 1.

To complement the calculation review, a comparison of the seismic criteria used in the analysis of structures with the FSAR commitments and SER provisions was initiated in July 1988. The purpose of this activity was to assure that the criteria used in the original seismic analysis of structures are technically adequate and consistent with the licensing requirements. The matrix comparing the seismic analysis criteria, FSAR and SER for Seismic Category I structures is shown in Table 2.

A review of Table 2 indicates that the seismic analysis criteria used in original analysis of WBN structures are consistent with the FSAR and SER. Due to the issues identified in Table 1, an evaluation of several Category I structures is planned to assure that the original seismic analysis of WBN is adequate.

The root cause of the issues identified in this CAP is attributed to the use of engineering judgments in the original seismic analysis without supporting documentation.

2.0 OBJECTIVE

The objectives of this CAP are to ensure that the criteria for and the seismic analysis of Category I structures, including the generation of the structural loads and ARS, are technically adequate and meet licensing requirements. Based on the results of the review thus far, some revisions will be necessary to the design criteria and the FSAR. Licensing commitment changes will be proposed only when technically justified.

3.0 SCOPE

The scope of this CAP includes:

- Review, revision, and augmentation of the seismic analysis criteria used for Category I structures to assure compliance with the licensing requirements.
- Review, revision, and augmentation of seismic calculations for Category I structures as required to resolve the issues identified in this CAP.
- Define the seismic criteria for future evaluations and new designs or modifications of structures, systems, and components.

4.0 DESCRIPTION OF PROGRAM

This CAP consists of the following activities:

- Review of seismic analysis criteria and licensing requirements for Category I structures.
- Review of seismic analysis calculations for Category I structures and revisions as required, or preparation of new calculations when necessary.
- Disposition of identified issues.
- Definition of the seismic criteria for future evaluations and new designs or modifications of structures, systems, and components.

Additionally, recurrence control is addressed and licensing assessment is provided. A flow chart and fragnet for the work are included in Attachments 2 and 3, respectively.

4.1 Review of Seismic Analysis Criteria and Licensing Requirements

The seismic analysis criteria have been reviewed for technical adequacy. The criteria have also been compared with the applicable FSAR and SER sections to ensure that the criteria are consistent with the licensing requirements. For the Category I structures, Table 2 shows the comparison between the original seismic analysis criteria, FSAR commitments, and SER provisions.

As demonstrated by Table 2, the seismic analysis criteria used in the original analysis of WBN structures are consistent with the FSAR requirements and SER provisions. In addition, in view of the current industry practice, a study has been initiated to evaluate the effects of floor vertical flexibility on the design of systems and components.

Table 2 is based on Revision 3 of the seismic design criteria, WB-DC-20-24, which has been revised to include the criteria discussed in Section 4.3.5 of this CAP.

The criteria for seismic analysis of the Additional Diesel Generator Building (ADGB), which was included in Amendment 57 of the FSAR (after the SER was issued), will be deleted and the ADGB will be reanalyzed as discussed in Section 4.3.5.

4.2 Review of Seismic Analysis Calculations

An independent review of the seismic analysis of each Category I structure has recently been performed. The review included the following structures.

- Reactor Building Interior Concrete Structure (ICS).
- Reactor Shield Building (SB).
- Steel Containment Vessel (SCV).
- Auxiliary Control Building (ACB).
- Intake Pumping Station (IPS).
- Diesel Generator Building (DGB).
- Additional Diesel Generator Building (ADGB).
- Refueling Water Storage Tanks (RWST).
- North Steam Valve Room (NSVR).
- Pipe Tunnels
- Waste Packaging Area (WPA).
- Condensate Demineralizer Waste Evaporator Building (CDWE).

The WPA and CDWE contain no safety-related systems or components. They were designed as Category I structures to ensure that they will not impact the adjacent ACB during a seismic event.

Several engineering judgments without supporting documentation were identified during the review of the calculations for the ADGB, DGB, and the CDWE. There are also two CAQRs related to the modeling of the supporting piles in the seismic analysis of the ADGB and CDWE (See Attachment 1). In order to resolve questions related to the engineering judgments and the CAQRs, reanalysis of these structures is being performed as discussed in Section 4.3.2.

The calculation review also identified the need to review the torsional modeling of the ICS, ACB, and NSVR. This issue is being addressed as described in Section 4.3.3.

4.3 Disposition of Identified Issues

Issues have been identified through employee concerns, CAQRs, and review of seismic analysis calculations, criteria, and licensing requirements. These issues deal with the following areas:

- Integration time step used to perform time history analysis.
- Soil properties and soil-structure interaction concerns.
- Torsional modeling of structures.
- Seismic analysis criteria for the Additional Diesel Generator Building (ADGB).

The above issues and the approach to resolve them are discussed in the following sections. The effects of these issues on the analysis of Seismic Category I structures are discussed in Section 4.3.5.

4.3.1 Integration Time Step Used in Time History Analysis

An integration time step of 0.01 second was used in the original time history analysis of structures to generate the ARS. An engineering judgment was made that 0.01 second was adequate for structural analysis and the earthquake records were digitized at 0.01 second. An employee concern identified that this integration time step might be too large and could result in an underestimation of the response of those modes which have frequencies greater than 20 Hz.

Seismic Category 1 structures are being reanalyzed, addressing the integration time interval issue, as discussed in the subsection 4.3.5.5. Evaluation of existing structures, systems and components using the new analysis results will also address the adequacy of integration timestep used in the original analysis. New designs or modifications will be based on new analysis results which incorporate an adequate integration time step in the development of ARS.

4.3.2 Soil Properties and Soil-Structure Interaction Concerns

The value of shear modulus for the crushed stone supporting media used in the analysis of the Diesel Generator Building (DGB) and the Waste Packaging Area (WPA) was identified as a concern in a CAQR. The design value originally used was based on the assumption that in situ geophysical measurements made on other similar materials were suitable for the crushed stone. Later, in situ testing of crushed stone and review of technical literature resulted in a lower shear modulus than the one used in the DGB and WPA analysis.

In order to resolve this issue for the DGB, a new soil-structure interaction (SSI) analysis using the revised shear modulus will be performed.

As stated previously, the WPA does not house any safety-related systems and components and the original analysis predicted conservative internal structural forces. In the original analysis, a decoupled, two-stage SSI analysis was used to determine the seismic response of the structure and the results were conservative. An analysis using the revised shear modulus is being performed to confirm that the gap between the WPA and adjacent ACB is adequate. Preliminary results confirm that there is sufficient gap between the two structures such that they will not impact each other during a seismic event.

The Condensate Demineralizer Waste Evaporator Building (CDWE) and the Additional Diesel Generator Building (ADGB) analyses included engineering judgments related to the modeling of the supporting piles. The engineering judgments were questioned by a CAQR and involved stiffness consideration of pile groups and an assumption of full contact between the building's mat foundation and the underlying soil. These judgments were made to maximize the structural responses and may not have predicted conservative reactions for the piles.

There are no safety-related systems and components in the CDWE. Calculations are being performed to more accurately consider the stiffness of the pile groups and the postulated gap between the slab and soil. Preliminary results confirm that the gap between the buildings is sufficient for seismic separation and the design of the structure and piles is adequate even when a gap is assumed to exist between the slab and soil.

The seismic analysis of the ADGB is addressed in Sections 4.3.4 and 4.3.5.

4.3.3 Torsional Modeling

During the review of the calculations discussed in Section 4.2, two torsional modeling issues identified were the mechanics of modeling eccentric masses and the methodology used in calculating torsional constants for open cross sections.

Modeling of Eccentricities

In the original seismic models, the eccentricity between the center of mass and the center of rigidity was included at each mass point. However, the physical location of the center of rigidity was not incorporated into the model.

The Interior Concrete Structure (ICS) and the Auxiliary Control Building (ACB) are the two structures affected by the issue of modeling of eccentricities. Seismic analysis calculation of these two structures has been performed, taking into account actual location of shear centers.

Torsional Constants

The only two structures with significant open sections, where the issue of the effect of warping on the calculation of the torsional constant becomes important, are the ICS and the North Steam Valve Room (NSVR). In both of these cases the original calculations did not include the warping contribution to torsional stiffnesses and thus the resulting calculated torsional constant was lower. This approach was considered to be conservative since calculated torsional responses would be greater. However, the lower torsional constant can cause shifts in the calculated frequencies of the structure and thus, the shape of the ARS can be affected.

Calculations were performed for the ICS considering the modeling of eccentric masses and the revised torsional constants for open sections. An equivalent stick model was developed from a 3-dimensional finite element model to study the effect of the revised torsional constants. The calculations indicated that further evaluations will be required to justify the adequacy of the original calculations (see Section 4.3.5).

A reanalysis of the NSVR considering torsional constants including the warping contribution will also be performed to evaluate the adequacy of the original calculations.

4.3.4 Seismic Analysis Criteria for the Additional Diesel Generator Building (ADGB)

When the ADGB was added to the WBN design, new criteria for seismic analysis of the ADGB were developed. These criteria were based on the current NRC Standard Review Plans (Revision 1) and Regulatory Guides. These criteria were incorporated in the FSAR by Amendment 57, after the NRC had issued the SER and the supplements. The criteria defined in Amendment 57 will be eliminated and the ADGB will be reanalyzed as discussed in Section 4.3.5. This will bring the criteria for ADGB analysis in line with other Category 1 structures at the plant.

4.3.5 Summary of Seismic Analysis Review for Category I Structures

4.3.5.1 Original Analyses

The original analyses of Category I structures were performed consistent with the FSAR requirements and using methodologies that were prevalent at that time. The criteria used and analytical results were reviewed by the NRC prior to issuance of the SER. The seismic analysis results, in the form of structural loads and floor or amplified response spectra (ARS) were used in the design of structures, systems, and components. The Additional Diesel Generator Building (ADGB) was designed at a later date using a different criteria, added to the FSAR in Amendment 57.

The criteria used to perform the original analyses and the significant analysis parameters, called Set A, are shown in Table 3. As can be seen from this table, the original analyses (except for the Additional Diesel Generator Building) utilized four different time-history records. The average of the response spectra of the four time-history records enveloped the Modified Newmark ground response spectrum which was the design basis. The same four records were used in three directions independently. The vertical input was taken as two-thirds of the horizontal. The structural models used in analyses and described in the FSAR were essentially one-dimensional models but included the torsional effects in the direction of excitation.

4.3.5.2 Analyses Using Site Specific Response Spectra

As a result of the issues discussed in Sections 4.3.1 through 4.3.4, it is concluded that reanalysis of some structures is necessary. The intent of the reanalysis is to demonstrate the adequacy of structures, systems, and components considering the effects of the issues identified through the calculation review, employee concern, and CAQR programs. In order to determine the significance of these issues, i.e., whether the existing hardware meets design requirements or whether modifications would be required, the evaluations will be based on criteria compatible with current practices. This will include the Site Specific Response Spectra (SSRS) developed for WBN evaluations which were reviewed and concurred by the NRC in the SER. It will also include soil-structure interaction analysis methods (strain-dependent soil properties and damping) that are consistent with the Standard Review Plan.

The criteria for SSRS analysis and the significant parameters related to the criteria, called Set B, are shown in Table 4.

4.3.5.3 Reanalysis Using the Original Criteria and Current Modeling Techniques

Category I structures will be reanalyzed using the original criteria with modeling improvements, consistent with the current techniques, to develop a new set of response spectra, called Set C. The new analyses will also include soil-structure interaction analysis methods (strain-dependent soil properties and damping) that are consistent with the Standard Review Plan.

The criteria for this reanalysis and the significant analysis parameters are shown in Table 5. Comparison of Tables 3 and 5 indicates that the two sets of criteria are identical except in the area of modeling (including modal and spatial combinations, and SSI methodology) where current practices differ from those used during the original analysis.

4.3.5.4 Use of Results from Various Analyses

In order to assure uniformity of application and correct interpretation of the results generated from various analyses, the following guidelines are established:

Set A Results: Results of Set A analyses (e.g., ARS, forces, and displacements) are the design data of record. Calculations supporting existing structures, systems, and components are based on Set A results. Any new calculations for new design or modifications shall be generated as described later in this section.

ARS from the original analyses are termed <u>Original Spectra</u>.

Set B Results: Results of Set B analyses (e.g., ARS, forces, and displacements) will be used to evaluate adequacy of structures, systems, and components. The evaluations will be performed in accordance with specific CAP requirements such as HAAUP, HVAC, Cable Tray, Conduit, Equipment Seismic, and Instrument Lines. These evaluations can be performed at the spectra level or component level.

ARS from Set B results are termed <u>Evaluation Spectra</u>.

Envelope of Set B and C Results:

Results obtained by enveloping the Set B and Set C (e.g., ARS, forces, and displacements) will be used for new designs or modifications. Any class of components or individual items that are not designed using the original seismic design basis i.e., Set A ARS, will require new calculations based on the envelope of Set B and C results.

The envelope of Set B and C ARS are termed New Design or Modification Spectra.

4.3.5.5 Criteria for Evaluation and New Design or Modification of Structures, Systems, and Components

The various structural analyses discussed above and their use are summarized in Table 6 for each structure housing Category I systems and components. The criteria used for original analysis of the Additional Diesel Generator Building will be eliminated. New analysis will be performed to generate Set B and Set C results.

The Young's and shear moduli of the concrete have been reevaluated for use in the reanalyses. The evaluation concluded that lower moduli values should be used for Interior Concrete Structure, Additional Diesel Generator Building, and North Steam Valve Room. The revised moduli will be incorporated into both Set B and Set C analysis.

As shown in Table 6 and as discussed in Section 4.3.5.4, evaluation of structures and the systems and components contained in these structures will be based on Set B. For rigorously analyzed piping, the envelope of Set B and C response spectra will be used in the HAAUP program. The scope of evaluations for systems and components are discussed in the other CAPs (Cable Tray, Conduit, HVAC, Instrument Lines, HAAUP, and Equipment Seismic Qualification) in detail. Any new design or modification of structures, systems, and components will be based on the envelope of Set B and C.

The criteria and methodology to be used in the evaluations and new designs or modifications of systems and components are shown in Table 7. As shown in the table, damping values based on Regulatory Guide 1.61, Code Case N411, and applicable test data will be used. Damping values for each commodity are shown in Table 8. Use of higher damping is justified since the evaluation criteria (Set B) are consistent with the SRP provisions. The new design or modification criteria (envelope of Set B and C) is more conservative than the evaluation criteria and, therefore, use of higher damping values is appropriate for this case also.

The analysis techniques to be used for system and component analysis in the new work are also consistent with the SRP provisions. Equivalent static, response spectrum, and time-history analysis methods will be used. The time-history analysis may be used if the system input time-history records are demonstrated to contain sufficient energy over the entire frequency range by an analysis of its power spectral density. Uncertainties in time-history analysis will be addressed through the use of peak shifting technique.

In the design of structures, systems, and components the 2D absolute sum method will continue to be used for structures and commodities except piping. This is consistent with the FSAR requirements. Studies show that the difference resulting from the use of 2D absolute sum and 3D square-root-of-sum-of-squares is small and, therefore, use of the 2D absolute sum method for maintaining the licensing basis and continuity is acceptable. For piping analysis, the 3D SRSS approach will be used, as indicated in Table 7 in order to be able to use N411 damping values.

In summary, the seismic criteria for systems and components as shown in Tables 7 and 8, when used in conjunction with ARS from the new analyses, will provide assurance that WBN plant will have been designed to meet licensing requirements and to be consistent with the current SRP provisions.

4.4 Recurrence Control

The root cause identified in this CAP has been addressed through procedural improvement.

A procedure is now in place (NEP 3.1) to ensure that engineering judgments used in the design process will be adequately documented.

4.5 Licensing Assessment

In order to resolve the issues identified in this CAP and to establish the seismic design basis for future work, revisions to the design criteria and FSAR may be necessary. Any changes to the licensing commitments will be proposed only when technically justified.

5.0 PROGRAM INTERFACES

The ARS are used in the design of safety-related systems and components. Therefore, the output of this CAP will provide input to other CAPs such as HAAUP, Cable Trays, Conduit, Instrument Lines, HVAC, and Equipment Seismic Qualification.

6.0 PROGRAM IMPLEMENTATION

Nuclear Engineering (NE) is the lead organization for implementing and completing the Seismic Analysis CAP. Calculations will be performed in accordance with standard TVA procedures and practices.

7.0 PROGRAM DOCUMENTATION

Results of this CAP will be documented in design criteria, calculations and reports. The FSAR revisions resulting from this CAP will be submitted to the NRC. Affected documents will be revised in accordance with NE procedures. A final report will be prepared documenting the results of evaluations performed to resolve identified issues.

8.0 CONCLUSION

The completion of the Seismic Analysis CAP will confirm that the seismic analysis of structures and the ARS generated from the analyses are technically adequate and satisfy licensing requirements. In addition, related employee concern and CAQRs dealing with seismic analysis issues will be resolved.

9.0 REFERENCE

1. Dynamic Earthquake Analysis of Category I Structures and Earth Embankments, WB-DC-20-24, Revision 3, July 1988

TABLE 1

Issues Identified from Review of Seismic Analysis Calculations of Category I Structures, Employee Concerns, and CAQRs.

Issue	Disposition
1. Integration time step used to perform time history analysis. (ECP-87-KX-009-01)	New analyses will be performed using a time step of 0.005 seconds. (See Section 4.3.1)
2. Dynamic soil properties and soil-structure interaction concerns (CAQR WBF 870038R1, CAQR WBF 870039R1, and CAQR WBP 870396R0)	
° Waste Packaging Area and Condensate Demineralizer Waste Evaporator Building	° Calculations are being completed and preliminary evaluation shows the adequacy of existing analyses.
° Diesel Generator Building and Additional Diesel Generator Building	° Further analyses will be performed to evaluate structures and components.
3. Torsional modeling	
° Reactor Building Interior Concrete Structure.	° New analyses are being performed to develop ARS which will be used to evaluate structures and components.
° North Steam Valve Room.	° New analyses will be performed to evaluate structures and components.
° Auxiliary Control Building.	° New analysis will be performed to evaluate structures and components.

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COMPARISON OF SEISMIC ANALYSIS CRITERIA, FSAR, AND SER
FOR SEISMIC CATEGORY I STRUCTURES

ATTRIBUTE	DESIGN CRITERIA (DC)(1,2) WB-DC-20-24, Rev. 3	FSAR	SER	DIFFERENCES BETWEEN DC AND FSAR, SER
DYNAMIC ANALYSIS PARAMETERS				
Design Response Spectra (input ground motion spectra)	° Modified Newmark	° Same (3) (2.5.2.6) (2.5.2.7) (3.7.1.1.1)	° Same (4) (2.5.2.1) (3.7.1)	° None - Design basis has been accepted by NRC based on site specific spectra evaluation
Max. top-of-rock accelerations SSE: OBE:	° 0.18 gH, 0.12gV ° 0.09gH, 0.06gV (3.1)	° Same (5) (2.5) (3.7.1.1.1)	° Same (2.5) (3.7.1)	° None
Ratio of vertical to horizontal response spectrum	° 2/3 to 1 (3.1) (3.2.1.2)	° Same (3.7.1.1.1) (3.7.2.4.1)	° No explicit statement. Concurrence with FSAR is implicit per section 3.7.1.	° None
Design time histories (input ground motion T-H)	° 4 artificial E/Q's ° The same 4 T-Hs are used in all 3 directions	° Same (3.7.1.2.1)	° No explicit statement. Concurrence with FSAR is implicit per section 3.7.2.	° None

Notes:

01) This Column indicates design criteria provisions. If no explicit statement is included in DC, the column indicates the actual methodology adopted in analysis.
(2) Basis of comparisons is revision 3 of WB-DC-20-24. Design criteria will be revised to include the criteria for evaluation and new design or modification of structures, systems, and components discussed in Section 4.3.5 of this CAP.
(3) FSAR provision same as DC. (Typical for FSAR column.)
(4) SER agrees with FSAR. (Typical for SER column.)
(5) Due to a typo, the FSAR in Section 3.7.1.1.1 states that the OBE horizontal acceleration is 0.08g, not 0.09g and that the OBE vertical acceleration is 0.6g, not 0.06g.

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COMPARISON OF SEISMIC ANALYSIS CRITERIA, FSAR, AND SER
FOR SEISMIC CATEGORY I STRUCTURES

ATTRIBUTE	DESIGN CRITERIA (DC) WB-DC-20-24, Rev. 3	FSAR	SER	DIFFERENCES BETWEEN DC AND FSAR, SER
Frequency (period) interval for generating ground motion input spectra	° Calculated at 55 periods (Table 6)	° Table does not cover period range .03 to .05 sec. (Table 3.7-1)	° No explicit statement. Concurrence with FSAR is implicit per Section 3.7.2	° Minor differences - FSAR will be updated
Damping values	° FSAR Table 3.7-2A	° Same (Table 3.7-2A)	° Same (3.7.1)	° None
Supporting media	° Shear wave velocity and embedment and overburden depths are defined	° Same (Table 3.7-3) (3.7.1.4)	° No explicit statement. Concurrence with FSAR is implicit per Section 3.7.2.	° None
DYNAMIC ANALYSIS METHODS				
Analysis method	° Time-history modal analysis using four artificial earthquake records for generation of ARS and Response Spectrum Analysis (RSA) for structural loads (3.2.1)	° Same (3.7.2.1)	° Same (3.7.2)	° None
	° Integration time step 0.01 sec	° Same (3.7.2.5.1)	° No explicit statement. Concurrence with FSAR is implicit per section 3.7.2.	° None - However adequacy of time step has been addressed.

TABLE

COMPARISON OF SEISMIC ANALYSIS CRITERIA, FSAR, AND SER
FOR SEISMIC CATEGORY I STRUCTURES

ATTRIBUTE	DESIGN CRITERIA (DC) WB-DC-20-24, Rev. 3	FSAR	SER	DIFFERENCES BETWEEN DC AND FSAR, SER
Soil-structure interaction	° Rock-Supported Fixed base (allows linear springs which indicate fixed base)	° Same (3.7.2.1)	° No explicit statement. Concurrence with FSAR is implicit per section 3.7.2.	° None
	° Soil-Supported Rock motion amplified through soil by linear shear beam w/10% soil damping. Soil modulus was varied. Structures modeled with linear soil springs with 10% damping.	° Same (3.7.2.1)	° No explicit statement. Concurrence with FSAR is implicit per section 3.7.2.	° None - However, dynamic soil properties and SSI for some structures are under review.
	° Half-space analysis except for ADGB and RWST which used FLUSH.	° Same (3.7.2.1)	° No explicit statement. Concurrence with FSAR is implicit per section 3.7.2. ADGB analysis is not addressed in SER.	° None - However, dynamic soil properties and SSI for some structures are under review.
Torsional, rocking, and translational responses	° Rocking and translation considered. Torsional response taken into account where significant.	° Same (3.7.2.1)	° Torsional responses were considered (3.7.2). Concurrence with treatment of rocking and translation is implicit per section 3.7.2.	° None

TABLE

COMPARISON OF SEISMIC ANALYSIS CRITERIA, FSAR, AND SER
FOR SEISMIC CATEGORY I STRUCTURES

ATTRIBUTE	DESIGN CRITERIA (DC) WB-DC-20-24, Rev. 3	FSAR	SER	DIFFERENCES BETWEEN DC AND FSAR, SER
Methods to account for torsional effects	° Lumped-mass models considered eccentricities between center of rigidity and center of mass ° Responses calculated at extreme points.	° Same (3.7.2.11)	° Torsional effects were considered. (3.7.2)	° None
Adequate number of masses	° Based on judgment. Mass points were located at floor slabs, change to geometry, and at intermediate points.	° Same (3.7.2.1)	° No explicit statement. Concurrence with FSAR is implicit per section 3.7.2.	° None
Adequate number of modes to assure participation of significant modes	° Response to be calculated using all significant modes (3.2.1)	° Same (3.7.2.1) Modes considered are shown in tables.	° No explicit statement. Concurrence with FSAR is implicit per Section 3.7.2.	° None
Maximum relative displacements between structures	° Maximum relative displacements were calculated by sum of the absolute values	° Same (3.7.3.8.4)	° No explicit statement. Concurrence with treatment of support motions is implicit in Section 3.7.3.	° None
Acceleration time history or response spectra at floors	° Response spectra generated at ground level, at all major floors, and at other points where input is needed for further analysis (3.2.2).	° Same (3.7.2.5.1)	° No explicit statement. Concurrence with FSAR is implicit in Section 3.7.2.	° None

2

COMPARISON OF SEISMIC ANALYSIS CRITERIA, FSAR, AND SER
FOR SEISMIC CATEGORY I STRUCTURES

ATTRIBUTE	DESIGN CRITERIA (DC) WB-DC-20-24, Rev. 3	FSAR	SER	DIFFERENCES BETWEEN DC AND FSAR, SER
ANALYTICAL MODELING				
Decoupling criteria for subsystems	° No explicit statement for decoupling. Subsystems were considered rigid in analysis.	° Same (3.7.2.3).	° No explicit statement. Concurrence with FSAR is implicit per Section 3.7.2.	° None
Modeling for three components of input motion	° Three components of input motion were considered. However, no coupling of horizontal and vertical analyses (3.2.1).	° Same (3.7.2.1)	° No explicit statement. Concurrence with FSAR is implicit per section 3.7.2.	° None
DEVELOPMENT OF FLOOR RESPONSE SPECTRA	° Fixed and variable (N411) Damping (2) ° Spectra broadened by ± 10% ° Optional use of ASME Code Case N-397 for peak shifting (2) ° Torsion calc at extreme edges ° Spectra was computed for 55 periods given in Table 6 and at significant periods of the structure and at structural periods shifted by fine interval. ° Vertical spectra generated using wall stiffnesses and vertical input motion only.(1) (3.2.2)	° Same (3.7.2.9) ° Table does not cover period range .03 to .05 sec (Table 3.7-1)	° Development of floor response spectra was reviewed (3.7.2).	° Minor differences in period range. FSAR will be updated.

Note:

(1) A study is being performed to evaluate the effects of floor vertical flexibility on systems and components.
(2) Added after SER.

TABLE
COMPARISON OF SEISMIC ANALYSIS CRITERIA, FSAR, AND SER
FOR SEISMIC CATEGORY I STRUCTURES

ATTRIBUTE	DESIGN CRITERIA (DC) WB-DC-20-24, Rev. 3	FSAR	SER	DIFFERENCES BETWEEN DC AND FSAR, SER
THREE COMPONENTS OF EARTHQUAKE MOTION	° Critical horizontal responses combined with vertical by ABSUM.	° Same (3.7.2.10.1.1)	° Same (3.7.3)	° None
COMBINATION OF MODAL RESPONSES	° Modes combined by SRSS. Closely spaced modes by the grouping method in RG 1.92.	° Same (3.7.2.7.1.1)	° Same (3.7.2)	° None
INTERACTION OF NON-CAT I WITH CAT I STRUCTURES	° Need to consider interaction of non-Cat I with Cat I structures.	° Same (3.7.2.8)	° Same (3.7.2)	° None
USE OF EQUIVALENT STATIC FACTORS	° Vertical ARS were developed considering structural amplification	° Same (3.7.2.10.1.1)	° Same (3.7.2)	° None
COMPARISON OF RESPONSES	° Results for response spectra and time history analysis of ICS provided in FSAR Figure 3.7-38.	° Same (3.7.2.12) (Figure 3.7-38)	° No explicit statement. Concurrence with FSAR is implicit per Section 3.7.2.	° None
COMPOSITE MODAL DAMPING	° For rock-supported structures, no need to consider composite modal damping. ° For soil-supported structures, modal damping was limited to 10%.	° Same (3.7.2.15)	° No explicit statement. Concurrence with FSAR is implicit per Section 3.7.2.	° None
STRUCTURE OVERTURNING MOMENTS	° Structure overturning moments were calculated for critical horizontal response combined with vertical.	° Same (3.7.2.14.1)	° Same Stability against overturning was considered. (3.7.2)	° None

Table 3. Original Seismic Analysis Criteria for Category I structures - Set A

Attributes	Criteria
Design Spectra	Modified Newmark
Peak Ground Accel.	
SSE	0.18 G Hor. 0.12 G Vert.
OBE	0.09 G Hor. 0.06 G Vert.
Artificial Time - History Records	Four artificial T-H records - Use average of four responses. Same four used in each direction independently. Average of T-H spectra envelop modified Newmark.
Structural Models	As described in the FSAR
Integration Time Step	0.01 second
Modal Combination	RG 1.92 (No coupling between directions per FSAR models)
Spatial Combination for ARS Generation	As described in FSAR (No coupling between directions)
SSI Methodology	As described in FSAR (see Table 2)
Peak Broadening	± 10%

Damping	OBE	SSE
Steel Containment Vessel (SCV)	1	1
Shield Build. and Interior Conc. Struct.	2	5
Other Concrete Structures	5	5
Other Welded Steel Structures	2	2

Table 4. Site Specific Response Spectra (SSRS) Analysis Criteria for Category I Structures - Set B

Attributes	Criteria
Design Spectra	SSRS
Peak Ground Accel.	
SSE	0.215 G Hor. 0.15 G Vert.
OBE	0.09 G Hor. 0.06 G Vert.
Artificial Time - History Records (1)	Three statistically independent records - one for each direction. T-H spectra envelop SSRS
Structural Models	3D - Coupling effects included
Integration Time Step (2)	0.005 second
Modal Combination	RG 1.92
Spatial Combination for ARS Generation	3D SRSS or simultaneous input
SSI Methodology	Elastic Half-Space or Finite Element Approach with strain-dependent soil properties and damping
Peak Broadening	± 15%

Damping	OBE	SSE
Welded Steel Structures	2	4
Concrete Structures (3)	4	7

(1) In performing T-H analysis with single set of T-Hs, adequacy of energy content shall be demonstrated.

(2) If frequency domain analysis method is used, input time-history interval of 0.01 second is adequate.

(3) Includes Interior Concrete Structure, Shield, Auxiliary Control, Diesel and Additional Diesel Generator buildings, North Steam VAlve Room, and Intake Pumping Station.

Table 5. Seismic Reanalysis Using Original Criteria and Current Modeling Techniques for Category I structures - Set C

Attributes	Criteria
Design Spectra	Modified Newmark
Peak Ground Accel.	
SSE	0.18 G Hor. 0.12 G Vert.
OBE	0.09 G Hor. 0.06 G Vert.
Artificial Time - History Records	Four artificial T-H records - Use average of four responses. Same four used in each direction independently. Average of T-H spectra envelop modified Newmark.
Structural Models	3D - Coupling effects included
Integration Time Step (1)	0.005 second
Modal Combination	RG 1.92
Spatial Combination for ARS Generation	3D SRSS
SSI Methodology	Elastic Half-space or Finite Element Approach with strain-dependent Soil properties and damping.
Peak Broadening	± 10%

Damping	OBE	SSE
Steel Containment Vessel (SCV)	1	1
Shield Build. and Interior Conc. Struct.	2	5
Other Concrete Structures	5	5
Other Welded Steel Structures	2	2

(1) If frequency domain analysis method is used, input time-history interval of 0.01 second is adequate.

Table 6. Seismic Analysis Matrix

Structure	Set A(1)	Set B(2)	Set C(3)	Eval	New Design or Modif (4)
Interior Concrete Structure	E	Y	Y	B	B+C
Steel Containment Vessel	E	Y	Y	B	B+C
Shield Building	E	Y	Y	B	B+C
Diesel Generator Building	E	Y	Y	B	B+C
Additional Diesel Generator Building	*	Y	Y	B+C	B+C
North Steam Valve Room	E	Y	Y	B	B+C
Auxiliary/Control Building	E	Y	Y	B	B+C
Refueling Water Storage Tank	E	Y	Y	B	B+C
Intake Pumping Station	E	Y	Y	B	B+C
Pipe Tunnels	**	Y	Y	B+C	B+C

E = existing analysis
* = original analysis criteria were established subsequent to SER. These criteria will be eliminated.
Y = yes, analysis is needed
** = Original spectra were not explicitly developed.

Notes:

1. Set A refers to original analysis.
2. Set B refers to SRP - compatible analysis using SSRS
3. Set C refers to reanalysis using original criteria and current modeling
4. B + C indicates envelop of B and C results.

Table 7. Seismic Criteria and Methodology for Systems and Components for Evaluation and New Design or Modification

Attributes	Criteria and Methodology
Damping for Sets B, C, and the envelop of B and C (See Table 8 for values)	Use damping values based on ° RG 1.61 ° N411 ° Test Results
Analysis techniques	Use SRP - Compatible approaches ° Equivalent Static ° Response Spectrum Analysis (RSA) ° T-H Analysis (THA) (1)
Accounting for Uncertainties	Peak broadening (RSA) Peak shifting (THA)
Spatial Combinations	2D Absolute sum except for piping 3D Square-root-of-sum-of-squares for piping

(1) In performing T-H analysis with single set of T-Hs, adequacy of energy content shall be demonstrated.

Table 8. Seismic Criteria for System and Component Damping for Evaluation and New Design or Modification

Item	Proposed For Evaluation and Modification or New Design		Justification Source For Proposed Values
	OBE	SSE	
Piping			
12" or Larger	2	3	RG 1.61
Less than 12"	1	2	RG 1.61
Optional (code Case)	N411	N411	RG 1.84
Cable Tray System	7	7	Test Results (1)
Conduit System	7	7	Test Results (1)
HVAC Systems			Nuclear Air Cleaning Handbook and Test Results (1)
Companion Angle	5	7	
Pocket Lock	7	7	
Welded Duct	2	5	
Equipment	2	3	RG 1.61

(1) Higher Damping may be used in specific applications if supported by test data and endorsed by NRC.

ATTACHMENT 1

LIST OF EMPLOYEE CONCERN AND CAQRs

Item	Document	Description
1	ECP-87-KX-009-01 (L77 870608 804)	Concern with integration time step used to perform the time-history analysis. The time step used may be too large to calculate high frequency response adequately.
2	CAQR WBF 870038R1 (B05 870706 300)	Concern with soil structure interaction (SSI) analysis for the design of the pile foundation for Condensate Demineralizer Waste Evaporator Building. The analysis may not reflect the maximum loading condition for the piles and the soil spring constants used in analysis may not be realistic.
3	CAQR WBF 870039R1 (B05 870729 306)	Concern with SSI analyses for the design of the pile foundation for the Additional Diesel Generator Building. The concern is similar to that of Condensate Demineralizer Waste Evaporator Building analysis.
4	CAQR WBP 870396R0 (T42 870528 975)	Concern regarding the soil modulus for crushed stone for Diesel Generator Building and Waste Packaging Area.

ENCLOSURE 2

LIST OF COMMITMENTS

For the Watts Bar Nuclear Plant unit 1, TVA commits to:

1. Review of seismic analysis calculations for category I structures and revisions as required or preparation of new calculations when necessary.

2. In order to resolve the issues identified in this CAP and to establish the seismic design basis for future work, revisions to the design criteria and Final Safety Analysis Report may be necessary. Any changes to the licensing commitments will be proposed only when technically justified.

3. Disposition issues identified through employee concerns, condition adverse to quality reports, and review of seismic analysis calculations, criteria, and licensing requirements.

4. Submit to NRC the references justifying the use of damping values for conduit, cable tray, and heating, ventilating, and air conditioning (i.e., ANCO testing) by July 31, 1989.

ENCLOSURE 3

RESPONSES TO ACTION ITEMS RESULTING
FROM NUCLEAR REGULATORY COMMISSION (NRC)/TVA MEETING
OF FEBRUARY 7 AND 8, 1989
ON SEISMIC ANALYSIS CORRECTIVE ACTION PROGRAM (CAP) PLAN

1. TVA is to submit a revision to the seismic analysis CAP to reflect the "Set B" and "Set C" criteria and how TVA will use each criteria set.

   Response: The revised CAP which reflects the "Set B" and "Set C" criteria and how TVA will use each criteria set is provided as enclosure 1 to this submittal. The revision also incorporates comments resulting from an informal NRC/TVA meeting held on March 22, 1989, at Knoxville, TN.

2. TVA will provide a copy of the seismic design criteria when revised to reflect "Set B" and "Set C" criteria.

   Response: The revised Seismic Design Criteria WB-DC-20-24, revision 4, reflects the "Set B" and "Set C" criteria incorporated in this CAP plan revision and is available for NRC review.

3. TVA and NRC are to assess the use of envelope of the amplified response spectra (ARS) obtained from "Set B" and "Set C" analysis for use with American Society of Mechanical Engineers (ASME) Code Case N-411.

   Response: A conference call was held on March 9, 1989, between TVA and NRC, and it was agreed that the envelope of "Set B" and "Set C" response spectra may be used for piping analysis with ASME Code Case N-411 damping. It was also agreed that any modifications or new design of structures or other commodities can also use an envelope of "Set B" and 'Set C" response spectra. This position is incorporated in the revised seismic analysis CAP and design criteria.

4. Table 2 of the seismic CAP needs to be clarified in a number of areas to explicitly convey TVA's intent (e.g., explain what is meant by "same").

   Response: The revised CAP (enclosure 1) incorporates the appropriate revisions in Table 2 to clarify areas where "same" is used and the intent of the table.

5. TVA and NRC are to check on the previous NRC acceptance of use of SASSI computer code.

   Response: TVA has determined that the SASSI computer program to be used for WBN plant structures is the same version which was applied to the seismic soil-structure interaction for the Diablo Canyon Power Plant Long Term Seismic Program. The results obtained from this version of the SASSI program were audited by the NRC staff and its consultants and, subsequently, documented July 1987 in a trip report prepared by NRC's consultant, Dr. C. J. Constantino. Also, an NRC audit to assess the use of SASSI on WBN was held at Bechtel's San Francisco office on April 26, 1989.

ENCLOSURE 3

6. TVA will provide a comparison of the damping values for equipment (Table 8) against the Sequoyah values (use of Regulatory Guide [RG] 1.61 as basis for equipment damping values).

   Response: Sequoyah damping values for equipment are two percent for operating base earthquake (OBE) and three percent for safe shutdown earthquake (SSE). The same damping values shall be used for WBN as shown in Table 8 of the seismic analysis CAP. These damping values are consistent with RG 1.61 equipment damping values.

7. Provide a copy of the references justifying the use of damping values for conduit, cable tray, and heating, ventilating, and air conditioning (i.e., ANCO testing).

   Response: The references currently are being assembled and will be submitted on or about July 31, 1989.

8. Provide copies of the critical case walkthrough procedure when issued and keep NRC apprised of the progress.

   Response: The conduit walkthrough procedure is issued and available for NRC review. Walkthroughs were started in late April 1989, and the resident inspectors are kept apprised of progress.

9. TVA will submit a revision to the Hanger and Analysis Update Program (HAAUP) CAP to reflect the revised support evaluations for small bore ASME piping.

   Response: The HAAUP CAP has been revised and is being provided as a separate submittal.

ENCLOSURE 4

APPENDIX O

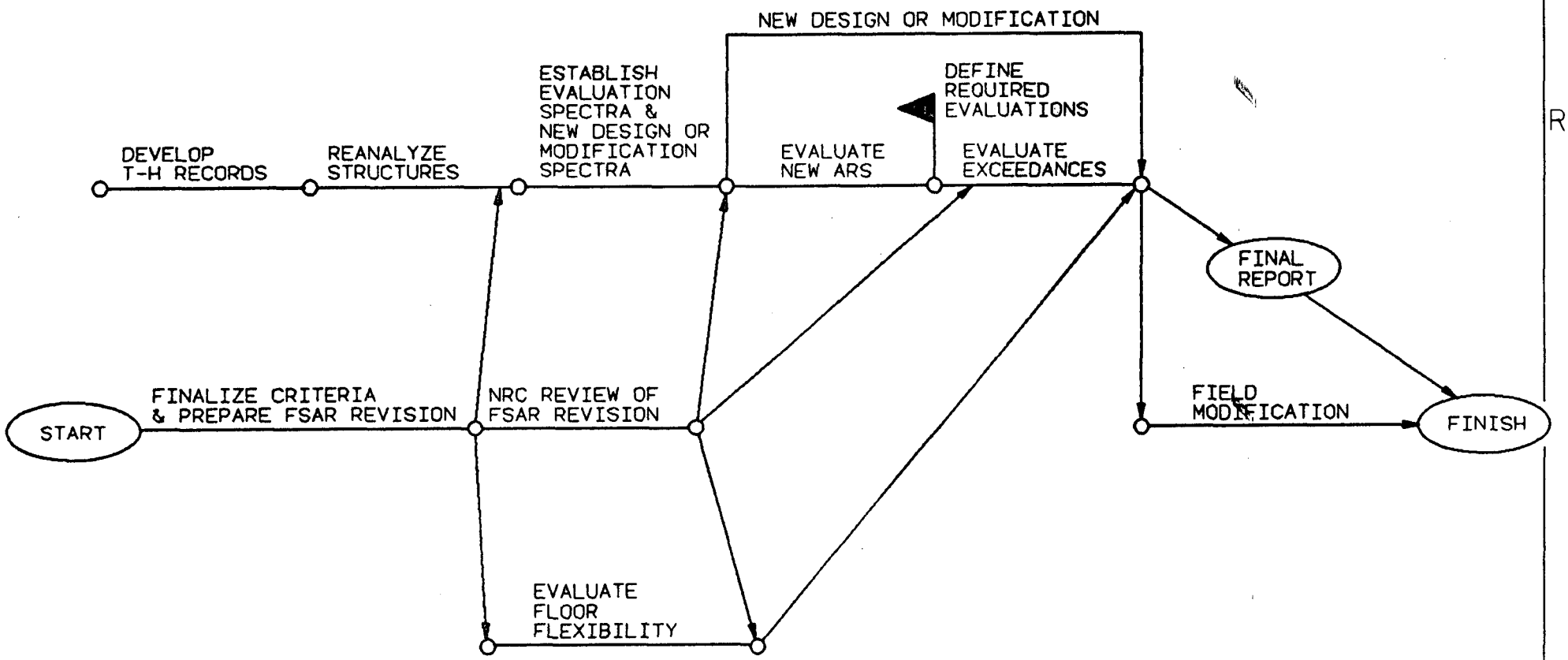
DYNAMIC SOIL AND BACKFILL PARAMETERS

SEISMIC ASSESSMENT REPORT

FOR

WATTS BAR NUCLEAR PLANT

WATTS BAR NUCLEAR PLANT
SEISMIC ANALYSIS CAP FRAGNET



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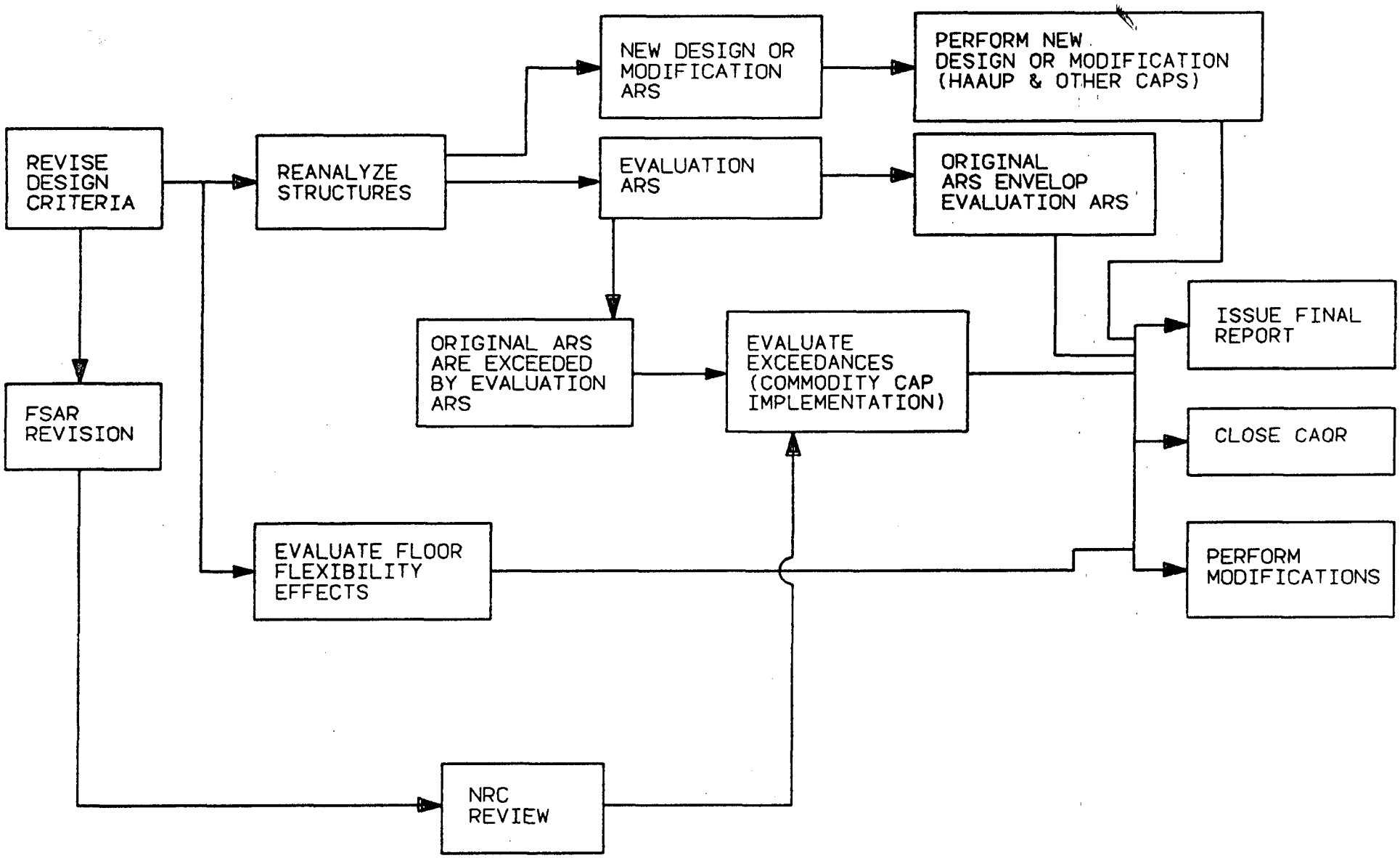
ATTACHMENT 3

WBN COMPUTER GRAPHICS (CNC)

WATTS BAR NUCLEAR PLANT
SEISMIC ANALYSIS CAP FLOWCHART



ATTACHMENT 2

WBN SEISMIC ANALYSIS CAP FLOWCHART

WBN COMPUTER GRAPHICS UNIT (CNC)

APPENDIX O

DYNAMIC SOIL AND BACKFILL PARAMETERS

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APPENDIX O

DYNAMIC SOIL AND BACKFILL PARAMETERS

01.0 INTRODUCTION

Major structures and features at WBN are supported on a variety of foundation media. Relatively large structures bear directly on the bedrock. Other structures are either founded on fill material over basal gravel or supported by piles driven through in-situ soils to rock.

For soil supported structures, some concerns (CAQRs) have been raised regarding the appropriateness of soil properties used in the soil structure interaction and other analyses.

Soil properties selected by TVA and used in the original analyses of WBN structures have been previously reviewed by other TVA consultants such as SWEC and Sargent and Lundy. As a part of SAP, Bechtel has evaluated all previous work performed to date and using objective evidence such as contained in the borehole logs, laboratory test results, geophysical surveys, and the research performed by others, evaluated the soil properties that would be appropriate for the seismic analysis of WBN structures. This Appendix documents the Bechtel effort and provides recommended dynamic soil properties for TVA's consideration and use.

02.0 BACKGROUND

This section briefly describes the WBN site geology, and provides a listing of the various soil supported structures whose seismic responses may be affected by use of modified dynamic soil properties.

02.1 WBN Site Geology

The site is characterized by unconsolidated alluvial deposits overlying weathered to fresh shale bedrock.

The alluvial deposits have an average thickness of about 25 feet, and consist of clay, silty clay, sandy clay, clayey sand, silt, silty sand, and gravel. Soils are generally cohesive in the upper zone with clay fractions predominating, grading to less cohesive or non-plastic soils at depth where silt and sand fractions predominate. The lowest portion consists of basal gravel of varying thickness, but generally not exceeding five feet.

The alluvial deposits are underlain by weathered to fresh shale bedrock with thin limestone interbeds. Reported depth of weathering is variable, ranging from a few feet to more than 20 feet in the main plant area.

02.2 WBN Soil Supported Structures

The following is a listing of the main WBN structures that are soil supported or influenced:

- o Diesel Generator Building
- o Additional Diesel Generator Building ("H" piles to rock)
- o Waste Packaging Area
- o Condensate Demineralizer and Waste Evaporator Building ("H" piles to rock)
- o North Steam Valve Room (concrete walls to rock)
- o Refueling Water Storage Tank
- o Essential Raw Cooling Water Pipe Tunnel
- o Essential Raw Cooling Water Pipe Support Slab ("H" piles to rock) and Sheet Pile Retaining Wall

Site geotechnical materials underneath these structures are described in Table 0-1.

03.0 DYNAMIC SOIL PROPERTIES

03.1 Original Evaluation

Dynamic soil properties used by TVA for the original evaluations are listed in Table 0-2 (Ref. 1, Table 3.7-3 and TVA calculations). As reflected in Table 0-2, the original properties were revised in August 1987 to account for incorrect dynamic soil properties used for in-situ materials and for crushed stone backfill, and to more precisely define separate zones of in-situ materials. This revision applied to the DGB only.

03.2 Consultants' Review of Dynamic Soil Properties

Selected structures were evaluated in 1988 by Sargent & Lundy (Ref. 3) and SWEC (Refs. 4 & 5). Dynamic soil properties used in these reviews are shown in Table 0-3.

04.0 FIELD AND LABORATORY DATA

For the present review, dynamic soil properties were evaluated on the basis of field and laboratory data available at TVA headquarters in Knoxville, as described below:

Geophysical surveys were available for the following structures/features:

- o Low Level Radwaste Borrow Area (Reference 6)
- o Diesel Generator Building (Reference 1, Figure 2.5-233)
- o Intake Channel (Reference 1, Figures 2.5-228 to 232)
- o Class 1E Conduits and Essential Raw Cooling Water Piping (Reference 1, Table 2.5-17)
- o Crushed Stone Backfill (Reference 13)

Other field data included:

- o Borehole logs with standard penetration testing (Reference 1).
- o Placement records for crushed stone backfill (Reference 1, Table 2.5-56 and 2.5-57).

Laboratory test data utilized in this evaluation included:

- o Index properties and static strength properties for undisturbed and remolded samples (Reference 1).
- o Engineering properties of crushed stone backfill (Reference 1).
- o Engineering properties of Class A backfill (Reference 1).
- o Resonant column testing for undistrubed and remolded samples for the Low Level Radwaste Borrow Area (Reference 7).

05.0 BACKFILL MATERIALS - BECHTEL REVIEW

Two types of backfill were used at the site, fine-grained soil (Class A Backfill) and crushed stone (1032 Engineered Fill). Due to concern about excessive settlement, in situ materials were excavated down to the top of basal gravel for selected structures, or in the case of WPA to bedrock, and backfilled with crushed stone. Class A Backfill was used beneath selected auxiliary (Non-Category I) structures, and adjacent to selected Category I structures to bring the surface to final grade.

05.1 Class A Backfill

Class A Backfill was used beneath the CDWE Building and the ERCW pipe support slab, and was also placed adjacent to other structures to bring the surface to finish grade. This material consists of sandy or silty lean clay placed in six inch layers

and compacted to a minimum of 95 percent of maximum dry density as determined by ASTM D-698. Representative properties for this material are optimum moisture content of 17.9 percent, maximum dry density of 108 pcf, and undrained, unconsolidated shear strength of 2.1 ksf (Reference 1, Figures 2.5-235 and 2.5-244 and Attachment O-1, Section O-1.2).

Resonant column testing results are available for remolded samples of CL material obtained from the Low Level Radwaste Borrow Area for shear strain ranges of 10^{-3} to 10^{-2} percent (References 6 and 7). To obtain G max, results obtained at a shear strain amplitude of 10^{-3} percent were extrapolated for a shear strain amplitude of 10^{-4} percent using typical curves for reduction of shear modulus with strain for clays. Details are presented in Section O-6.0 of Attachment O-1. Results are shown in Figure O-1 as maximum shear modulus versus molding moisture content.

In addition, results of standard penetration testing from boreholes encountering lean clay backfill were used to provide an estimate of shear modulus using the method proposed by Egan and Ebeling (Reference 9). N values from standard penetration testing were used to provide an estimate of unconfined compressive strength and corresponding undrained shear strength. From undrained shear strength, estimated values of G_{max} were determined using the relationships proposed by Egan and Ebeling. Details are presented in Section O-7.0 of Attachment O-1. Results are shown in Figure O-2, as maximum shear modulus versus in situ moisture content.

It is noted that the resonant column testing was done using samples compacted to 95% of maximum dry density, and the clay backfill may have been compacted to only 90% of maximum dry density. The effect of this difference is not considered significant on dynamic soil properties.

For lean clay backfill at optimum moisture content of 18 percent, the available data suggests an approximate range of G_{max} of 3000 to 6000 ksf. A design value of G_{max} = 4500 ksf is recommended.

Figure O-3 shows the recommended curve of shear modulus reduction with shear strain, based on an undrained shear strength of 2.1 ksf. Details regarding selection of this curve are presented in Sections O-1.1 and O-1.2 of Attachment O-1.

Figure O-4 shows the recommended damping ratio curve. Details regarding selection of this curve are presented in Sections O-4.1 and O-4.2 of Attachment O-1.

05.2 Crushed Stone Backfill

Crushed stone, consisting of 3/4 inch minus processed limestone, was used as base material for the DGB, WPA, and RWST. Properties for this material beneath the DGB are summarized below (Reference 1, Tables 2.5-56 and 2.5-57).

Properties of Crushed Stone

Average Maximum Dry Density	144 pcf
Average Minimum Dry Density	102 pcf
Average Field Density	133 pcf
Average Relative Density	81%

Average Gradation

Size	1-1/4"	1"	3/4"	3/8"	#4	#10	#40	#200
% Passing	100.0	99.4	92.6	70.1	49.6	29.8	10.4	3.7

A comparison was made with materials tested by Seed, et al (Reference 12). Details are presented in Sections 0-2.2 and 0-10.0 of Attachment 0-1. The Pyramid material was selected for comparison based on the similarity in the shape of the gradation curves and because both materials were produced by crushing. Using curves prepared from test data for the Pyramid material, an interpolated value of $(K_2)_{max} = 94$ is obtained for a relative density of 81%.

Geophysical surveys were performed adjacent to the north side of the DGB to evaluate dynamic properties of the crushed stone (Reference 13). It should be noted that the data was affected by disturbance of the crushed stone during drilling, and the refraction data was judged to be more reliable than data from the crosshole/downhole surveys.

Results of the geophysical survey were evaluated in previous studies (Reference 4). A value of $(K_2)_{max} = 108$ was obtained from the refraction data, however, it was necessary to use an assumed shear wave velocity in the underlying basal gravel. In addition, the boreholes apparently did not intersect the full backfill section, because reported thicknesses of crushed stone do not agree with the typical section for the DGB (Reference 1, Figure 2.5-226). Boreholes encountered an average thickness of 10 feet of crushed stone backfill, overlying 10 feet of basal gravel, whereas the typical section indicates a 19 foot thickness of crushed stone overlying an 8 foot thickness of basal gravel. This is probably due to irregularity of the excavation and should not have any significant effect on evaluation of dynamic soil properties.

In summary, estimates of $(K_2)_{max}$ based on geophysical surveys may be influenced by disturbance during drilling and uncertainty regarding the subsurface profile. Accurate records are available for in-place relative density. Using data developed by Seed, et al (Reference 12), an approximate value of $(K_2)_{max} = 94$ is indicated. Previous studies (References 2 and 4) have suggested values of 100 and 120.

A design value of $(K_2)_{max} = 100$ is recommended for crushed stone.

Figure O-5 shows the recommended curve of shear modulus reduction with shear strain. Details regarding selection of this curve are presented in Sections O-2.1 and O-2.2 of Attachment O-1.

Figure O-6 shows the recommended damping ratio curve. Details regarding selection of this curve are presented in Sections O-4.1 and O-4.3 of Attachment O-1.

06.0 IN-SITU MATERIALS - BECHTEL REVIEW

In-situ materials include fine-grained soils, basal gravel, and weathered shale. Fine grained soil deposits consist of clay, silty clay, sandy clay, clayey sand, silt, and silty sand. Soils are generally cohesive in the upper zone with clay fractions predominating, grading to less cohesive or non-plastic soils at depth where silt and sand fractions predominate. Available geophysical data for in-situ soils is summarized in Table O-4. Two zones are of interest; cohesive soils (CL, ML) which are found in the upper portion of the profile, and non-plastic soils (SM, ML) found generally in the lower 2 to 5 feet of the profile. Underlying these materials are basal gravel and weathered shale.

06.1 In-Situ Cohesive Soils

In-situ soils are found beneath the ADGB and the CDWEB. Available geophysical data is summarized in Table O-4. Additional details are presented in Sections O-8.0 to O-8.5 of Attachment O-1.

This section applies to cohesive soils (CL, ML) found in the upper portion of the soil profile, at a depth of 0 to about 20 feet below original ground surface. These materials generally have N values ranging from 10 to 30 blows per foot. Occasional thin deposits of low plasticity or non-plastic sands or silty sands may be found in this portion of the profile. The presence of these thin deposits is not considered significant with regard to selection of recommended soil properties.

Relatively low shear wave velocities were obtained for surficial soil deposits. In the depth zone of 0 to 6 ft, typical values range between 500 and 600 fps. Surficial soils are not of interest in this study as this upper zone was normally removed

prior to construction of facilities. For the remaining soil profile, velocities range generally from 800 to 1200 fps, with a few values to 1600 fps. These values are nearly identical with shear wave velocities calculated from the range of G_{max} values for Class A Backfill. Additionally, it was found that moist density for in-situ soils was the same as moist density for Class A Backfill, i.e., 120 pcf.

Resonant column testing results are available for undisturbed samples of cohesive soils taken from the Low Level Radwaste Borrow Area for shear strain ranges of 10^{-3} to 10^{-2} percent (Reference 7). To obtain G_{max}, results obtained at a shear strain amplitude of 10^{-3} percent were extrapolated for a shear strain amplitude of 10^{-4} percent using typical curves for reduction of shear modulus with shear strain for clays. Details are presented in Section O-9.0 of Attachment O-1. Results are shown in Figure O-7.

For a moisture content of 30%, laboratory data indicates values of G_{max} ranging from 800 to 3500 ksf. Previous studies have shown that shear modulus values at low shear strain obtained by laboratory testing of undisturbed samples are lower than corresponding values obtained from in-situ field measurements. This is attributed primarily to sample disturbance and time-dependent increases in shear modulus for laboratory specimens (Reference 14). A factor of 2.5 was used by Seed and Idriss (Reference 11) to reflect these effects for various laboratory test methods. Subsequent studies by Arango, et al (Reference 10) obtained a factor of 2.0 specifically for resonant column testing, which is used here, giving values of G_{max} ranging from 1600 to 7000 ksf. Using a moist density of 120 pcf, corresponding shear wave velocities ranging from 650 to 1370 fps are obtained, which is consistent with values obtained from geophysical surveys.

For in-situ cohesive soils the available data suggests an approximate range of G_{max} of 3,000 to 6,000 ksf. A design value of G_{max} = 4,500 ksf is recommended.

Figure O-8 shows the recommended curve of shear modulus reduction with shear strain, based on an undrained shear strength of 1.2 ksf. Details regarding selection of this curve are presented in Sections O-1.1 and O-1.3 of Attachment O-1. For specific areas/structures where a higher strength can be justified, the family of curves shown in Figure O1-5 of Attachment 1 can be used to obtain the appropriate modulus reduction curve.

Figure O-9 shows the recommended damping ratio curve. Details regarding selection of this curve are presented in Sections O-4.1 and O-4.2 of Attachment O-1.

06.2 In-Situ Non-Plastic Soils

In-situ soils are found beneath the ADGB and the CDWEB. Available geophysical data is summarized in Table 0-4. Additional details are presented in Sections 0-8.0 to 0-8.5 of Attachment 0-1.

This section applies to non-plastic soils (SM, ML) which are found generally in the lower 2 to 5 feet of the soil profile, at an approximate depth of 20 feet below original ground surface. These materials typically have N values of less than 10 blows per foot.

An average shear wave velocity of 613 fps was obtained for this material by crosshole testing. Using this value and a moist unit weight of 120 pcf, a $(K_2)_{max} = 38$ was obtained for a typical profile. This is a low value when compared with typical ranges of $(K_2)_{max}$ for sands (Reference 11), but is verified by a separate evaluation based on measured in-situ void ratios for this material (see Section 0-11.0 of Attachment 0-1).

A design value of $(K_2)_{max} = 38$ is recommended for in-situ non-plastic soils in the lower portion of the profile.

Figure 0-10 shows the recommended curve of shear modulus reduction with shear strain. Details regarding selection of this curve are presented in Sections 0-1.1 and 0-1.4 of Attachment 0-1.

Figure 0-11 shows the recommended damping ratio curve. Details regarding selection of this curve are presented in Sections 0-4.1 and 0-4.3 of Attachment 0-1.

06.3 Basal Gravel

Basal gravel is present beneath the DGB, ADGB, CDWEB, RWST, and ERCW Pipe Tunnel. Representative properties for this material are dry density of 125 pcf, specific gravity of 2.60, and void ratio of 0.30. Average gradation and envelopes for samples taken from the main plant area are shown in Figure 0-12 (Reference 1, Tables 2.5-42 and 43 and Figures 2.5-366 through 371). Additional information is presented in Section 0-2.3 of Attachment 0-1.

Boreholes SS-26 through SS-30 (Reference 1, Figure 2.5-190) were drilled to define in-situ materials at each corner of the DGB. The drilling program included standard penetration testing. For these holes an extremely high average count of N = 124 blows per foot was obtained in the basal gravel.

A subsequent investigation was conducted in the main plant area with drilling of Boreholes US-125 through US-130 (Reference 1, Tables 2.5-42 and 43, and Figures 2.5-358 through 364). Blow counts in the basal gravel ranged from 14 to 50+. Relative density for the basal gravels was estimated using relationships between blow counts and effective overburden pressure (Reference 15). Details are presented in Section O-2.3 of Attachment O-1. Estimated relative density for nine of the eleven zones tested ranged from 80% to more than 90%, while values of slightly over 60% were obtained for the remaining two zones. It should be noted, however, that the basal gravel contains particles up to five inches in diameter and data from standard penetration testing may not be reliable. Separate calculations (see Section O-2.3 of Attachment O-1) indicate that actual relative density may range from 70% to 90%.

Very little data is available from geophysical surveys specifically for basal gravel. Downhole testing generally applies to a combination of in-situ soils and basal gravel/ weathered shale. Refraction surveys for the DGB (Reference 1, Table 2.5-233) gave an average shear wave velocity of 1660 fps in an approximate depth zone of 19 to 39 feet. Borehole logs indicate that this zone consists of 6 feet of basal gravel overlying 14 feet of weathered to fresh shale bedrock. The shear wave velocity of 1660 fps obtained in this zone is believed to be a representative value for the basal gravel because the seismic pulse recognized in the refraction data coming from this zone is most likely propagating along the top of bedrock.

Values of $(K_2)_{max}$ for natural gravels, based on laboratory testing of large diameter remolded samples, were determined by Seed, Wong, Idriss, and Tokimatsu in 1984 (Reference 12). $(K_2)_{max}$ values ranged from about 60 to 140, depending upon relative density of the sample. Subsequent investigations have shown that in-situ values are often much higher than those determined for remolded laboratory samples. The following are in-situ measured field values and corresponding values of $(K_2)_{max}$ for gravels in the same approximate depth range.

Location	Depth (ft)	Vs (fps)	$(K_2)_{max}$	Ref. No.
San Juan, Argentina	24	1540	204	16
Lima, Peru	26	2300	463	17
Alicura Dam, Argentina	35	1800	331	18
Santiago, Chile	44	2180	316	19

Using a shear wave velocity of 1660 fps and a saturated unit weight of 143 pcf for a typical section beneath the DGB, a $(K_2)_{max}$ = 365 is obtained (see Section 0-12.0 of Attachment 0-1). As can be seen, this is consistent with values obtained from in-situ field measurements.

A design value of $(K_2)_{max}$ = 365 is recommended for basal gravel.

Figure 0-13 shows the recommended curve of shear modulus reduction with strain. Details regarding selection of this curve are presented in Sections 0-2.0 and 0-2.3 of Attachment 0-1.

Figure 0-14 shows the recommended damping ratio curve. Details regarding selection of this curve are presented in Sections 0-4.1 and 0-4.3 of Attachment 0-1.

06.4 Weathered Shale

Weathered shale is present beneath all listed structures except the WPA and NSVR. Original subsurface investigations generally show less than five feet of weathered shale overlying bedrock. For these investigations bedrock was defined as auger refusal. Subsequent investigations in the main plant area show a range in thickness of 8 to 23 feet, with an average of 15 feet. The basis for defining top of bedrock is not given. Additional information is presented in Section 0-3.2 of Attachment 0-1.

For the DGB, an average shear wave velocity of 1660 fps was obtained in the zone of silty sands and gravel directly above bedrock (Reference 1, Figure 2.5-233). For the Intake Channel, an average shear wave velocity of 2218 fps was obtained in the zone of gravel directly above bedrock (Reference 1, Figure 2.5-232).

Original seismic surveys in bedrock beneath the reactor building consistently show shear wave velocities in the range of 5300 to 7200 fps, with an average of 6300 fps (Reference 1, Figure 2.5-109).

There is no site specific data regarding shear wave velocity in weathered shale for WBN. TVA reports a measured shear wave velocity of 1800 fps in similar material at the Fort Loudoun Dam. In the absence of other data, this shear wave velocity of 1800 fps is recommended for use at WBN.

Figure 0-15 shows recommended damping ratios and reduction of shear modulus with shear strain. Details regarding selection of these curves are presented in Sections 0.30 and 0.50 of Attachment 0-1.

07.0 CONCLUSIONS AND RECOMMENDATIONS

The previous sections provide estimates of dynamic soil properties for WBN geotechnical materials, based on general reference sources and available site-specific data. Recommended values are summarized in Table 0-5.

It is TVA's practice to vary shear moduli by ± 50% to allow for possible variations in soil properties. TVA has requested Bechtel's comments on this practice.

For Class A Backfill and in-situ cohesive soils, this appears to be reasonable, and brackets the possible range of shear moduli identified for these materials.

For in-situ non-plastic soils, a relatively low value of $(K_2)_{max} = 38$ was obtained. Application of a + 50% variation in shear modulus would appear to be appropriate, however, use of a - 50% variation would result in a shear modulus well below reasonable limits for this material. Typical values of $(K_2)_{max}$ obtained by Seed and Idriss for sands (Reference 11) range from about 30 to 70.

For crushed stone backfill, a value of $(K_2)_{max} = 100$ is recommended. Typical values for similar materials obtained by Seed, et al (Reference 12) range from about 70 to 140. Use of a ± 50% variation in shear modulus would go well beyond what appear to be reasonable limits for this material.

For basal gravel, a value of $(K_2)_{max} = 365$ is recommended, based on in-situ data for similar materials. Typical values range from about 200 to 500. Application of a - 50% variation in shear modulus would appear reasonable, however, use of a + 50% variation would result in a shear modulus well above observed values, but may not significantly affect the analysis.

A shear wave velocity of 1800 fps is assumed for weathered shale. Due to the lack of site specific information for this material, a ± 50% variation in shear modulus is considered appropriate.

08.0 REFERENCES

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TABLE O-1

CATEGORY I STRUCTURES/FEATURES
SUPPORTED BY SOILS OR INFLUENCED BY SOILS

CATEGORY I	SITE GEOTECHNICAL MATERIALS				
STRUCTURE/FEATURE	CLASS A BACKFILL	CRUSHED STONE	IN SITU SOIL	BASAL GRAVEL	WEATHERED SHALE
Diesel Generator Building	X	X	NA	X	X
Additional Diesel Generator Building	X	NA	X	X	X
Waste Packaging Area*	NA	X	NA	NA	NA
Condensate Demineralizer Waste Evaporator Building*	X	?	?	?	X
North Steam Valve Room	X	NA	NA	NA	NA
Refueling Water Storage Tank	X	X	NA	X	X
Essential Raw Cooling Water Pipe Tunnel	X	X	NA	X	X
Essential Raw Cooling Water Pipe Support Slab and Sheet Pile Retaining Wall	X	NA	NA	NA	X

* - Included due to potential effect on adjacent Category I structures.
X - Geotechnical material involved with structure/feature
NA - Not applicable for structure/feature
? - Uncertain about geotechnical material involved with structure/feature.

TABLE O-2

DYNAMIC SOIL PROPERTIES - ORIGINAL EVALUATIONS

CATEGORY I STRUCTURE/FEATURE	SITE GEOTECHNICAL MATERIALS CLASS A BACKFILL	CRUSHED STONE	IN SITU SOIL	BASAL GRAVEL	WEATHERED SHALE
Diesel Generator Building** (Ref. 1, Table 3.7-3)	*	V_s=1650 λ=10	NA	V_s=1650 λ=10	V_s=1650 λ=10
Additional Diesel Generator Building (Ref. 1, Table 3.7-3)	*	NA	V_s=1000 λ=10	V_s=1000 λ=10	V_s=1000 λ=10
Waste Packaging Area (Ref. 1, Table 3.7-3)	NA	V_s=1650 λ=10	NA	NA	NA
Condensate Demineralizer Waste Evaporator Building (Ref. 1, Table 3.7-3)	*	V_s=761 λ=10	V_s=761 λ=10	V_s=761 λ=10	V_s=761 λ=10
North Steam Valve Room	*	NA	NA	NA	NA
Refueling Water Storage Tank (Ref. 1, Table 3.7-3)	*	V_s=1008 λ=10	NA	V_s=1008 λ=10	V_s=1008 λ=10
Essential Raw Cooling Water Pipe Tunnel (Ref. TVA Calcs.)	*	V_s=1150 λ=10	NA	V_s=1150 λ=10	V_s=1150 λ=10
Essential Raw Cooling Water Pipe Support Slab and Sheet Pile Retaining Wall (Ref. TVA Calculations)	V_s=1200 λ=10	NA	NA	NA	V_s=1200 λ=10

V_s = Shear wave velocity (fps)
λ = Percent damping ratio

* Not available

** In August 1987 (Ref. 2) soil properties for the DGB were revised as follows:

Material	Properties
Crushed stone	$(K_2)_{max}$ = 120, G & λ strain dependent
Basal gravel	$(K_2)_{max}$ = 90, G & λ strain dependent
Weathered shale	V_s 1650fps, G & λ strain dependent

TABLE O-3

DYNAMIC SOIL PROPERTIES - CONSULTANT'S REVIEW

STRUCTURE	MATERIAL	PROPERTIES
Diesel Generator Building[1]	Crushed Stone	(K_2) max = 90 to 100, G& λ Strain Dependent
	Basal Gravel	(K_2) max = 72, G& λ Strain Dependent
Waste Packaging Area[2]	Crushed Stone	(K_2) max = 100 G& λ Strain Dependent
Condensate Demineralizer Waste Evaporator Building[2]	Class A Backfill	Vs = 1062 fps, G& λ Strain Dependent
	In Situ Soil	Vs = 1062 fps, G& λ Strain Dependent

1. Sargent & Lundy, 1988

2. Stone & Webster, 1988

TABLE 0-4

SHEAR WAVE VELOCITY FROM SEISMIC SURVEYS

IN-SITU SOILS

Location	Type of Measurement	Appx. Depth Zone (ft)	Shear Wave Velocity (fps) Average	Range
DGB (Ref. 1, Figure 2.5-233)	Downhole	0 to 30	1040	970-1160
	Refraction	0 to 6	600	550-640
	Refraction	6 to 19	1380	1180-1590
Low Level Radwaste Borrow Area (Ref. 6)	Crosshole	3 to 23	810	510-940
	Downhole	3 to 28	940	-
	Crosshole	6 to 24	800	450-960
	Downhole	6 to 26	1600	-
	Crosshole	3 to 20	760	720-780
	Downhole	3 to 20	1420	-
Intake Channel (Ref. 1, Figures 2.5-228 to 232)	Downhole	0 to 40	940	830-1060
	Refraction	0 to 40	970*	840-1190*
Class IE Conduits and ERCW Piping (Ref. 1, Table 2.5-17)	Downhole	0 to 34	1350**	930-1760**

* - Includes all materials above bedrock. Refraction zones do not reflect conditions described in borehole logs.

** - Blow counts from standard penetration testing were higher than for other areas listed.

TABLE 0-5

SUMMARY OF RECOMMENDED DYNAMIC SOIL PROPERTIES

Property	Class A Backfill	Crushed Stone	In-Situ Soils Cohesive	In-Situ Soils Non-Plastic	Basal Gravel	Weathered Shale
γ_{total}	120 pcf	133 pcf[1]	120 pcf	120 pcf	143 pcf[2]	127 pcf[2]
G max	4500 ksf	--	4500 ksf	--	--	(V_s=1800 fps)
$(K_2)_{max}$	--	100	--	38	365	--
G vs. Strain	Figure 0-3	Figure 0-5	Figure 0-8	Figure 0-10	Figure 0-13	Figure 0-15
Damping Ratios	Figure 0-4	Figure 0-6	Figure 0-9	Figure 0-11	Figure 0-14	Figure 0-15

1 - For total weight below water table, use γ_{sat} = 142 pcf (Reference 1, Table 2.5-44).

2 - Reference 1, Table 2.5-44

Note: Groundwater level will vary depending on structure under consideration, and should be selected from appropriate borehole logs and groundwater measurements.

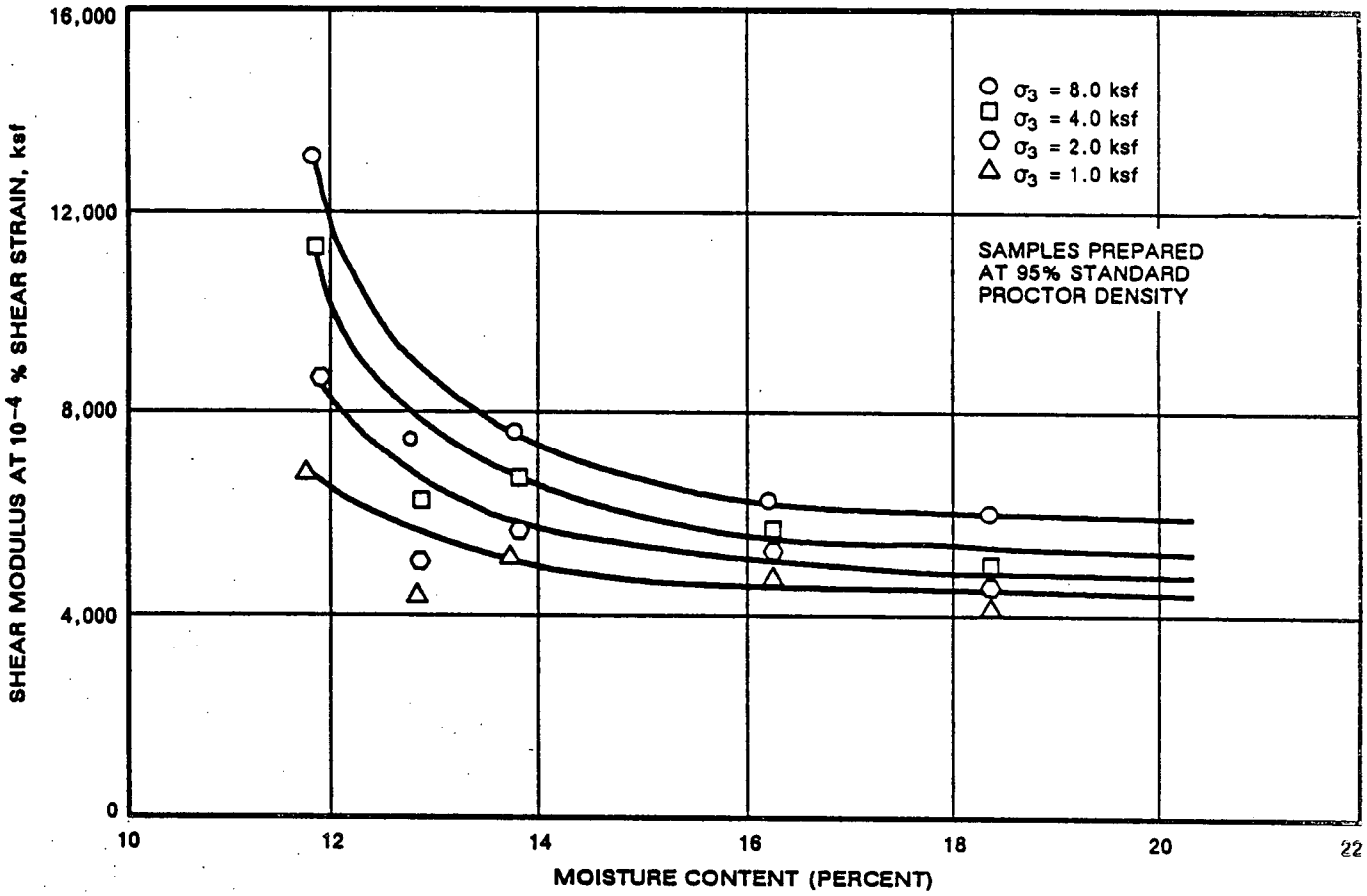

Figure O-1
SHEAR MODULUS vs. MOISTURE CONTENT, REMOLDED CL
LOW LEVEL RADWASTE BORROW AREA

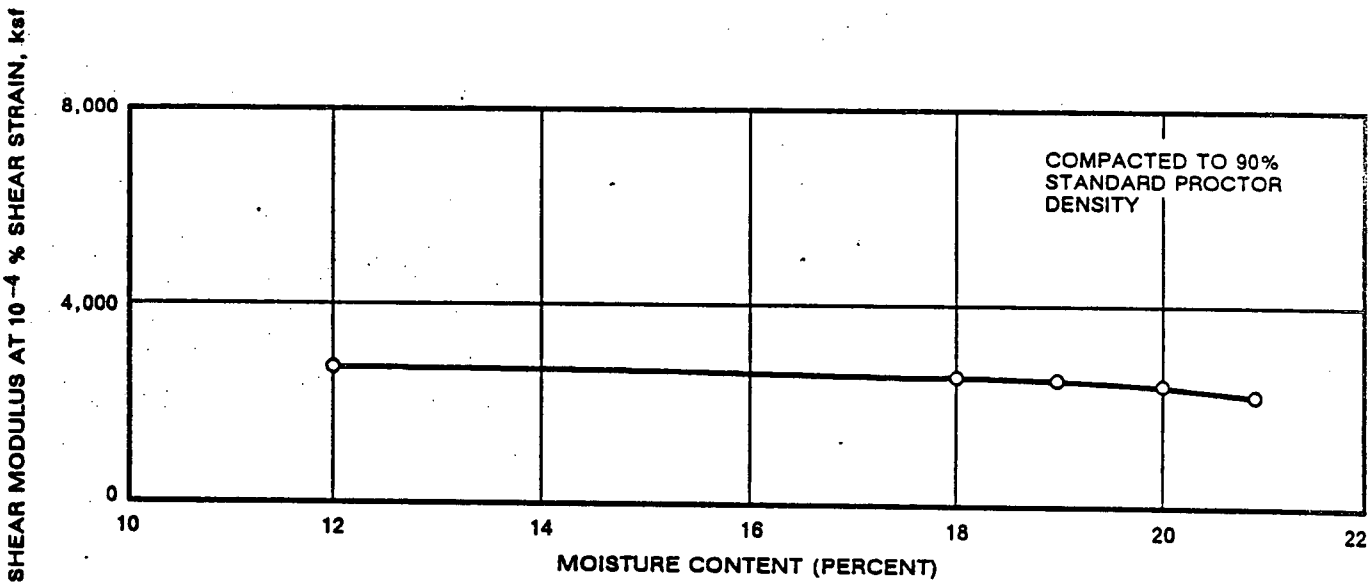

Figure O-2
SHEAR MODULUS vs. MOISTURE CONTENT FROM SPT
EMERGENCY RAW COOLING WATER PIPELINE

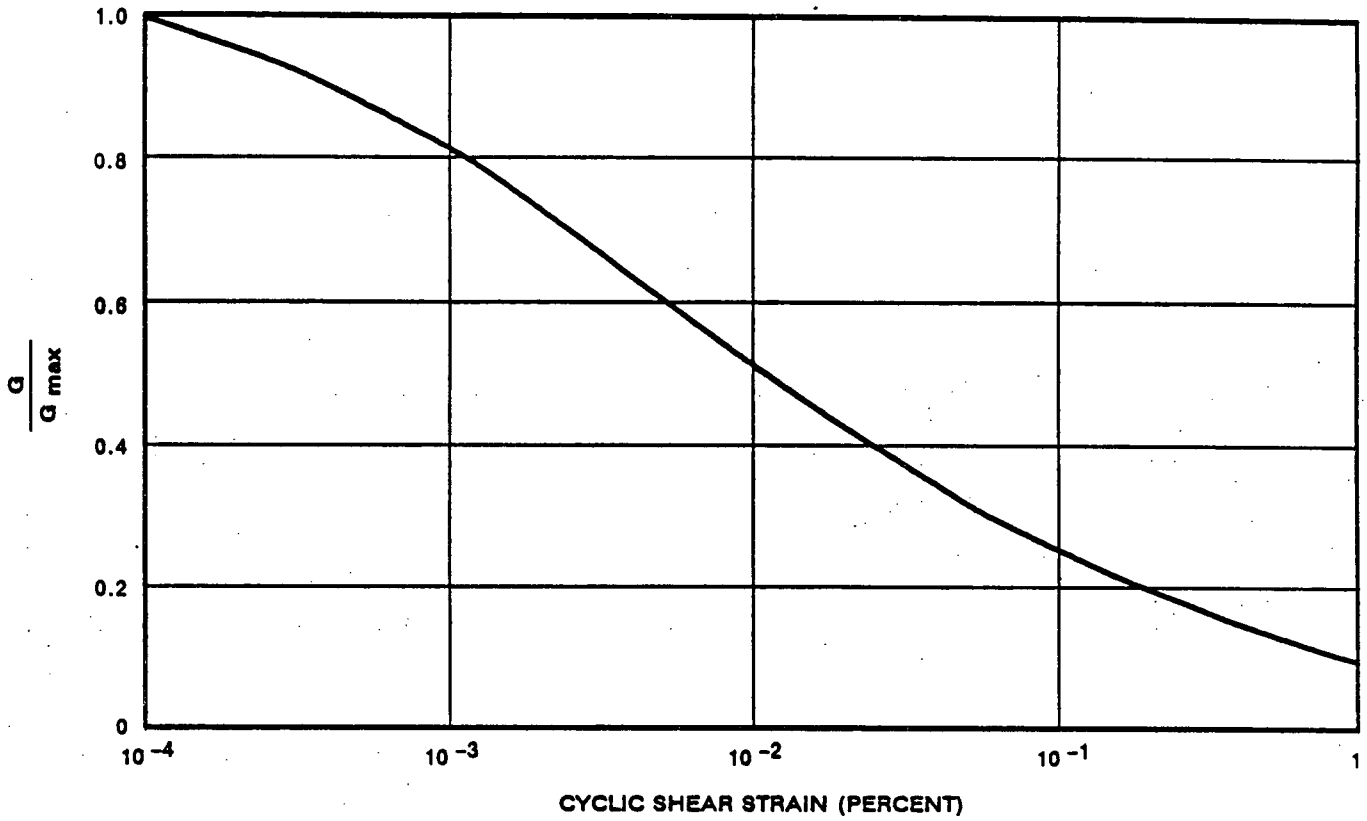

Figure O-3
SHEAR MODULUS REDUCTION WITH SHEAR STRAIN - CLASS A BACKFILL

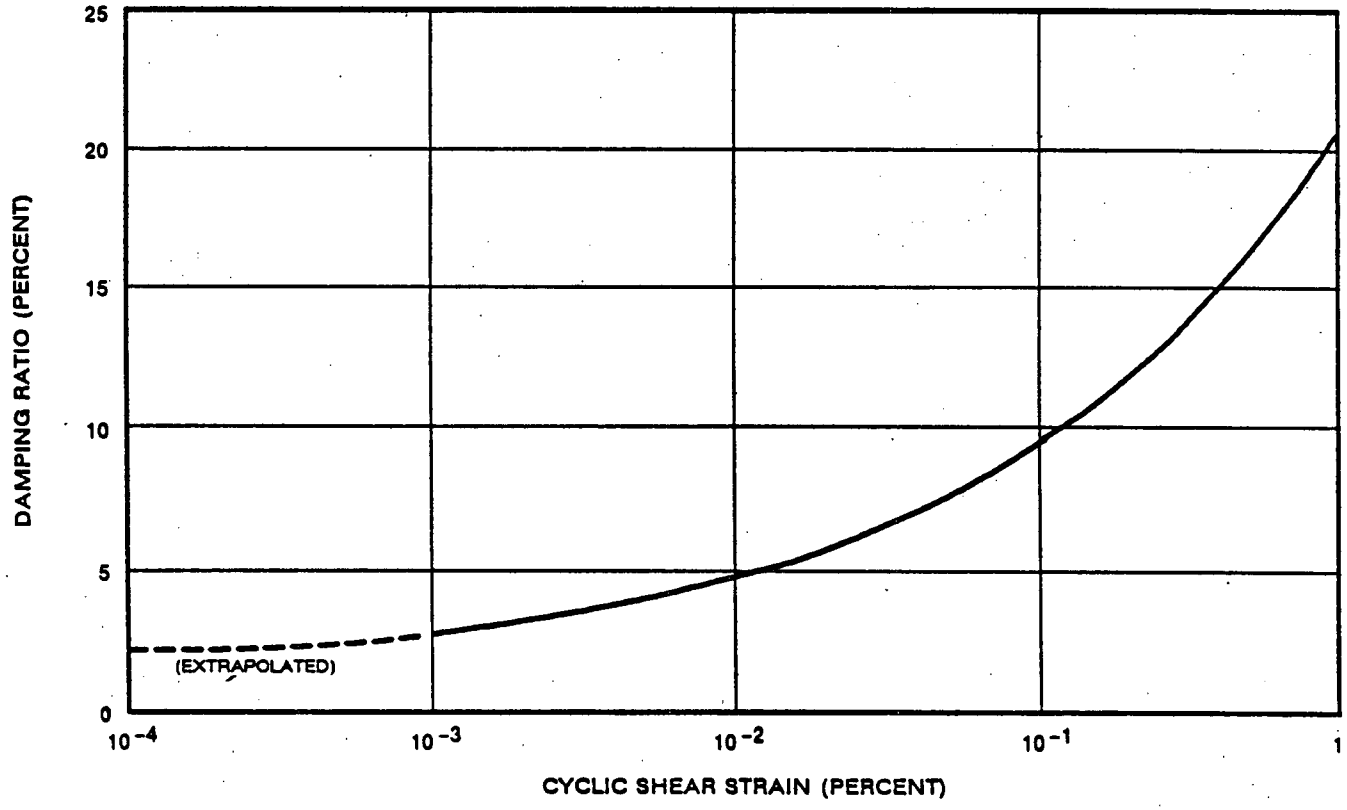

Figure O-4
DAMPING RATIOS - CLASS A BACKFILL

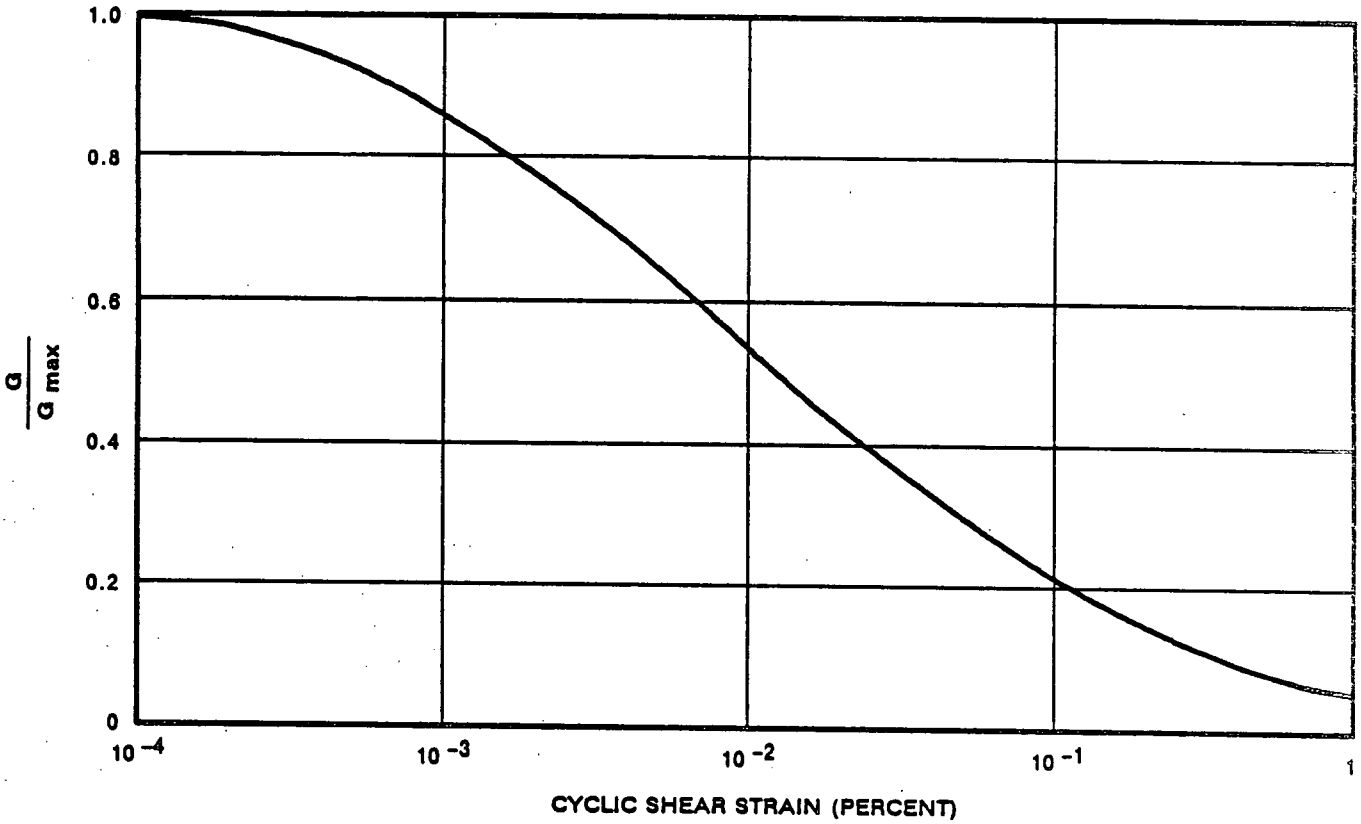

Figure O-5
SHEAR MODULUS REDUCTION WITH SHEAR STRAIN - CRUSHED STONE BACKFILL

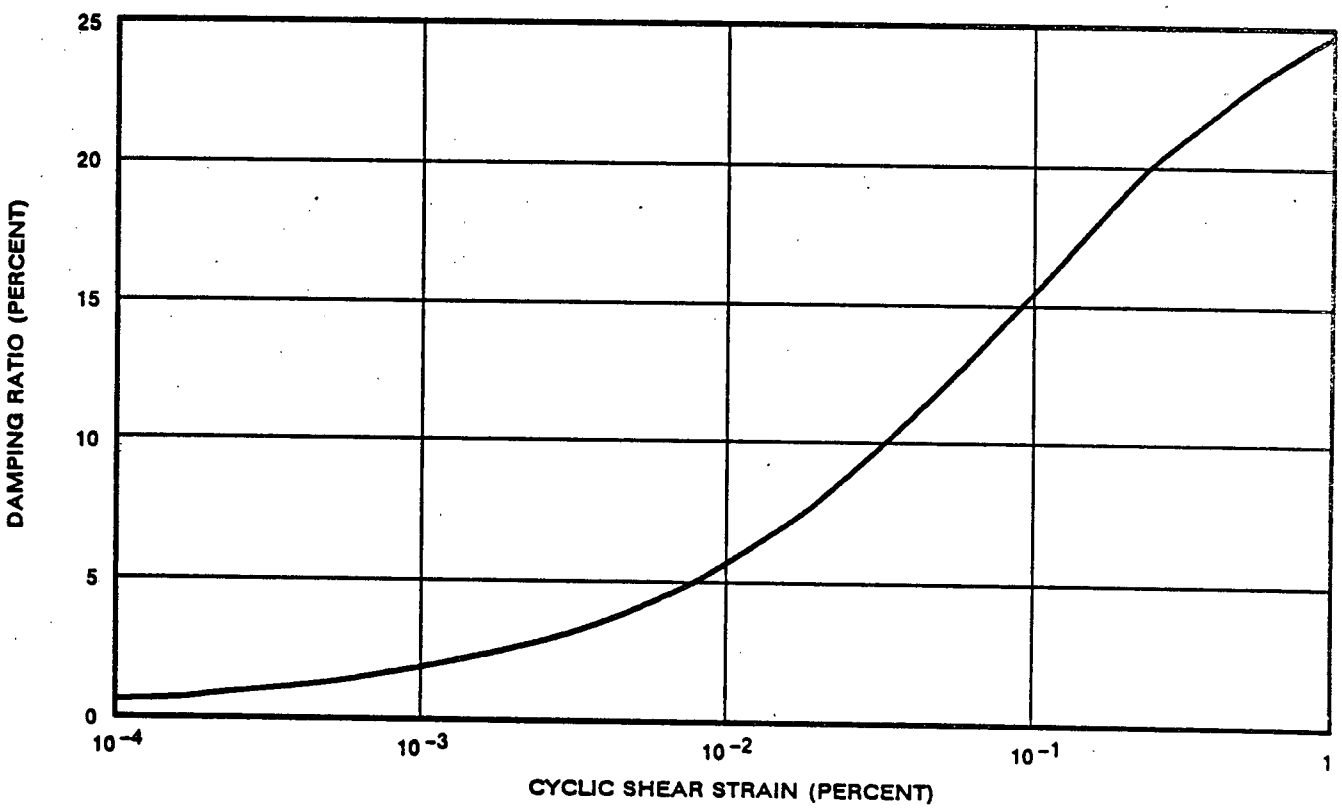

Figure O-6
DAMPING RATIOS - CRUSHED STONE BACKFILL

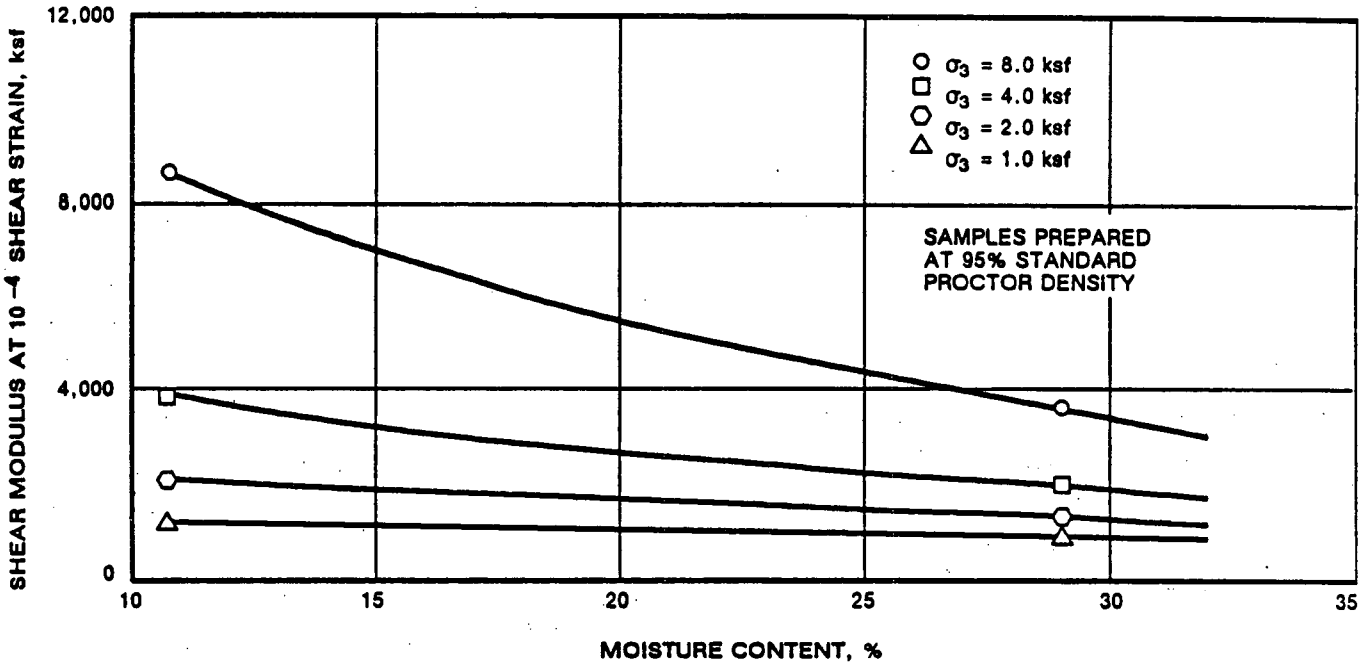

Figure O-7
SHEAR MODULUS vs. MOISTURE CONTENT, UNDISTURBED CL
LOW LEVEL RADWASTE BORROW AREA

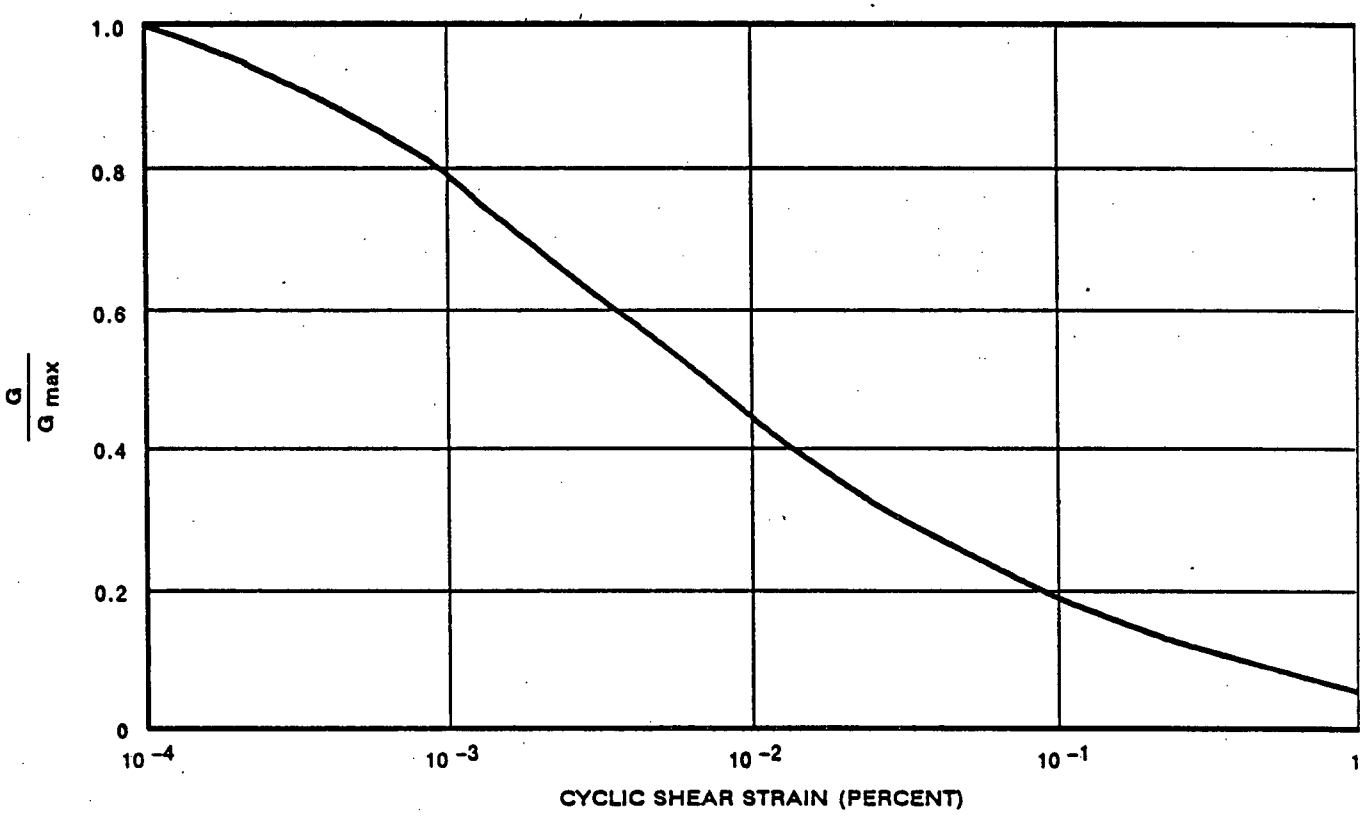

Figure O-8
SHEAR MODULUS REDUCTION WITH SHEAR STRAIN - IN SITU COHESIVE SOILS

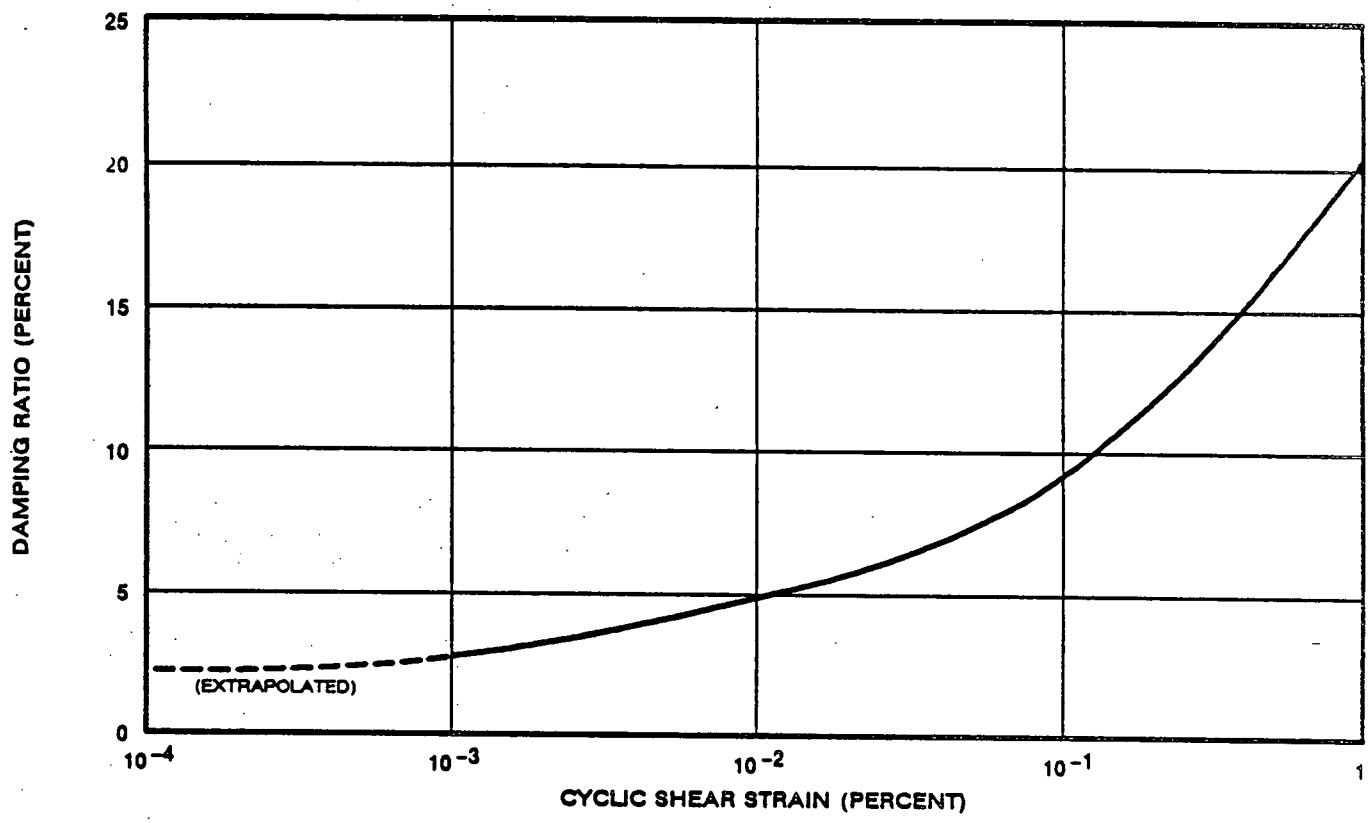

Figure O-9
DAMPING RATIOS - IN SITU COHESIVE SOILS

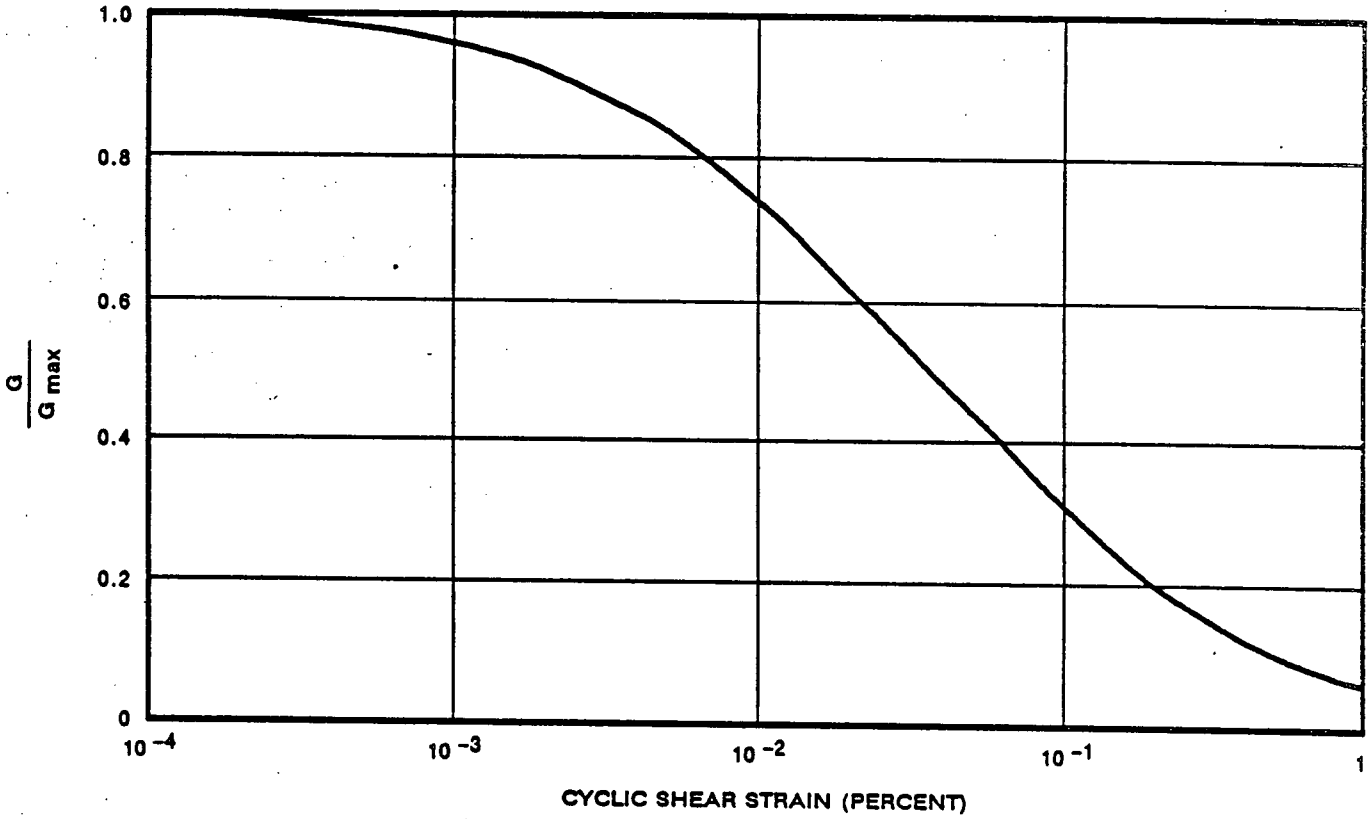

Figure O-10
SHEAR MODULUS REDUCTION WITH SHEAR STRAIN - NON-PLASTIC IN SITU SOILS

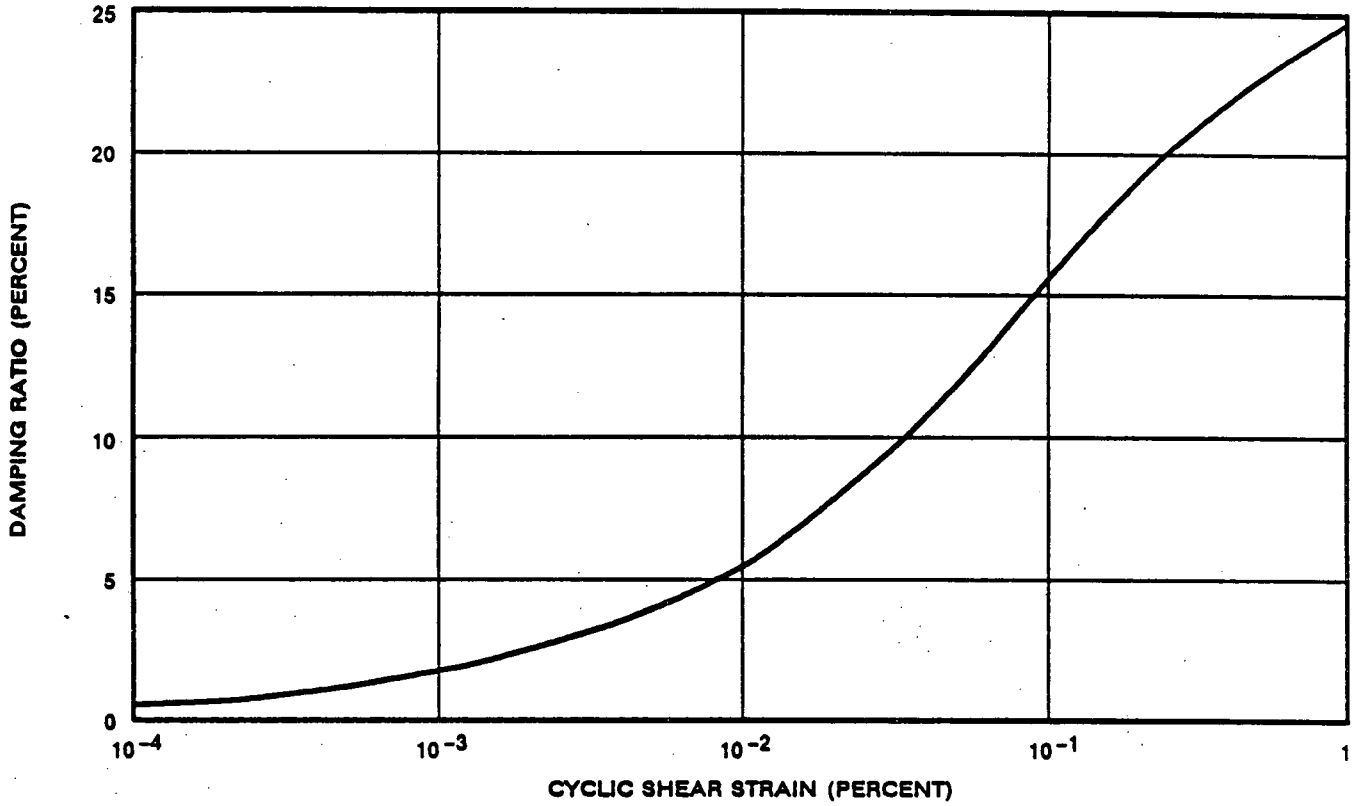

Figure O-11
DAMPING RATIOS - NON-PLASTIC IN SITU SOILS

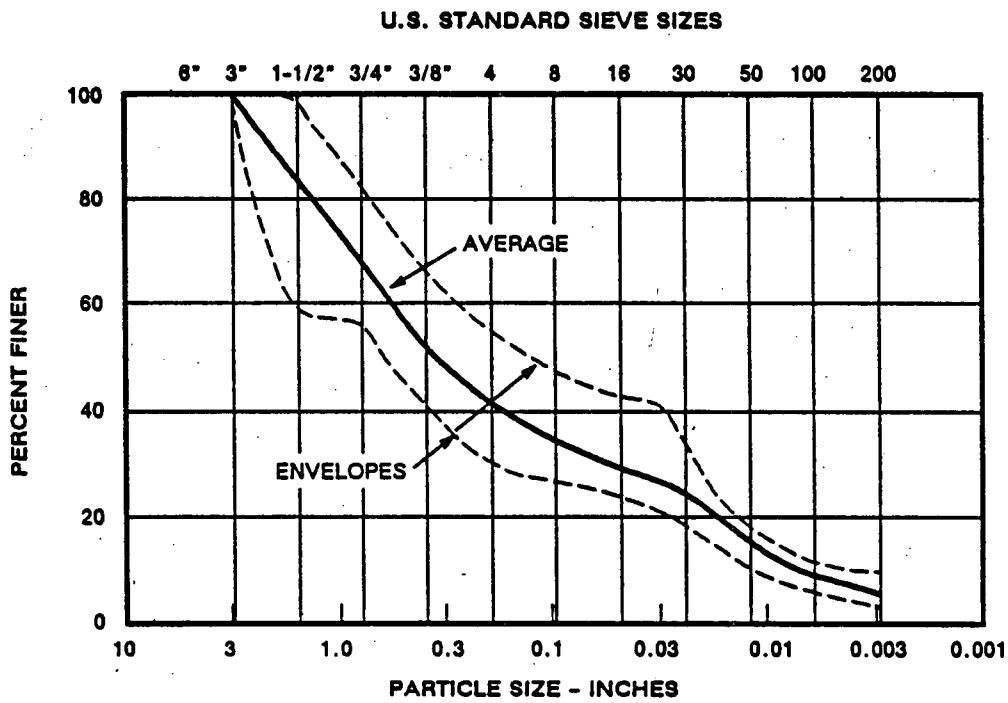

Figure O-12
PARTICLE SIZE DISTRIBUTION FOR BASAL GRAVEL

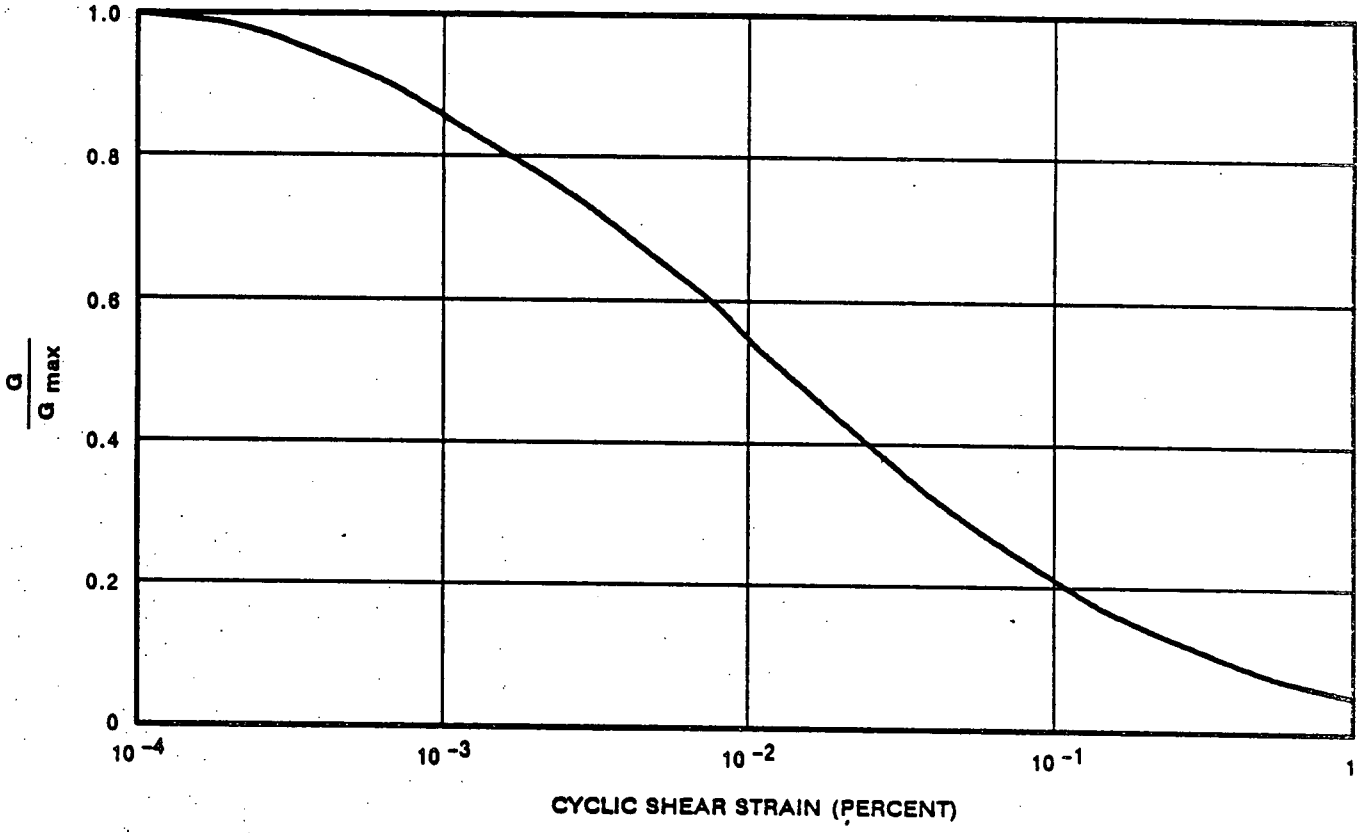

Figure O-13
SHEAR MODULUS REDUCTION WITH SHEAR STRAIN - BASAL GRAVEL

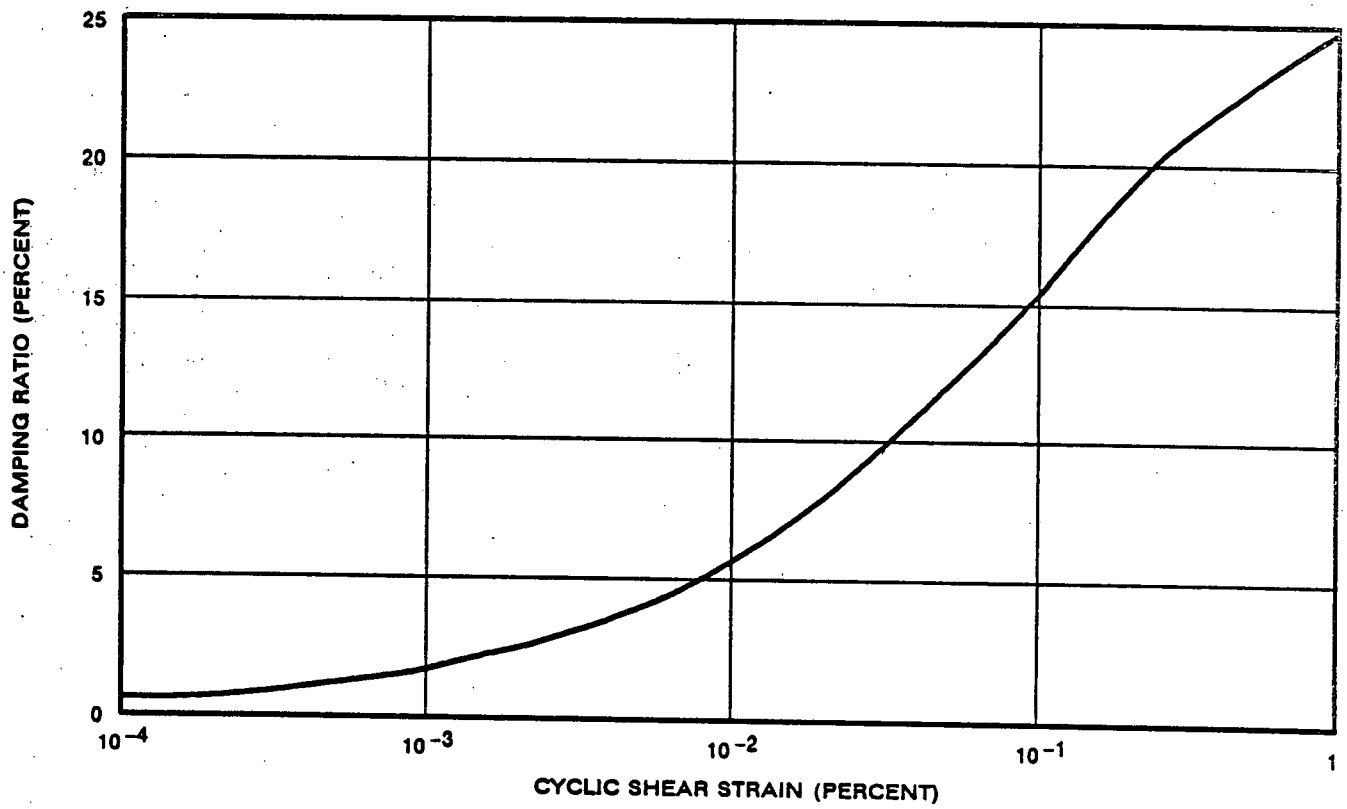

Figure O-14
DAMPING RATIOS - BASAL GRAVEL

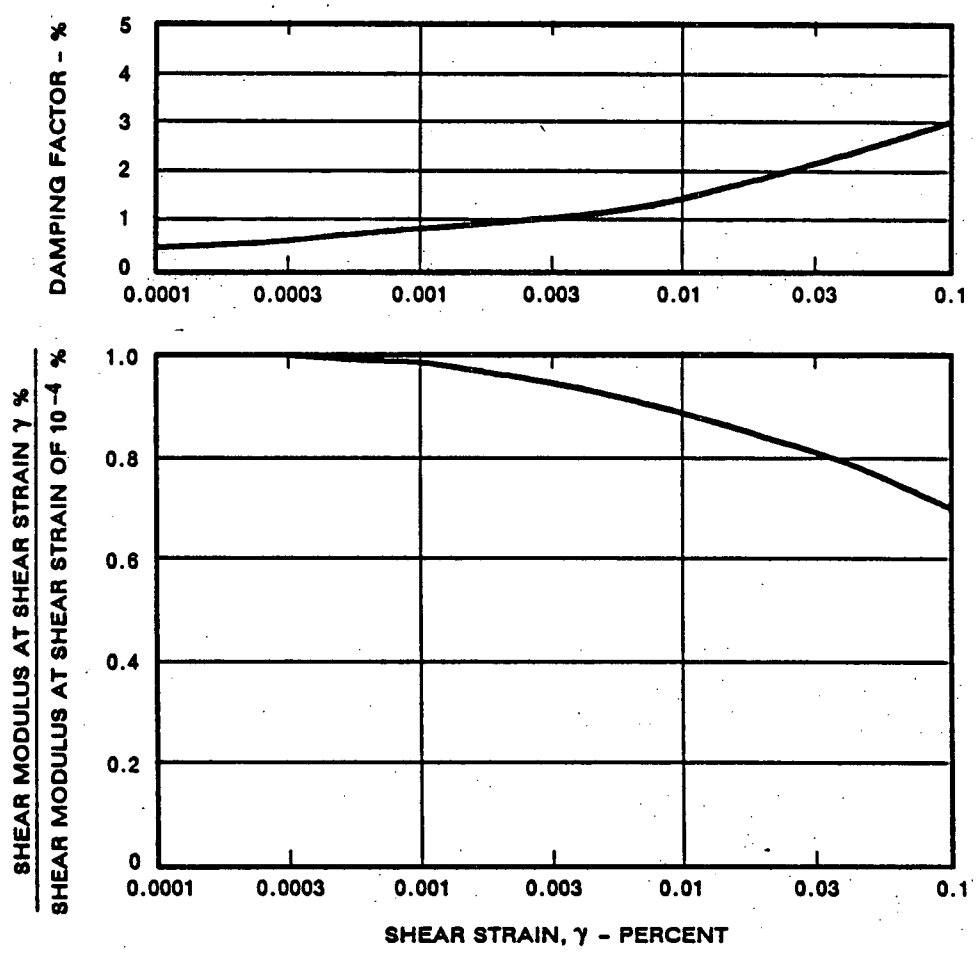

Figure O-15
SHEAR MODULI AND DAMPING CHARACTERISTICS FOR ROCK

ATTACHMENT O-1

CALCULATIONS

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ATTACHMENT O-1

CALCULATIONS

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CALCULATIONS

0-1.1.0 CURVES OF REDUCTION OF SHEAR MODULUS WITH SHEAR STRAIN FOR SANDS AND CLAYS

Purpose: To derive appropriate modulus reduction curves for WBN in-situ soils using general reference sources and site specific geotechnical data.

0-1.1.1 Discussion

Basic modulus reduction curves for sands and soft clays were presented by Seed and Idriss in 1970 (Reference 11). These curves were based primarily on data obtained from laboratory testing, supplemented by field shear wave velocity measurements at low strain levels. Data for in-situ field measurements at that time were limited to shear strain levels of less than 10^{-3} percent. These curves provided a basis for determining strain dependent soil properties.

A need was recognized for a test method which would provide in-situ modulus values over a wider range of shear strain, particularly in the range normally caused by an actual earthquake (Reference 21). Subsequent investigations were conducted by Shannon & Wilson, Inc. - Agbabian Associates (Reference 22) using a specialized cross-hole geophysical technique to obtain in-situ modulus values for shear strains ranging from 10^{-4} to 10^{-1} percent. Several sites were investigated in areas with various geologic conditions, including terrace deposits of alluvial or marine origin, glacial deposits, and residual profiles. This data was evaluated by Arango, Moriwaki, and Brown in 1978 (Reference 10), and compared with the average modulus reduction curves proposed by Seed and Idriss. Results are shown in Figure 0-1-1.

Additional studies involving clays at a variety of sites have shown a relationship between undrained shear strength and shear modulus. Curves showing this relationship were published by Woodward - Clyde Consultants in 1985 (Reference 8), and are shown in Figure 0-1-2. A composite plot of all data is shown in Figure 0-1-3. It can be seen that there is very close agreement for the lower boundary curves representing soft clays. For stiff clays, the upper boundary is only slightly below the Seed and Idriss curve for sand at lower strain levels. At higher strain levels, the upper boundary agrees well with the upper limit of in-situ values. These curves are applied in the following sections to site specific data obtained for the Watts Bar Nuclear Plant.

O-1.1.2 Class A Backfill

Class A backfill consists of sandy or silty lean clay (CL) compacted to a minimum of 95% of Standard Proctor Density. The modulus reduction curve for this material will be obtained using undrained shear strength and the relationships presented in Section O-1.1.1.

Borrow Area No. 4 was one source for any soil necessary for the construction of qualified fills (Reference 1, pages 2.5-114). For this material, an undrained shear strength (UU triaxial shear strength at zero normal stress) of 2.1 ksf was obtained (Reference 1, Table 2.5-12 and Figure 2.5-244). For a strength of 2.1 ksf, the modulus reduction curve shown in Figure O-1-4 is recommended.

For other borrow areas, modulus reduction curve shall be interpolated on the curves shown in Figure O-1-4 for the appropriate undrained shear strength.

O-1.1.3 Cohesive In-Situ Soils (CL, ML)

Fine-grained soils, consisting primarily of silty or sandy lean clays with alternating layers of cohesive silts, are present in the upper portion of the profile. In the main plant area, these materials are found in the approximate depth zone of 0 to 20 feet below original ground surface. Blow counts from standard penetration testing generally range from 10 to 30 blows per foot. The modulus reduction curve for these materials will be obtained using undrained shear strength and the relationships presented in Section O-1.1.1.

Undrained shear strength (UU triaxial shear strength at zero normal stress) is summarized below:

Low Level Radwaste Borrow Area (Reference 7, Table 2)

Depth below ground surface	-	6 to 21 feet
Average undrained shear strength	-	1.2 ksf

Diesel Generator Building (Reference 1, Table 2.5-6)

Depth below ground surface	-	16 to 22 feet
Average undrained shear strength	-	1.2 ksf

Essential Raw Cooling Water Supply (Reference 1, Table 2.5-7)

Depth below ground surface	-	15 feet
Undrained shear strength	-	1.9 ksf

Intake Channel (Reference 1, Table 2.5-8)

Depth below ground surface	-	3 to 18 feet
Average Undrained shear strength	-	1.5 ksf

Class IE Conduits

Depth below ground surface	-	2 to 20 feet
Average Undrained shear strength	-	2.9 ksf

An undrained shear strength (UU triaxial shear strength at zero normal stress) of 1.2 ksf is selected as being representative for the main plant area. For a strength of 1.2 ksf, the modulus reduction curve shown in Figure O-1-5 is recommended. For specific areas/structures where a higher strength can be justified, the family of curves shown in Figure O-1-5 could be used to obtain the appropriate modulus reduction curve.

O-1.1.4 Non-Plastic In-Situ Soils (SM, ML)

A zone of weak, non-plastic silt and silty sand is often found in the lower portion of the soil profile. In the main plant area, this zone has an approximate thickness of 2 to 5 feet, occurs at a depth of about 20 feet below original ground surface, with N values generally ranging from 1 to 10 blows per foot.

The average modulus reduction curve for sand as presented by Seed and Idriss (Figure O-1-1) is considered appropriate for this material.

O-1.2.0 CURVES OF REDUCTION OF SHEAR MODULUS WITH SHEAR STRAIN FOR GRAVELS

Purpose: To derive appropriate modulus reduction curves for WBN gravels using general reference sources and site specific geotechnical data.

O-1.2.1 Discussion

Modulus reduction curves for gravels were presented by Seed, Wong, Idriss, and Tokimatsu in 1984 (Reference 12). This data was based on a comprehensive series of laboratory tests performed on 12-inch diameter samples of natural gravels and crushed stone. The modulus reduction curve obtained for these materials, which is slightly flatter than the corresponding curve for sands (Reference 11), is shown in Figure O-1-6.

O-1.2.2 Crushed Stone Backfill

WBN crushed stone backfill consists of a 3/4-inch minus processed limestone. This material was used for engineered backfill beneath structures in areas requiring removal of unsuitable in-situ soils. Relative density and particle size distribution for this material are listed in Reference 1, Table 2.5-56 and 2.5-57.

Average in-place relative density was calculated from the values listed in Table 2.5-56:

D_r = 3002/37 = 81.1, say 81%

This value falls almost exactly at the mid-point of values for materials tested by Seed, et. al. in 1984.

Average particle size distribution was calculated from the values listed in Table 2.5-57:

Size		% Passing
1-1/4"		100.00
1"	8746/88 =	99.4
3/4"	8053/87 =	92.6
3/8"	6166/88 =	70.1
#4	4368/88 =	49.6
#10	2622/88 =	29.8
#40	916/88 =	10.4
#200	319/87 =	3.7

Figure O-1-7 shows the average gradation calculated above superimposed on plots for materials tested by Seed, et. al.

As can be seen, the shape of the gradation curve for Watts Bar crushed stone backfill is very similar to the tested materials. On the basis of similarity in relative density and gradation, it is concluded that the average modulus reduction curve obtained by Seed, et. al. (Figure O-1-6) is appropriate for Watts Bar crushed stone backfill.

O-1.2.3 Basal Gravel

Basal gravel, consisting of alluvial deposits with particle size up to 5 inches in diameter, is found at the site overlying shale bedrock. In the main plant area, this material has an average thickness of about 6 feet.

Average particle size distribution for this material was calculated from the plots shown in Reference 1, Table 2.5-366 through 371:

Size	% Passing						Average
	Borehole US-125	Borehole US-126	Borehole US-127	Borehole US-128	Borehole US-129	Borehole US-130	
3"	100	100	100	100	100	100	100
1-1/2"	88	81	98	74	98	58	83
3/4"	68	70	61	57	82	56	66
3/8"	51	56	48	46	65	41	51
#4	37	45	38	39	55	32	41
#8	29	38	32	33	48	28	35
#16	23	32	27	30	43	25	30
#30	18	25	18	26	34	20	24
#50	10	15	10	16	17	10	13
#100	5	10	5	7	12	8	8
#200	3	7	4	5	10	8	6

Figure O-1-8 shows the average gradation calculated above superimposed on plots for materials tested by Seed, et. al.

The gradation curve for Watts Bar basal gravel agrees reasonably well with the Oroville materials, and very well with the Livermore natural gravel.

Relative density of the basal gravel is of particular interest in estimating dynamic soil properties, however, a direct computation is not possible because minimum and maximum dry densities are not available.

Standard penetration testing is available from the supplemental field investigations conducted in July 1979 (Reference 1, Figures 2.5-359 through 364). With these N values, an estimate of relative density was made using the relationships proposed by Tokimatsu and Seed (Reference 23). With this procedure the effective overburden pressure is calculated for each penetration test, and a coefficient C_n is obtained from curves of C_n versus effective overburden pressure. A value of $(N_1)_{60}$ is calculated using the equation $(N_1)_{60} = N\ C_n$, and relative density is obtained from a companion curve of relative density versus $(N_1)_{60}$:

Borehole SS-125, N = 50+, depth = 19', depth to groundwater = 4'
Effective overburden pressure = 4(120) + 15(58) = 1350 psf $(N_1)_{60}$ = 50+ (1.20) = 50+; $D_r > 90\%$

Borehole SS-125, N = 16, depth = 22', depth to groundwater = 4'
Effective overburden pressure = 4(120) + 18(58) = 1520 psf
$(N_1)_{60}$ = 16 (1.07) = 17; D_r = 61%

Borehole SS-125, N = 17, depth = 24', depth to groundwater = 4'
Effective overburden pressure = 4(120) + 20(58) = 1640 psf
$(N_1)_{60}$ = 17 (1.05) = 18; D_r = 63%

Borehole SS-126, N = 50+, depth = 20', depth to groundwater = 8'
Effective overburden pressure = 8(120) + 12(58) = 1660 psf
$(N_1)_{60}$ = 50+ (1.05) = 50+; D_r > 90%

Borehole SS-126, N = 35, depth = 23', depth to groundwater = 8'
Effective overburden pressure = 8(120) + 15(58) = 1830 psf
$(N_1)_{60}$ = 35 (1.02) = 36; D_r = 87%

Borehole SS-127, N = 50+, depth = 19', depth to groundwater = 15'
Effective overburden pressure = 15(120) + 4(58) = 2030 psf
$(N_1)_{60}$ = 50+ (1.00) = 50+; D_r > 90%

Borehole SS-128, N = 30, depth = 19', depth to groundwater = 17'
Effective overburden pressure = 17(120) + 2(58) = 2160 psf
$(N_1)_{60}$ = 30 (0.98) = 29; D_r = 80%

Borehole SS-129, N = 50+, depth = 17', depth to groundwater = 10'
Effective overburden pressure = 10(120) + 7(58) = 1610 psf
$(N_1)_{60}$ = 50+ (1.05) = 50+; D_r > 90%

Borehole SS-129, N = 30, depth = 19', depth to groundwater = 10'
Effective overburden pressure = 10(120) + 9(58) = 1720 psf
$(N_1)_{60}$ = 30 (1.04) = 31; D_r = 81%

Borehole SS-130, N = 29, depth = 14', depth to groundwater = 14'
Effective overburden pressure = 14(120) = 1680 psf
$(N_1)_{60}$ = 29 (1.04) = 30; D_r = 80%

Borehole SS-130, N = 46, depth = 16', depth to groundwater = 14'
Effective overburden pressure = 14(120) + 2(58) = 1800 psf
$(N_1)_{60}$ = 46 (1.02) = 47; D_r > 90%

These calculations indicate an in-place relative density of 80 to greater than 90%, however, the standard penetration test results may have been affected by oversize particles in the basal gravel.

A second approach is used to provide an estimate of relative density. Results of testing for in-situ dry density listed in Reference 1, Table 2.5-43 are summarized below:

Borehole No.	Material Class.	Moisture (%)	% Saturation	Dry Density (pcf)	Void Ratio
US-126	GP-GM	9.1	--	131.3	--
US-127	GP	7.8	85.4	135.0	.243
US-128	GP-GM	8.6	--	128.7	--
US-128A	GP-GM	10.1	--	122.6	--
US-129	GP-GM	15.3	--	111.2	--
HD-129	GP-GM	14.8	100.0	127.7	.367
US-130	GP-GM	11.0	95.9	129.0	.311
US-130	GP-GM	8.8	100.0	138.5	.213
HD-130	GP-GM	11.1	--	107.5	--

An average in-place dry density of 126 pcf is obtained, however, actual in-place density may be higher. The field samples were obtained with a 5-inch diameter Shelby Tube sampler. It was noted that some of the samples increased in volume, showing more material recovered than the sampler penetrated (Reference 1, page 2.5-84 and 85). Due to this sample disturbance, actual in-place density may be higher. A range of 130 to 135 pcf would not be unreasonable for this material.

Average specific gravity for this material is 2.60 (Reference 1, Figures 2.5-366 through 371.

As part of the direct shear testing program, material was loosely placed in a specimen box prior to consolidation. It was found that the minimum density which could be obtained was 114 pcf (Reference 1, page 2.5-86). A corresponding maximum void ratio is computed as:

$$e_{max} = \frac{SP.GR.\ (\gamma_w)}{\gamma_d} - 1$$

$$= \frac{2.60\ (62.4)}{114} - 1 = 0.42$$

Data for natural gravels tested by Seed, et. al. (Reference 12) is reproduced below:

Material	Size	Sp.Gr.	e_{max}	e_{min}
Oroville	2" minus	2.90	0.46	0.18
Livermore	2" minus	2.65	0.46	0.17

It was previously noted that the gradation for Watts Bar basal gravel is almost identical to that of the Livermore natural gravels. The specific gravity is also similar. On this basis, a minimum void ratio of 0.17 is assumed for Watts Bar basal gravel.

For an in-place dry density of 130 pcf,

$$e = \frac{2.60\ (62.4)}{130} - 1 = 0.25$$

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}}$$

$$= \frac{0.42 - 0.25}{0.42 - 0.17} = 68, \text{ say } 70\%$$

For an in-place dry density of 135 pcf,

$$e = \frac{2.60\ (62.4)}{135} - 1 = 0.20$$

$$D_r = \frac{0.42 - 0.20}{0.42 - 0.17} = 88, \text{ say } 90\%$$

It would not be unreasonable to expect actual in-place relative density to be in the range of 70 to 90%.

Based on the foregoing information, it is concluded that the average modulus reduction curve obtained by Seed, et. al. (Figure O-1-6) is appropriate for Watts Bar basal gravel.

O-1.3.0 CURVES OF REDUCTION OF SHEAR MODULUS WITH SHEAR STRAIN FOR ROCK

Purpose: To derive appropriate modulus reduction curves for Watts Bar weathered shale using general reference sources and site specific geotechnical data.

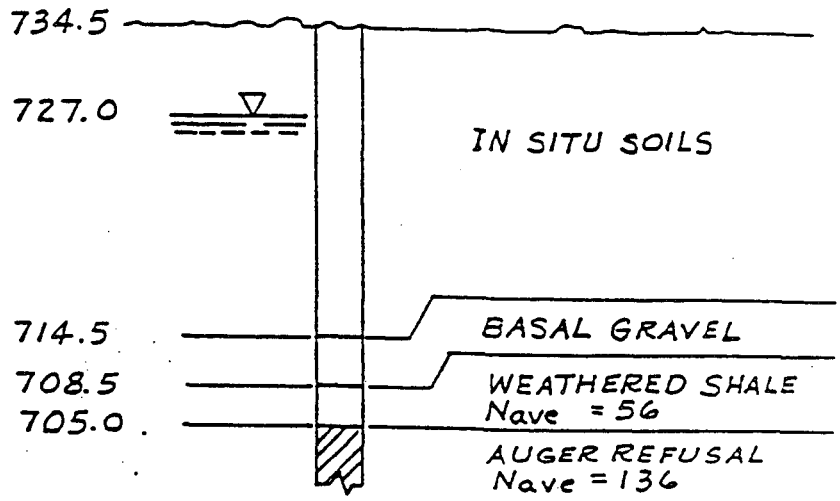
0-1.3.1 Discussion

A modulus reduction curve for rock was presented by Schnabel, Seed, and Lysmer in 1971 (Reference 20). This curve is shown in Figure 0-1-9.

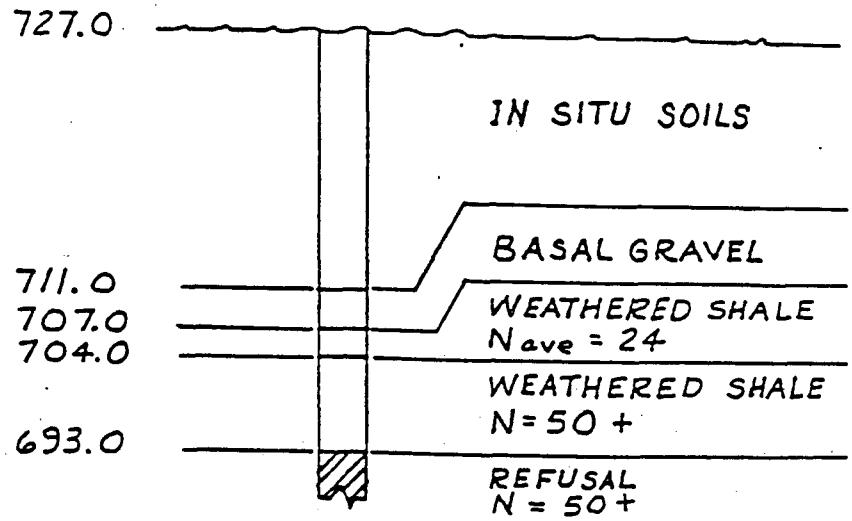
0-1.3.2 Weathered Shale

A zone of weathered shale with a thickness of up to 23 feet has been identified in the main plant area. Very little information is available regarding the degree of weathering or variation of weathering with depth.

Original subsurface investigations for the Diesel Generator Building (Reference 1, Figure 2.5-190), showed auger refusal in the shale bedrock at approximate elevation 705, where standard penetration testing gave an average count of 136 blows per foot. A generalized profile is shown below.



Subsequent subsurface investigations (Reference 1, Figures 2.5-358 through 364) show a much deeper zone of weathered shale. Blow counts of 50+ were obtained in all but the upper 2 or 3 feet. A generalized profile is shown below:



The available data suggests the presence of a ±3 foot thick zone of softer shale overlying what can be considered competent bedrock. Reasonably high blow counts were obtained even in the upper zone.

The modulus reduction curve for rock presented by Schnabel, et. al. (Figure 0-1-9) is considered appropriate for the zone of material identified as weathered shale.

0-1.4.0 DAMPING RATIOS FOR SANDS, GRAVELS, AND CLAYS

Purpose: To derive appropriate damping ratio curves for Watts Bar fine-grained soils and gravels using general reference sources and site specific geotechnical data.

0-1.4.1 Discussion

Basic damping ratio curves for sands and soft clays were presented by Seed and Idriss in 1970 (Reference 11). These curves are shown in Figure 0.1-10.

Subsequent investigations by Seed, Wong, Idriss, and Tokimatsu in 1984 (Reference 12) confirmed use of the sand curves for natural gravels and crushed stone. No modification of the curve for soft clays was attempted. Typical cyclic shear strain for earthquake response would be in the 10^{-3} to 10^{-2} percent range, and here there is very little difference in damping ratios between soft clays and sand. If higher shear strains do occur, use of the clay curve will be conservative.

0-1.4.2 Class A Backfill and Cohesive In-Situ Soils (CL, ML)

Watts Bar Class A backfill and cohesive in-situ soils are described in Sections 0-1.1.2 and 0-1.1.3 of this Attachment. Resonant column test data for undisturbed and remolded samples of CL material obtained from the Low Level Radwaste Borrow Area is presented in Reference 7. Values of damping ratios from these tests vary somewhat from sample to sample and with confining pressure, but are generally consistent with Figure 0-1-10 within the range of shear strain tested. The damping ratio curve for clay proposed by Seed and Idriss (Figure 0-1-10) is considered appropriate for these materials.

0-1.4.3 Non-Plastic In-Situ Soils (SM, ML) and Gravel

Watts Bar non-plastic in-situ soils, crushed stone backfill, and basal gravel are described in Sections 0-1.1.4, 0-1.2.2, and 0-1.2.3 of this Attachment. Resonant column test data for undisturbed and remolded samples of non-plastic SM and ML material obtained from the Low Level Radwaste Borrow Area is presented in Reference 7. Values of damping ratios from these tests are limited in number, and provide no apparent basis for deviation from Figure 0-1.1-10. The damping ratio curve for sand proposed by Seed and Idriss (Figure 0-1-10) is considered appropriate for these materials.

0-1.5.0 DAMPING RATIOS FOR ROCK

Purpose: To derive appropriate damping ratio curves for Watts Bar weathered shale using general reference sources and site specific geotechnical data.

0-1.5.1 Discussion

A damping ratio curve for rock was presented by Schnabel, Seed, and Lysmer in 1971 (Reference 20). This curve is shown in Figure 0-1-11.

0-1.5.2 Weathered Shale

Watts Bar weathered shale is described in Section 0-1.3.2 of this attachment. The damping ratio curve for rock presented by Schnabel, et. al. (Figure 0-1-11) is considered appropriate for this material.

0-1.6.0 EVALUATION OF RESONANT COLUMN TEST DATA FOR REMOLDED CL

Purpose: To evaluate resonant column test data for remolded CL material for use in prediction of shear modulus for Class A backfill.

Resonant column test data for samples of remolded CL material obtained from the Low Level Radwaste Borrow Area is presented in Reference 7, pages 129 through 133. Test results are given for shear strain amplitudes ranging between 10^{-3} and 10^{-2} percent. In evaluating the data, the zero intercept was assumed to represent the shear modulus at 1×10^{-3} percent strain. In order to obtain G_{max}, these values were extrapolated to a strain of 1×10^{-4} percent using the following relationship (see Figure 0-1-4 of this Attachment):

$$\frac{\text{Shear Modulus at Shear Strain 10-3 percent}}{\text{Shear Modulus at Shear Strain } 10^{-4} \text{ percent}} = 0.81$$

Shear Modulus in ksf at 10^{-3} percent shear strain, remolded CL (Reference 7, pages 129 through 133).

Report Page	Moisture Content	Confining Pressure, σ_3 1.0 ksf	2.0 ksf	4.0 ksf	8.0 ksf
129	11.6%	5470	7060	9220	10,660
130	12.7%	3530	4100	5040	6,050
131	13.7%	4030	4540	5400	6,120
132	16.2%	3740	4250	4540	4,900
133	18.3%	3240	3530	3960	4,750

Convert to 10^{-4}% shear strain:

G at 10^{-3}% strain (5470)/0.81
= G at 10^{-4}% strain = 6753, say 6750

Shear Modulus in ksf at 10^{-4}% shear strain, remolded CL.

Moisture Content	Confining Pressure, σ_3 1.0 ksf	2.0 ksf	4.0 ksf	8.0 ksf
11.6%	6750	8720	11,380	13,160
12.7%	4360	5060	6220	7,470
13.7%	4980	5600	6670	7,560
16.2%	4620	5250	5600	6,050
18.3%	4000	4360	4980	5,860

It is noted that the tested samples were compacted to 95% of Standard Proctor density.

0-1.7.0 EVALUATION OF STANDARD PENETRATION TEST RESULTS FOR CL BACKFILL

Purpose: To evaluate results of standard penetration testing to

provide an estimate of shear modulus for lean clay backfill.

Standard penetration testing is available for lean clay backfill which was placed along the Emergency Raw Cooling Water Pipeline and Conduits (Reference 1, Figures 2.5-275, 277, 280, 283, 286, 288, 289, 290, 291, 292, 328, 329, 330, 359, and 362). This information is summarized below:

Borehole No.	Depth (ft)	Moisture (%)	N (blows/ft)
SS-172	3	19.4	17
SS-172	5	20.9	14
SS-172	6.5	23.3	8
SS-172	9	21.4	15
SS-174	4.5	19.4	18
SS-174	8.5	19.1	47
SS-177	3	16.7	50+
SS-131	4	19.5	25
SS-131	6	19.3	21
SS-131	8	20.7	18
SS-133	3	15.7	23
SS-133	5	16.6	18
SS-133	7	18.9	16
SS-133	9	19.7	12
SS-133	11	22.9	12
SS-133	13	21.7	11
SS-133	15	22.5	9
SS-133	17	23.6	2
SS-133	19	32.9	4
SS-134A	2	13.4	13
SS-134A	4	16.1	25
SS-134A	6	15.6	17
SS-135	2	--	7
SS-135	4	19.4	13
SS-135	5.5	19.3	13
SS-135A	2.5	16.1	19
SS-135A	4.5	16.6	20
SS-65B	4	12.5	20
SS-136	2	19.2	19
SS-136	3.5	22.5	6
SS-166	2	20.5	12
SS-167	4	13.4	12
SS-167	6	21.8	13
SS-168	4	16.5	11
SS-168	6.5	16.9	12
SS-168	8	17.3	12
SS-125	4	--	9
SS-125	7	26.3	11
SS-125	9	25.1	9
SS-128	9	25.4	4
SS-128	11	25.8	3
SS-128	12	27.2	2

It is reported that this material was placed at a minimum of 90% of Standard Proctor density, corresponding to specifications for Class B backfill. A plot of standard penetration resistance versus moisture content is shown in Figure 0-1-12.

Blow counts represented by the dashed line in Figure 0-1-12 are converted to unconfined compressive strength (q_u) using Figure 0-1-13 (Reference 24), and to undrained shear strength (S_u) using the relationship $S_u \simeq 0.5\ q_u$. In Figure 0-1-13, the Terzaghi and Peck solid line plot is most widely used, and is adopted here as being representative of the range of possible values.

Moisture (%)	N (blows/ft)	qu (ksf)	$Su \simeq qu/2$ (ksf)
12	17	4.5	2.2
18	17	4.5	2.2
19	16	4.2	2.1
20	15	4.0	2.0
21	14	3.7	1.8

Relationships between small strain shear modulus and undrained shear strength for clays were presented by Egan and Ebeling in 1985 (Reference 9). These relationships reflected theoretical considerations and empirical data indicating that the ratio G_{max}/S_u for clays is not a constant but rather decreases with increasing undrained shear strength. The data base included normalized soil parameter concepts, and data available from a variety of clay soils for in-situ and laboratory test conditions. Figure 0-1-14 shows suggested relationships for clay materials used in the following calculation:

Moisture (%)	S_u (ksf)	$Gmax/S_u$	G_{max} (ksf)
12	2.2	1290	2840
18	2.2	1290	2840
19	2.1	1305	2740
20	2.0	1325	2650
21	1.8	1340	2410

0-1.8.0 EVALUATION OF GEOPHYSICAL SURVEY DATA FOR IN-SITU SOILS

Purpose: To evaluate Watts Bar geophysical survey data for in-situ soils to establish representative values for shear wave velocity.

O-1.8.1 Discussion

In-situ soils are described in Sections O-1.1.3 and O-1.1.4 of this Attachment. Two zones are of interest; cohesive soils (CL, ML) which are found in the upper portion of the soil profile, and weak, non-plastic soils (SM, ML) which are found generally in the lower 2 to 5 feet of the profile. These non-plastic soils typically have N values of less than 10 blows/foot. Most geophysical survey data was obtained by downhole and refraction methods, and does not give sufficient definition to distinguish between these two soil zones. For the Low Level Radwaste Borrow Area, crosshole testing was conducted, and will be used to differentiate between cohesive and non-plastic soils.

O-1.8.2 Diesel Generator Building

Results of geophysical surveys for the Diesel Generator Building are shown in Reference 1, Figure 2.5-233. Borehole logs are shown in Figure 2.5-190, and laboratory data in Table 2.5-6.

The thickness of in-situ soils at this location is approximately 22 feet.

Borehole No.	Elev. of Ground Surface	Elev. top of Basal Gravel	Thickness of In-Situ Soils
SS-25	734.6	712.7	21.9
SS-26	735.0	713.1	21.9
SS-27	734.1	712.4	21.7
SS-28	734.4	712.7	21.7

Shear wave velocities obtained for in-situ soils are summarized below:

Elevation Zone (feet)	Type of Measurement	Average V_s (fps)
734-704	Downhole	1040*
734-728	Refraction	600
728-715	Refraction	1380

* - May include basal gravel

Arithmetic average for refraction survey = 990 fps
Thickness weighted average for refraction survey = 1130 fps

From Reference 1, Table 2.5-6:

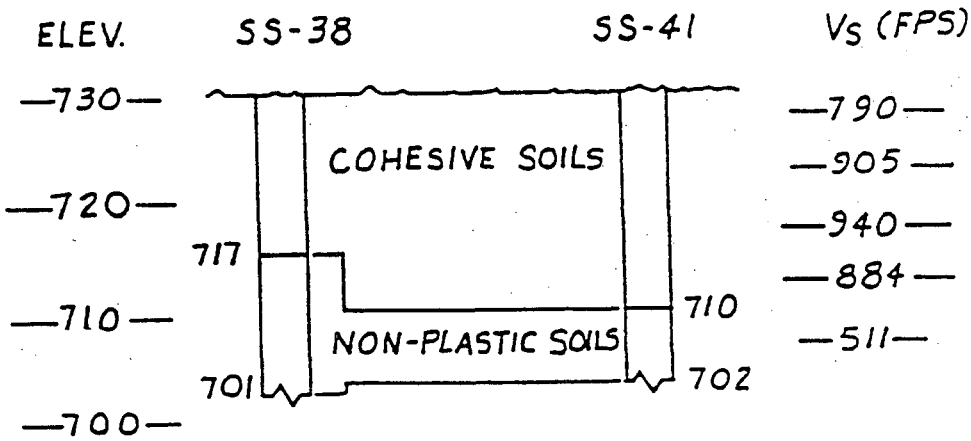
Average in-situ dry density	= 90.0 pcf
Average in-situ moisture content	= 31.2%
Average in-situ moist density	= 90.0 (1.312)
	= 118 pcf

0-1.8.3 Low Level Radwaste Borrow Area

Results of geophysical surveys for the Low Level Radwaste Borrow Area are presented in Reference 6. Results of the laboratory testing program are presented in Reference 7.

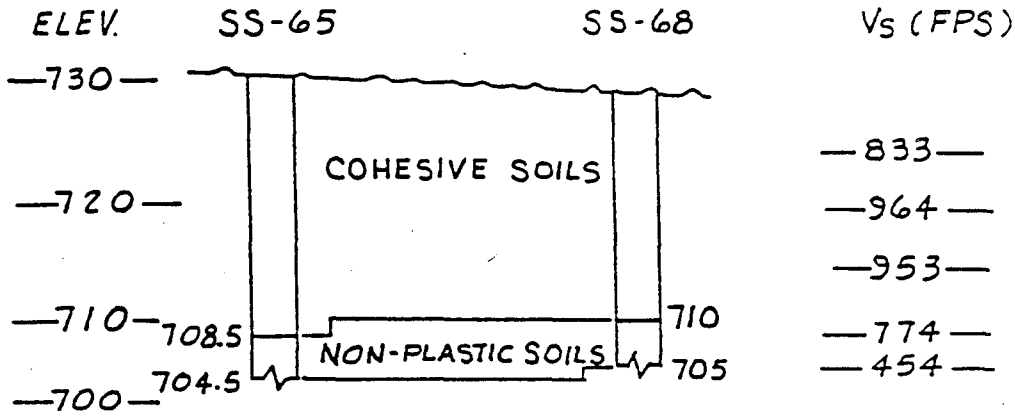
Geophysical data is presented in Reference 6 for Arrays SD-1 (see Boreholes SS-38 and 41), SD-2 (see Boreholes SS-65 and 68), and SD-3 (see Boreholes SS-11 and 14). Crosshole data is available at 5 foot depth intervals (Tables 1, 2, and 3). Generalized subsurface profiles with measured shear wave velocities are shown below. The soil profile is subdivided into an upper zone of cohesive soils, and the underlying zone of weak, non-plastic sands and silts (N values less than 10 blows per foot).

ARRAY SD-1



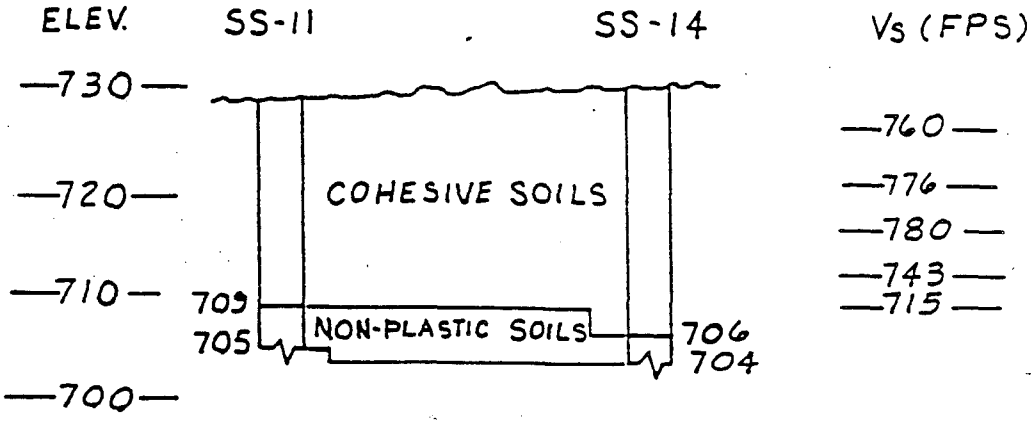
Average V_s in cohesive soils	= 880 fps
V_s in non-plastic soils	= 511 fps

ARRAY SD-2



Average V_s in cohesive soils = 917 fps
Average V_s in non-plastic soils = 614 fps

ARRAY SD-3



Average V_s in cohesive soils = 765 fps
V_s in non-plastic soils = 715 fps

Overall average V_s in cohesive soils = 854 fps
Overall average V_s in non-plastic soils = 613 fps

For cohesive soils (Reference 7, Table 2):

Average in-situ dry density = 96.8 pcf
Average in-situ moisture content = 25.1%
Average in-situ moist density 96.8 (1.251)
= 121 pcf

For non-plastic soils (Reference 7, Table 2):

Average in-situ dry density	= 88.1 pcf
Average in-situ moisture content	= 32.4%
Average in-situ moist density	88.1 (1.251)
	= 117 pcf

O-1.8.4 Intake Channel

Results of geophysical surveys for the intake channel are shown in Reference 1, Figures 2.5-228 through 232, and laboratory data in Tables 2.5-8 and 9.

Results of geophysical surveys are summarized below:

Elevation Zone (feet)	Type of Measurement	Average V_s (fps)
650-697	Downhole	940
640-697	Refraction	970

From Reference 1, Tables 2.5-8 and 9:

Average in-situ dry density	= 95.7 pcf
Average in-situ moisture content	= 25.5%
Average in-situ moist density	95.7 (1.255)
	= 120 pcf

O-1.8.5 Class IE Conduits and Essential Raw Cooling Water Piping

Results of geophysical surveys for Class IE Conduits and Essential Raw Cooling Water Piping are shown in Reference 1, Table 2.5-17. Applicable borehole logs are shown in Figure 2.5-196 through 202. Applicable laboratory data is shown in Tables 2.5-10, 11, and 24.

Boreholes encountering primarily in-situ fine-grained soils within the zone of measured shear wave velocity were selected from Table 2.5-17, and are summarized below:

Borehole No.	Appx. Depth Zone (feet)	Average V_s (fps)
SS-49	0 to 37	1169
SS-55	0 to 45	1172
SS-56	0 to 37	933
SS-63	0 to 35	1357
SS-75	0 to 30	1502
SS-84	0 to 34	1756
SS-88	0 to 37	1100
SS-101	0 to 24	1637
SS-107	0 to 26	1498
Average	0 to 34	1347

From Reference 1, Tables 2.5-10, 11, and 24 for fine-grained soils:

Average in-situ dry density	= 100.9 pcf
Average in-situ moisture content	= 22.4%
Average in-situ moist density	100.9 (1.224)
	= 124 pcf

It should be noted that in this area, as compared with other areas where geophysical surveys were conducted, blow counts from standard penetration were higher, moist density was slightly higher, and occasional zones of gravel were present in the profile. These factors account for the higher average shear wave velocity.

0-1.9.0 EVALUATION OF RESONANT COLUMN TEST DATA FOR COHESIVE IN-SITU SOILS

Purpose: To evaluate resonant column test data for undisturbed samples of cohesive in-situ soils for use in prediction of shear modulus.

Resonant column test data for undisturbed samples of CL material obtained from the Low Level Radwaste Borrow Area is presented in Reference 7, pages 96 and 97. Test results are given for shear strain amplitudes ranging between 10^{-3} and 10^{-2} percent. In evaluating the data, the zero intercept was assumed to represent the shear modulus at 10^{-3} percent strain. In order to obtain G_{max}, these values were extrapolated to a strain of 1×10^{-4} percent, using the following relationship for an assumed undrained shear strength of 1.2 ksf (see Figure 0-1-5 of this Attachment):

$$\frac{\text{Shear Modulus at Shear Strain } 10^{-3} \text{ percent}}{\text{Shear Modulus at Shear Strain } 10^{-4} \text{ percent}} = 0.78$$

Shear Modulus in ksf at 10^{-3}% shear strain, undisturbed CL (Reference 7, pages 96 and 97).

Report Page	Moisture Content	Confining Pressure, σ_3 1.0 ksf	2.0 ksf	4.0 ksf	8.0 ksf
96	10.8%	910	1610	3080	6,830
97	29.3%	620	1040	1610	2,880

Convert to 10^{-4}% shear strain:

G at 10^{-3}% strain (910)/0.78
= G at 10^{-4}% strain = 1167, say 1170

Shear Modulus in ksf at 10^{-4}% shear strain, undisturbed CL.

Moisture Content	Confining Pressure, σ_3 1.0 ksf	2.0 ksf	4.0 ksf	8.0 ksf
10.8%	1170	2060	3950	8760
29.3%	790	1330	2060	3690

O-1.10.0 SECTION OF $(K_2)_{max}$ VALUE FOR CRUSHED STONE BACKFILL

Purpose: To select a value of $(K_2)_{max}$ for Watts Bar crushed stone backfill using general reference sources and site specific geotechnical data.

Values of $(K_2)_{max}$ for gravels were presented by Seed, Wong, Idriss, and Tokimatsu in 1984 (Reference 12). This data was based on a comprehensive series of laboratory tests performed on 12-inch diameter samples of natural gravels and crushed stone. The Pyramid material tested by Seed, et. al is selected for comparison on the basis of similarities in gradation and relative density (see Section O-1.2.2 of this Attachment), and because both materials were produced by crushing. Values of K_2 obtained for the Pyramid material are shown in Figure O-1-15.

$(K_2)_{max}$ for D_r 95% $\simeq$ 129
$(K_2)_{max}$ for D_r 80% $\simeq$ 92

$(K_2)_{max}$ for $D_r \simeq$ 81%
= 92 + (129-92/15) = 94, say 100

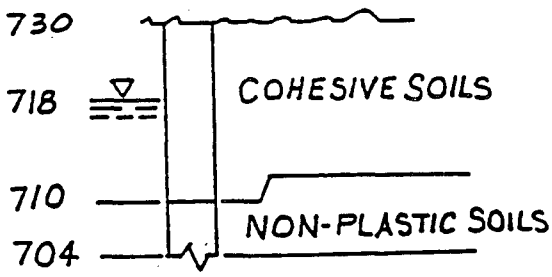
For use in the equation $G = 1000\ k_2\ (\bar{\sigma}_m)^{1/2}$

O-1.11.0 SELECTION OF $(K_2)_{MAX}$ VALUE FOR NON-PLASTIC IN-SITU SOILS (SM, ML)

Purpose: To select a value of $(k_2)_{max}$ for Watts Bar in-situ non-plastic soils using general reference sources and site specific geotechnical data.

Values of $(k_2)_{max}$ for sands were presented by Seed and Idriss in 1970 (Reference 11). These values were based primarily on data obtained from laboratory testing, supplemented by field shear wave velocity measurements at low strain levels.

An average shear wave velocity of 613 fps was obtained for the zone of weak, non-plastic soils present at Watts Bar (see Sections O-1.8.1 and O-1.8.3 of this Attachment). Using a typical profile for the Low Level Radwaste Borrow Area and a moist density of 120 pcf:



$\bar{\sigma}_v = 12(120) + 11(120 - 62.4)$
$= 2070$ PSF.

$\bar{\sigma}_n = \bar{\sigma}_v (1 - \sin\phi) = 0.5\,\bar{\sigma}_v$

$\bar{\sigma}_m = \dfrac{\bar{\sigma}_v + 2\bar{\sigma}_n}{3} = 1380$ PSF

FOR IN SITU SOILS, USE $\phi = 32°$ (REF. 1, TABLE 2.5-12.

$G_{max} = \rho V_s^2 = (120/322)\,613^2$
$= 1.40 \times 10^6$ PSF

$G = 1000\,K_2 (\bar{\sigma}_m)^{1/2}$

$(K_2)_{max} = G_{max}/1000\,(\bar{\sigma}_m)^{1/2}$
$= 1.40 \times 10^6 / 37.1 \times 10^3$
$= 38$

By the method of Hardin and Drnevich (Reference 11):

$$G_{max} = 14{,}760\,\frac{(2.973-e)^2}{1+e}\,(OCR)^a\,(\bar{\sigma}_m)1/2$$

From soil investigations for the Diesel Generator Building (Reference 1, Table 2.5-6):

Borehole No.	Material Type	Depth (feet)	Void Ratio
SS-25	SM (NP)	20	0.868
SS-25	SP-SM (NP)	22	0.870
SS-26	SM (NP)	20	0.884

Average void ratio = 0.874

From soil investigations for the Low Level Radwaste Borrow Area (Reference 7, Table 2):

Borehole No.	Material Type	Depth (feet)	Void Ratio
SS-5	SM (NP)	21	0.757
SS-12	SM (NP)	23	1.162
SS-13	SM (NP)	22	0.741
SS-13A	SM (NP)	20	0.802
SS-20	ML (NP)	20	0.943
SS-20	SM (NP)	23	1.004
SS-29	SP-SM (NP)	26	0.999
SS-34A	SM (NP)	21	1.154
SS-39A	SM (NP)	26	0.983
SS-39A	SM (NP)	31	0.794
SS-44A	SM (NP)	26	0.942
SS-44A	SM (NP)	26	0.931
SS-55	SM (NP)	24	0.833
SS-62	SP-SM (NP)	24	0.628
SS-71A	SM (NP)	21	0.913

Average void ratio = 0.906

Use: $\bar{e} = 0.906$
$\bar{\sigma}_m = 1380$ psf
OCR = 1.0 (normally consolidated)

$$G_{max} = 14{,}760 \; \frac{(2.973 - .906)^2}{1 + .906} \; (1) \; (1380)^{1/2}$$

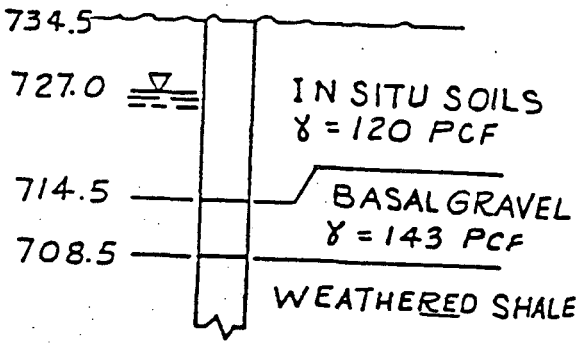
$= 1.22 \times 10^6$ psf versus 1.40×10^6 psf from shear wave velocity = 614 fps

The close agreement between G_{max} by the two methods supports the use of $(K_2)_{max} = 38$.

O-1.12.0 SELECTION OF $(K_2)_{MAX}$ VALUE FOR BASAL GRAVEL

Purpose: To select a value of $(k_2)_{max}$ for Watts Bar basal gravel using general reference sources and site specific geotechnical data.

Geophysical refraction surveys for the Diesel Generator Building (Reference 1, Figure 2.5-233) gave an average shear wave velocity of 1660 fps in an approximate depth zone of 19 to 39 feet. Saturated unit weight of basal gravel is taken as 143 pcf (Reference 1, Table 2.5-44).



$$\bar{\sigma}_v = 7.5(120) + 12.5(120 - 62.4) + 3.0(143 - 62.4) = 1862 \text{ PSF}$$

$$\bar{\sigma}_n = \bar{\sigma}_v (1 - \sin\phi) = 0.4\,\bar{\sigma}_v$$

$$\bar{\sigma}_m = \frac{\bar{\sigma}_v + 2\bar{\sigma}_n}{3} = 1117 \text{ PSF}$$

$$G_{max} = \rho V_s^2 = (143/32.2)\,1660^2 = 12.2 \times 10^6 \text{ PSF}$$

FOR BASAL GRAVEL USE $\phi = 39°$
(REF. 1, TABLE 2.5-44)

$$G = 1000\,K_2\,(\bar{\sigma}_m)^{1/2}$$

$$(K_2)_{max} = G_{max} / 1000\,(\bar{\sigma}_m)^{1/2} = 12.2 \times 10^6 \text{ PSF} / 33.4 \times 10^3 = 365$$

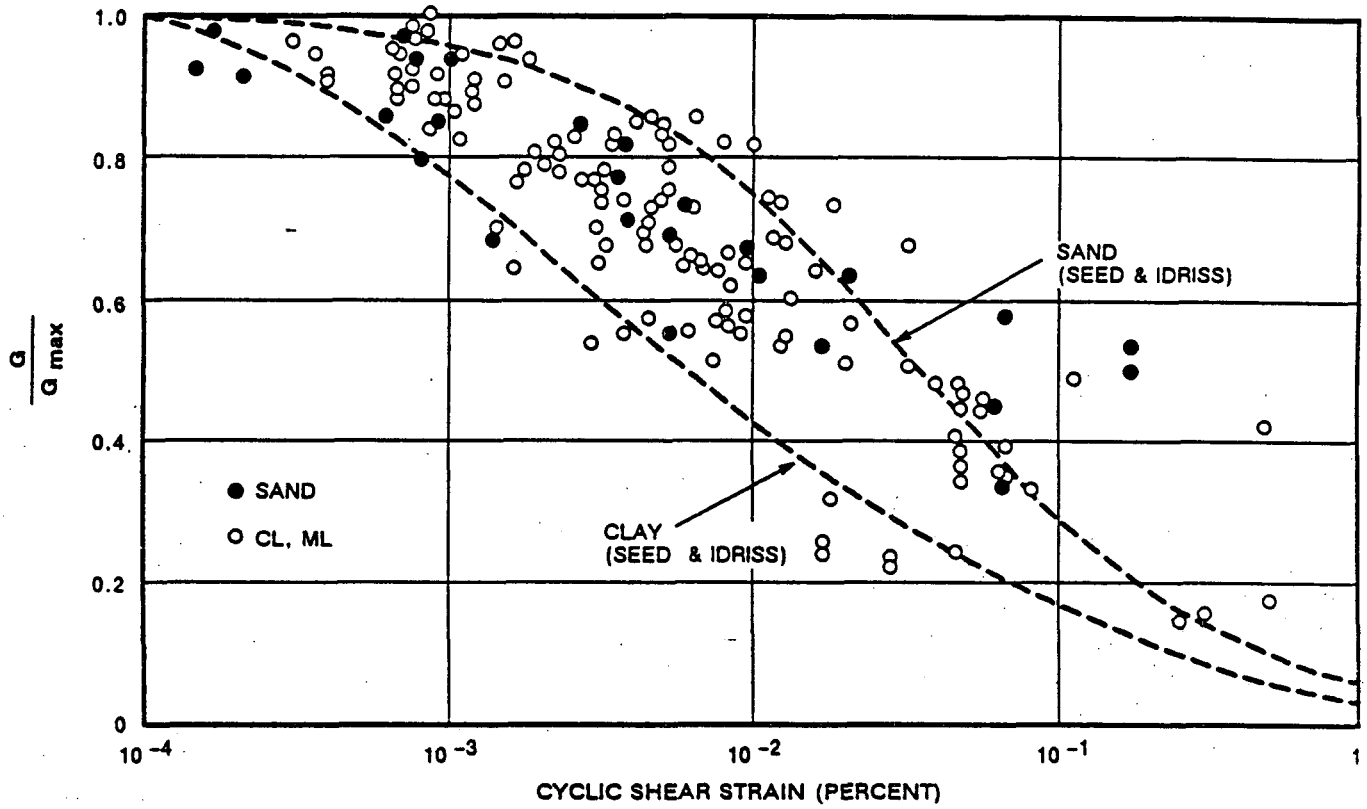


Figure 0-1-1
MODULUS REDUCTION WITH STRAIN: SANDS AND LOW PLASTICITY SILTS AND CLAYS

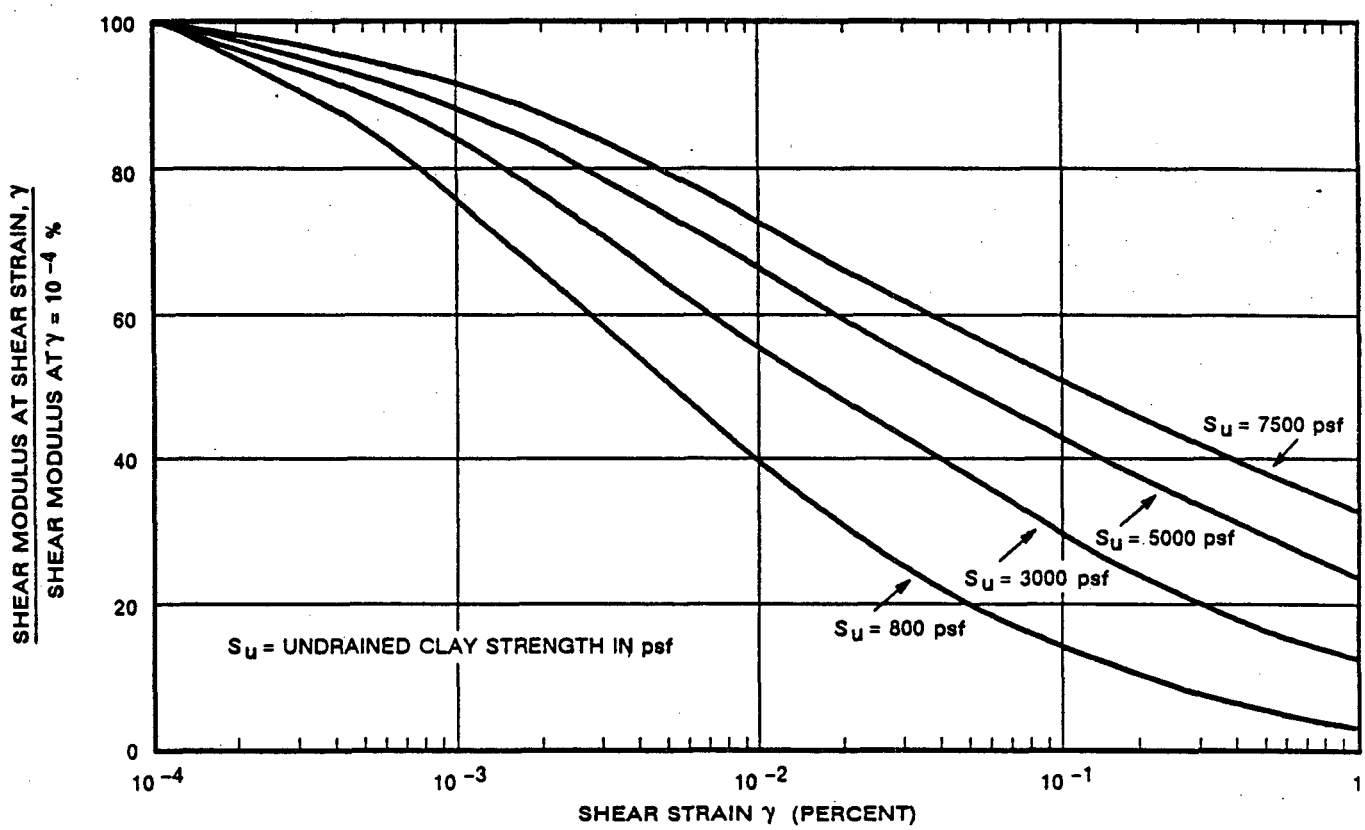


Figure 0-1-2
TYPICAL REDUCTION OF SHEAR MODULUS WITH SHEAR STRAIN IN CLAYS

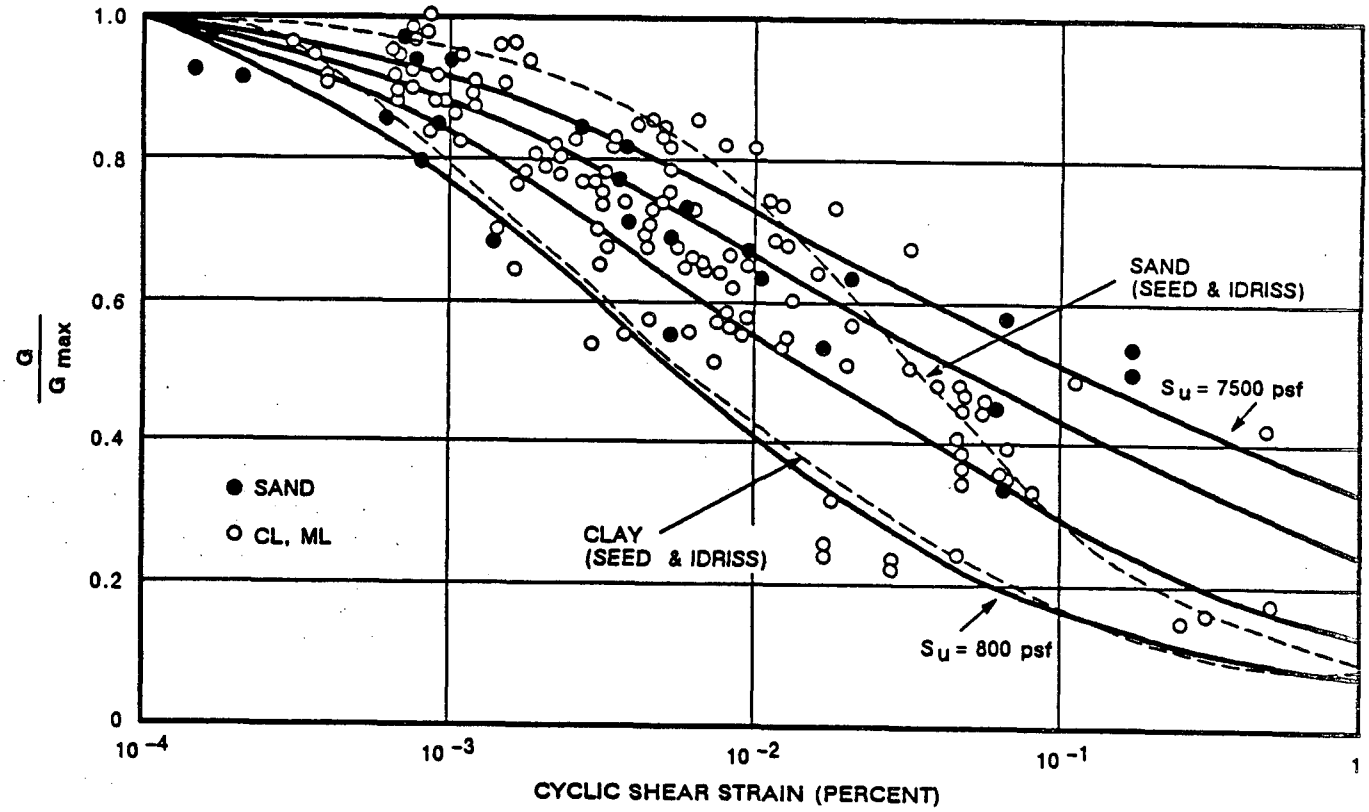


Figure 0-1-3
MODULUS REDUCTION WITH STRAIN

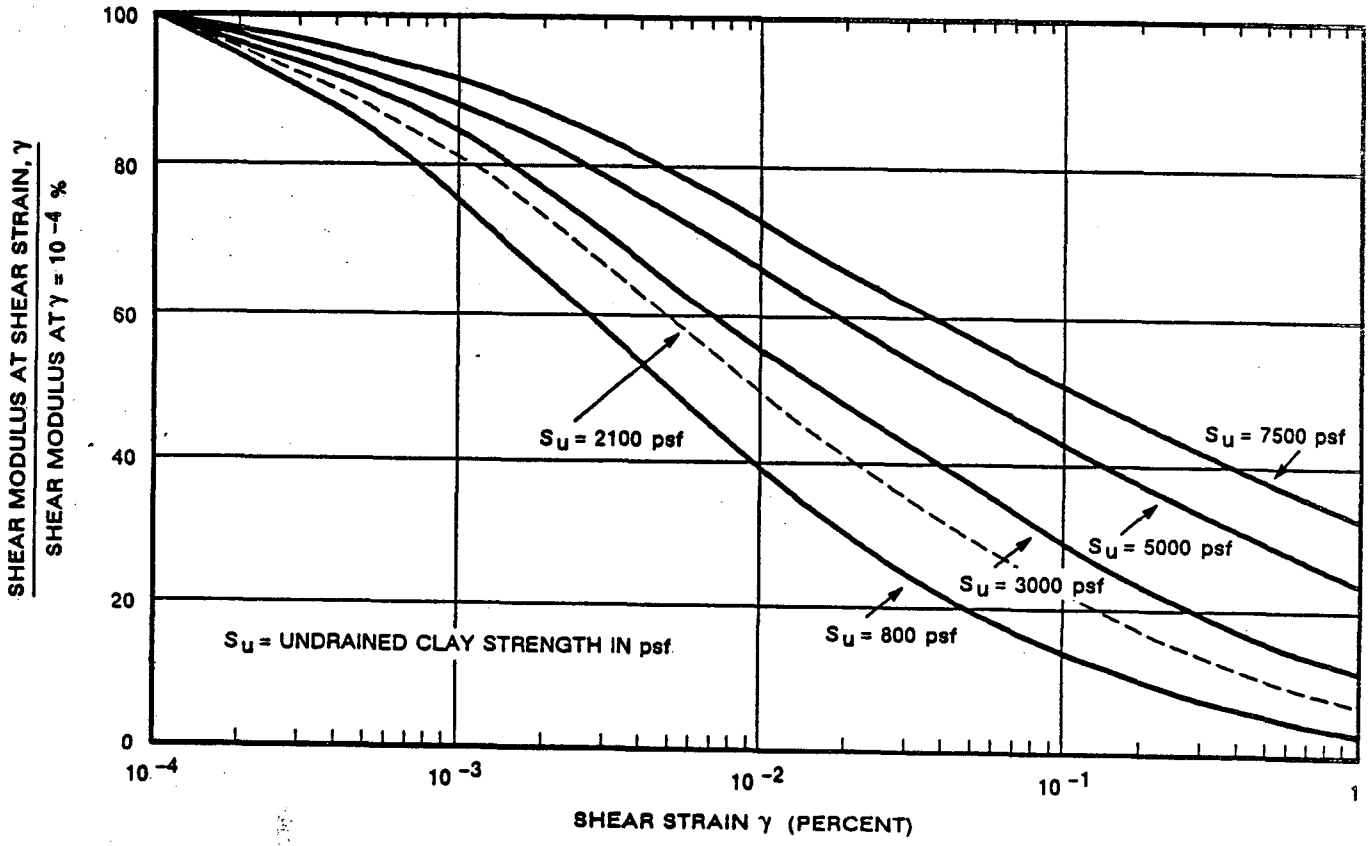


Figure 0-1-4
TYPICAL REDUCTION OF SHEAR MODULUS WITH SHEAR STRAIN IN CLAYS

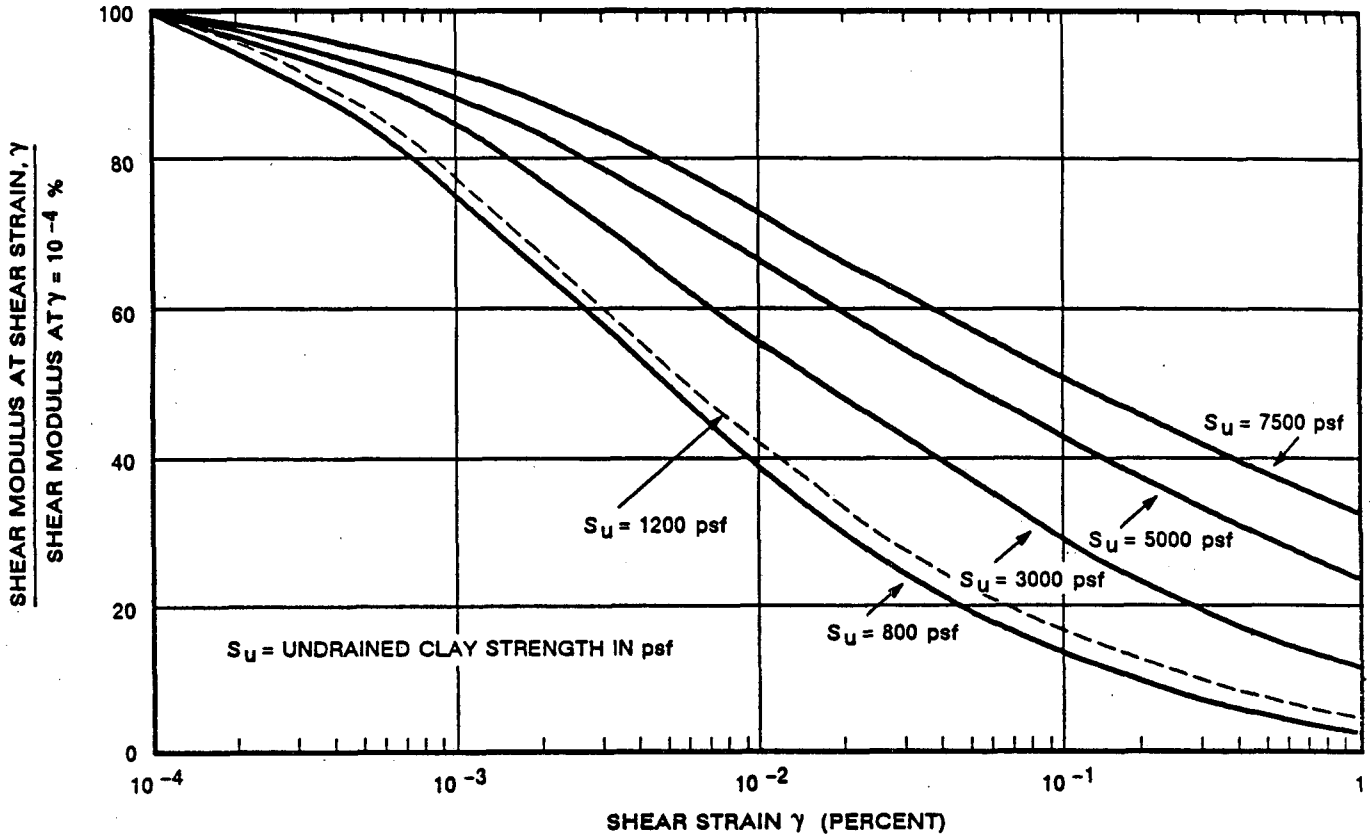


Figure 0-1-5
TYPICAL REDUCTION OF SHEAR MODULUS WITH SHEAR STRAIN IN CLAYS

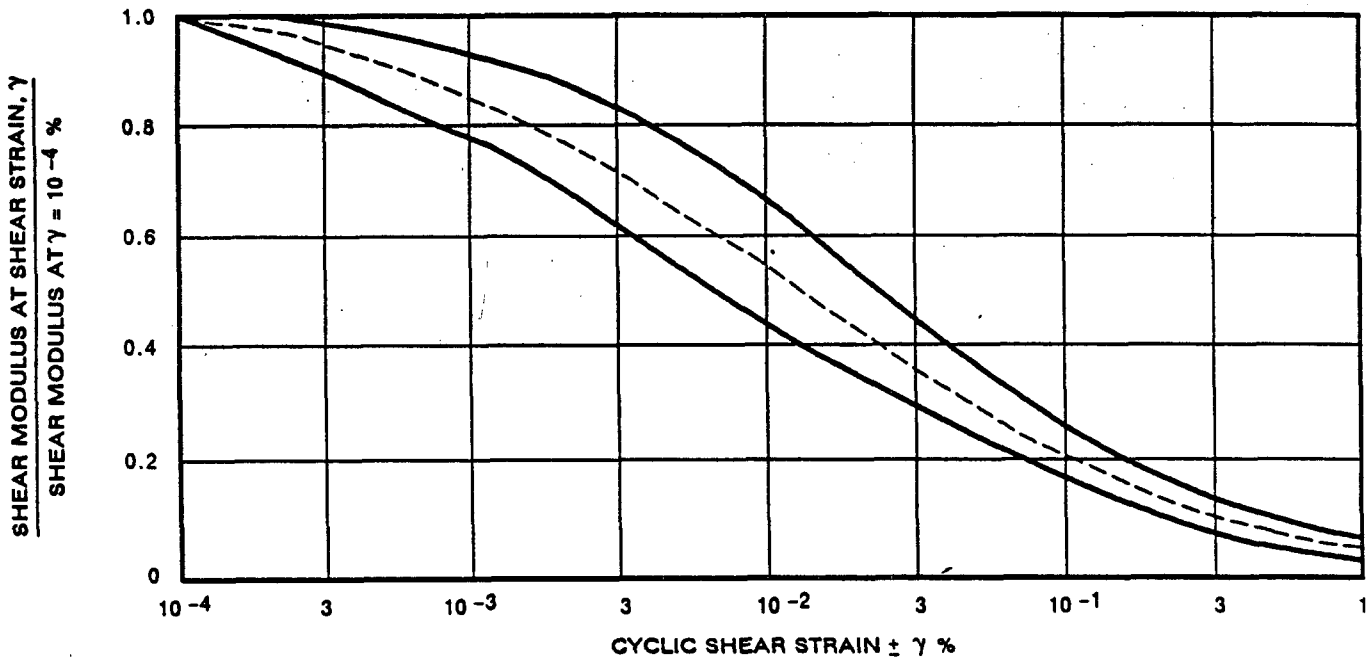


Figure 0-1-6
VARIATION OF SHEAR MODULUS WITH SHEAR STRAIN FOR GRAVELLY SOILS

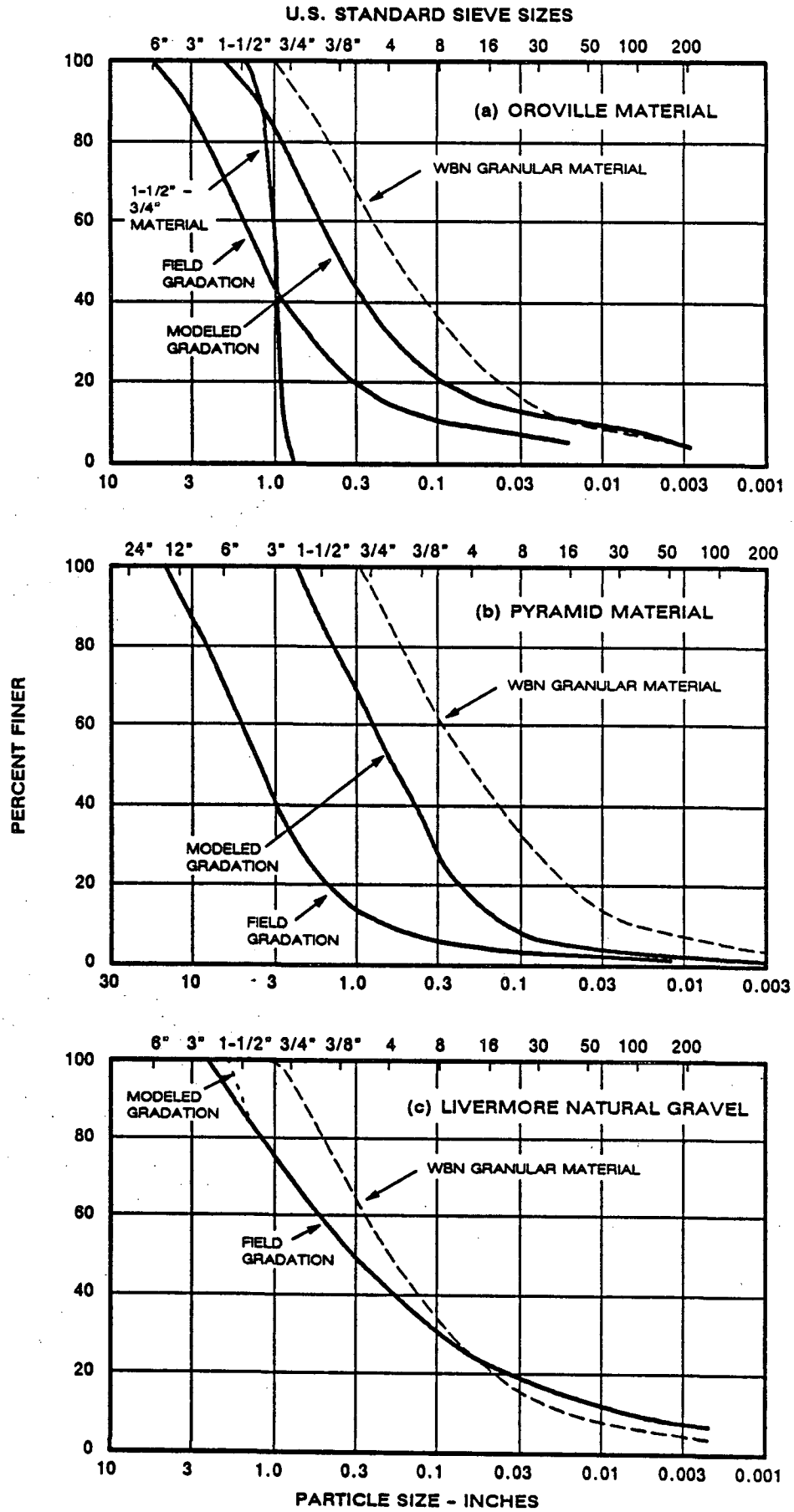


Figure 0-1-7
GRAIN SIZE DISTRIBUTION CURVES FOR FIELD AND MODELED GRADATIONS

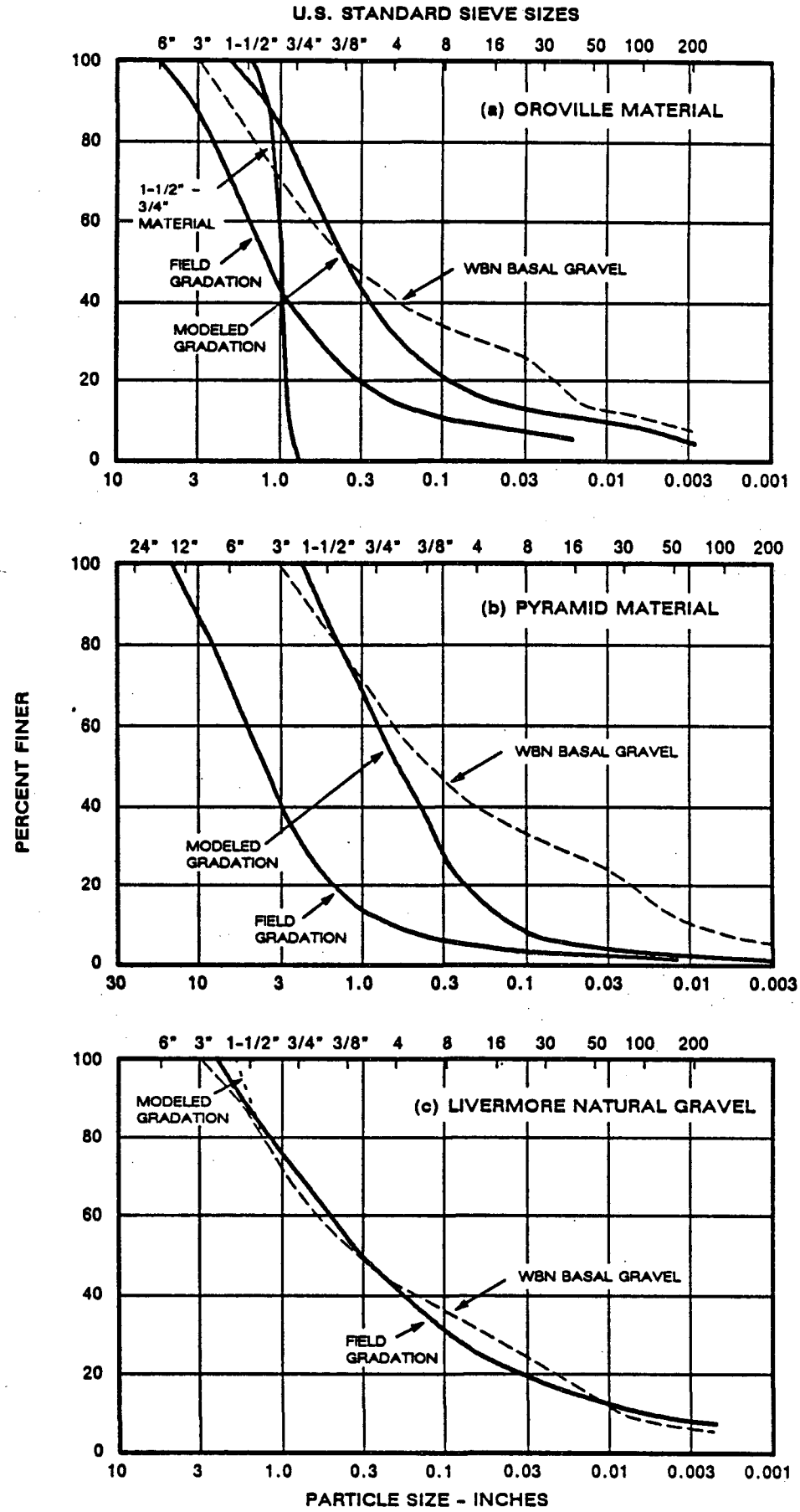


Figure 0-1-8
GRAIN SIZE DISTRIBUTION CURVES FOR FIELD AND MODELED GRADATIONS

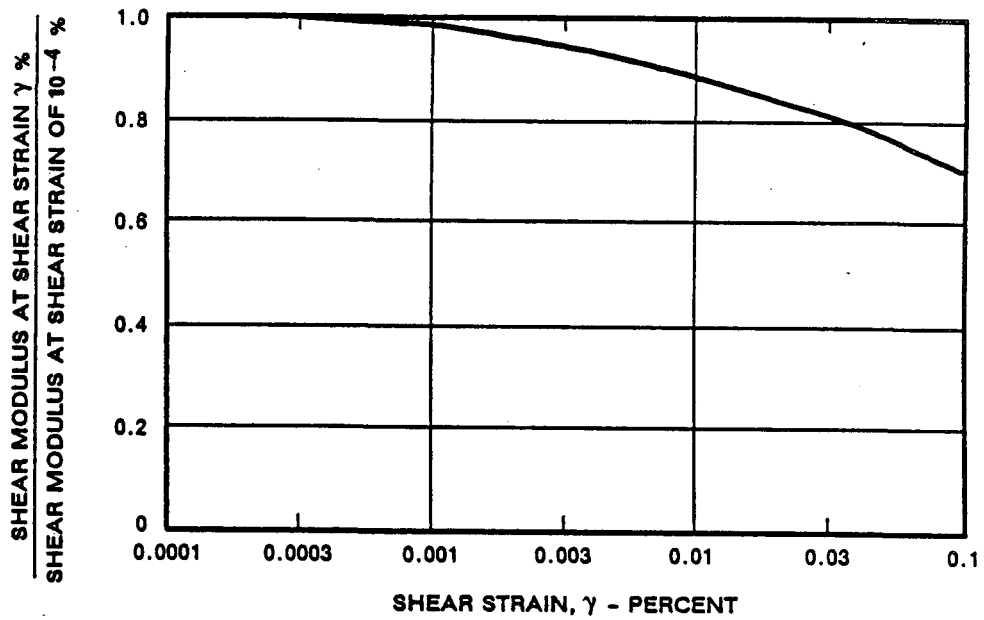


Figure 0-1-9
VARIATION OF SHEAR MODULUS WITH SHEAR STRAIN FOR ROCK

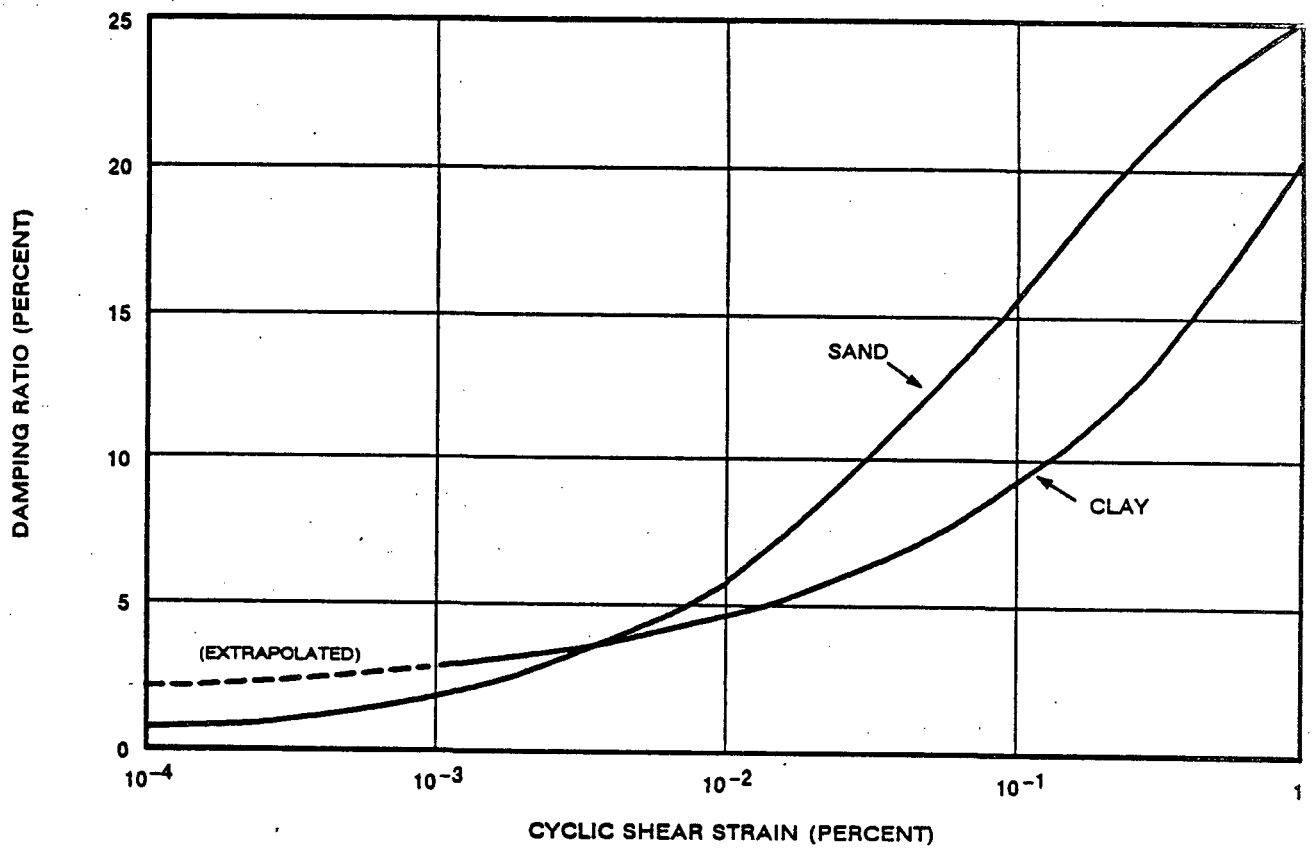


Figure 0-1-10
DAMPING RATIOS FOR SANDS AND CLAY

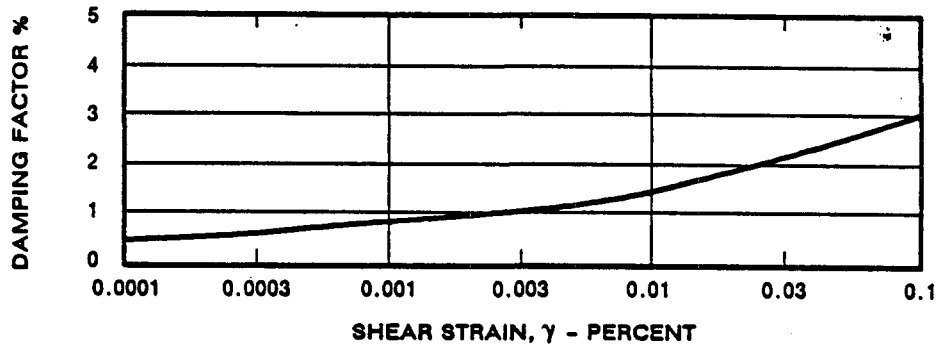


Figure 0-1-11
DAMPING RATIOS FOR ROCK

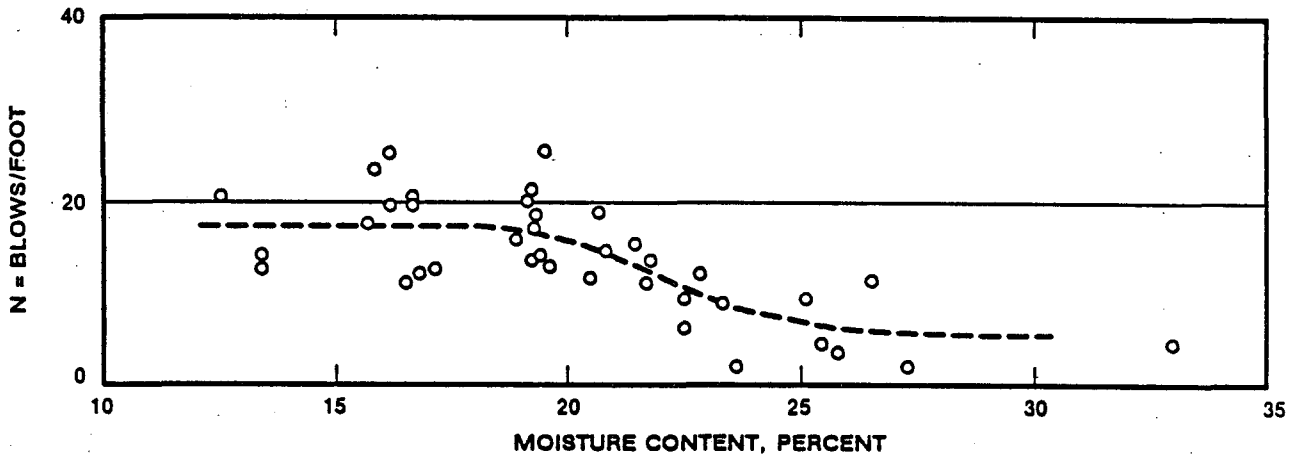


Figure 0-1-12
STANDARD PENETRATION RESISTANCE vs
MOISTURE CONTENT IN LEAN CLAY BACKFILL

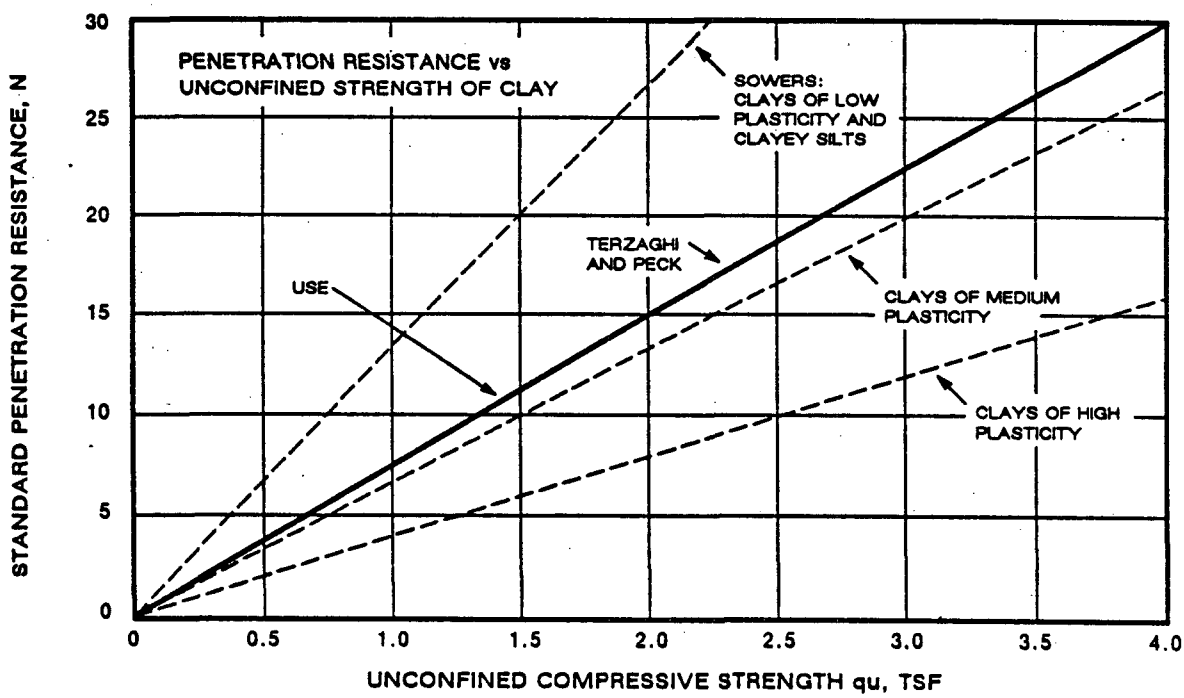


Figure 0-1-13
CORRELATION OF STANDARD PENETRATION RESISTANCE

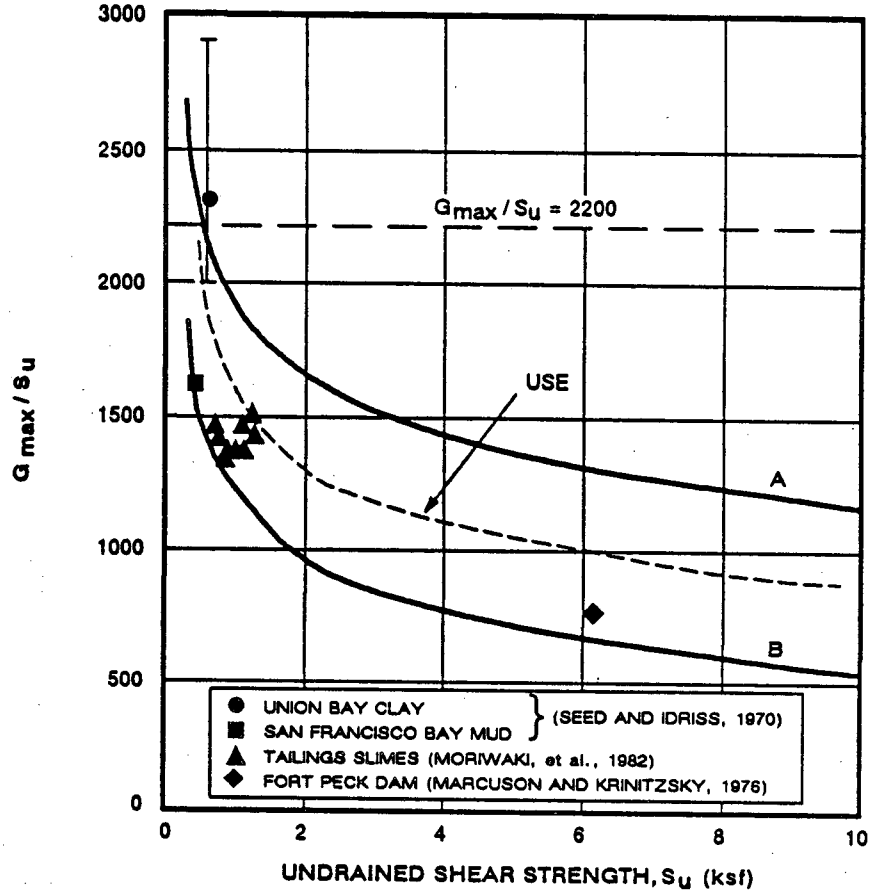


Figure 0-1-14
VARIATION OF NORMALIZED MODULUS AS MEASURED IN SITU WITH UNDRAINED SHEAR STRENGTH FOR SEVERAL SOILS

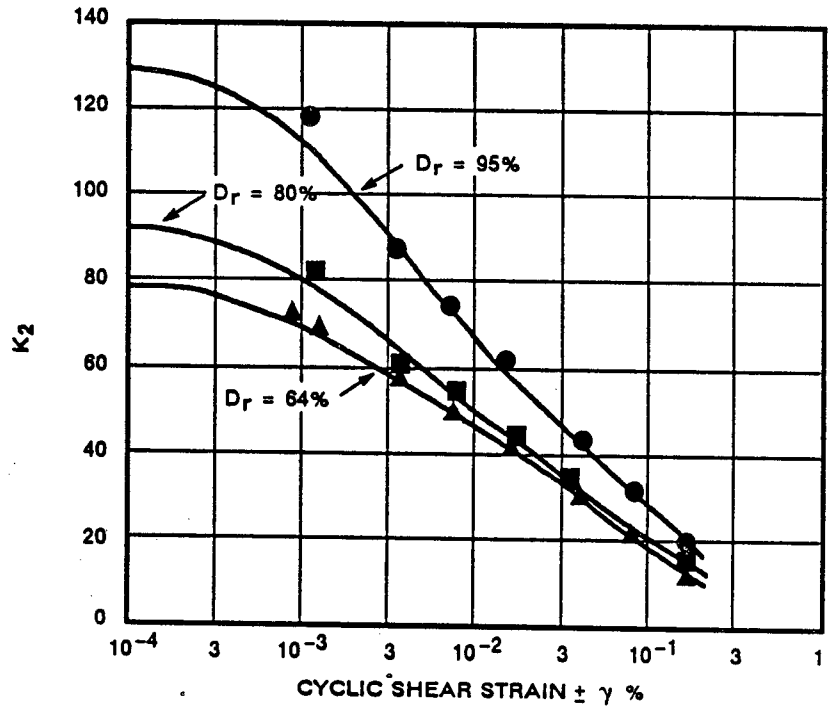


Figure 0-1-15
SHEAR MODULI OF MODEL-GRADED PYRAMID MATERIAL

ATTACHMENT O-2

RECOMMENDED SOIL PROPERTIES FOR STRUCTURES WITH SOIL STRUCTURE INTERACTION

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ATTACHMENT O-2

RECOMMENDED SOIL PROPERTIES FOR
STRUCTURES WITH SOIL
STRUCTURE INTERACTION

O-2.1.0 GENERAL

The following safety related WBN structures are to be analyzed considering Soil Structure Interaction.

- o Diesel Generator Building
- o Additional Diesel Generator Building
- o Refueling Water Storage Tanks
- o North Steam Valve Room

This Attachment describes the soil media profiles for these structures and provides recommendations on the appropriate soil properties to be used in their analysis.

O-2.2.0 REFERENCES

1. SAP Appendix O and Attachment O-1.
2. TVA, Division of Construction, Construction Services Branch, Report No. 9-2014, Watts Bar Nuclear Plant Site Investigation and Laboratory Testing.
3. Watts Bar Nuclear Plant, FSAR Figure 2.5-233.
4. Watts Bar Nuclear Plant, TVA Drawing 10W335.
5. Watts Bar Nuclear Plant, TVA Drawing 10W336.
6. Memo from Frank Van Meter to R. M. Pierce, September 13, 1979, Watts Bar Nuclear Plant, ERCW System Soils Investigation, En Des Soil Schedule No. 31.3A, RIMS CSB 790914003.
7. Memo from Frank Van Meter to R. M. Pierce, September 27, 1979, Watts Bar Nuclear Plant, Category I Soil Supported Structures, Additional Soils Investigations, RIMS CSB 790927002.

TABLE O-2-1

DYNAMIC SOIL PROPERTIES - DIESEL GENERATOR BUILDING

Elevation	Material	Total Weight	Poisson's Ratio	Low Strain Shear Modulus	Shear Modulus Vs. Strain	Damping Ratio Vs. Strain	Remarks
741 to 730	Class A Backfill	120 pcf	0.35	$G_{max} = 4.5 \times 10^6$ psf	Appendix O Figure O-3	Appendix O Figure O-4	Vary $G_{max} \pm 50\%$
732 to 727	Crushed Stone (above groundwater)	133 pcf	0.40	$G = 1000\ k_2\ (\bar{\sigma}m)^{1/2}$ (k_2) max = 100 ko = 1-sin ϕ = 0.4	Appendix O Figure O-5	Appendix O Figure O-6	Vary (k_2) max from 70 to 140
727 to 713	Crushed Stone (below groundwater)	142 pcf	0.40	$G = 1000\ k_2\ (\bar{\sigma}m)^{1/2}$ (k_2) max = 100 ko = 1-sin ϕ = 0.4	Appendix O Figure O-5	Appendix O Figure O-6	Vary (k_2) max from 70 to 140
730 to 718	In-Situ Cohesive Soils	120 pcf	0.35	$G_{max} = 4.5 \times 10^6$ psf	Appendix O Figure O-8	Appendix O Figure O-9	Vary $G_{max} \pm 50\%$
718 to 713	In-Situ Non-Plastic Soils*	120 pcf	0.38	$G = 1000\ k_2\ (\bar{\sigma}m)^{1/2}$ (k_2) max = 38 ko = 0.5	Appendix O Figure O-10	Appendix O Figure O-11	Vary (k_2) max from 30 to 70
713 to 708	Basal Gravel	143 pcf	0.46	$G = 1000\ k_2\ (\sigma m)^{1/2}$ (k_2) max = 365 ko = 1-sin ϕ = 0.4	Appendix O Figure O-13	Appendix O Figure 14-6	Vary (k_2) max from 200 to 500
708 to 693	Weathered Shale	127 pcf	0.35	$G_{max} = 12.8 \times 10^6$ psf (Vs. = 1800 fps)	Appendix O Figure O-15	Appendix O Figure O-15	Vary $G_{max} \pm 50\%$

* Includes CL-ML material with N values less than 10 blows per foot. For discussion of liquefaction potential for Watts Bar soils, see FSAR Section 2.5.4.8. Note: Design groundwater level = Elev. 727

TABLE O-2-2

DYNAMIC SOIL PROPERTIES - ADDITIONAL DIESEL GENERATOR BUILDING

Elevation	Material	Total Weight	Poisson's Ratio	Low Strain Shear Modulus	Shear Modulus Vs. Strain	Damping Ratio Vs. Strain	Remarks
741 to 730	Class A Backfill	120 pcf	0.35	$G_{max} = 4.5 \times 10^6$ psf	Appendix O Figure O-3	Appendix O Figure O-4	Vary $G_{max} \pm 50\%$
730 TO 718	In-Situ Cohesive Soils	120 pcf	0.35	$G_{max} = 4.5 \times 10^6$ psf	Appendix O Figure O-8	Appendix O Figure O-9	Vary $G_{max} \pm 50\%$
718 to-712	In-Situ Non-Plastic Soils*	120 pcf	0.38	$G = 1000\ k_2\ (\bar{\sigma}m)^{1/2}$ (k_2) max = 38 ko = 0.5	Appendix O Figure O-10	Appendix O Figure O-11	Vary (k_2) max from 30 to 70
712 to 709	Basal Gravel	143 pcf	0.46	$G = 1000\ k_2\ (\bar{\sigma}m)^{1/2}$ (k_2) max = 365 $ko = 1-\sin\phi = 0.4$	Appendix O Figure O-13	Appendix O Figure O-14	Vary (k_2) max from 200 to 500
709 to 693	Weathered Shale	127 pcf	0.35	$G_{max} = 12.8 \times 10^6$ psf	Appendix O Figure O-15	Appendix O Figure O-15	Vary $G_{max} \pm 50\%$

* Includes CL-ML material with N values less than 10 blows per foot. For discussion of liquefaction potential for Watts Bar soils, see FSAR Section 2.5.4.8. Note: Design groundwater level = Elev. 727

TABLE O-2-3

DYNAMIC SOIL PROPERTIES - REFUELING WATER STORAGE TANKS

Elevation	Material	Total Weight	Poisson's Ratio	Low Strain Shear Modulus	Shear Modulus Vs. Strain	Damping Ratio Vs. Strain	Remarks
732 to 713	Class A Backfill	120 pcf	0.35	$G_{max} = 4.5 \times 10^6$ psf	Appendix O Figure O-3	Appendix O Figure O-4	Vary $G_{max} \pm 50\%$
728 TO 713	Crushed Stone (below groundwater)	142 pcf	0.40	$G = 1000\ k_2\ (\bar{\sigma}m)^{1/2}$ (k_2) max = 100 ko = 1-sin ϕ = 0.4	Appendix O Figure O-5	Appendix O Figure O-6	Vary (k_2) max from 70 to 140
728 to 719	In-Situ Cohesive Soils	120 pcf	0.35	$G_{max} = 4.5 \times 10^6$ psf	Appendix O Figure O-8	Appendix O Figure O-9	Vary $G_{max} \pm 50\%$
719 to 713	In-Situ Non-Plastic Soils*	120 pcf	0.38	$G = 1000\ k_2\ (\bar{\sigma}m)^{1/2}$ (k_2) max = 38 ko = 1-sin ϕ = 0.5	Appendix O Figure O-10	Appendix O Figure O-11	Vary (k_2) max from 30 to 70
713 to 706	Basal Gravel	143 pcf	0.46	$G = 1000\ k_2\ (\sigma m)^{1/2}$ (k_2) max = 365 ko = 1-sin ϕ = 0.4	Appendix O Figure O-13	Appendix O Figure O-14	Vary (k_2) max from 200 to 500
706 to 693	Weathered Shale	127 pcf	0.35	$G_{max} = 12.8 \times 10^6$ psf (Vs. = 1800 fps)	Appendix O Figure O-15	Appendix O Figure O-16	Vary $G_{max} \pm 50\%$

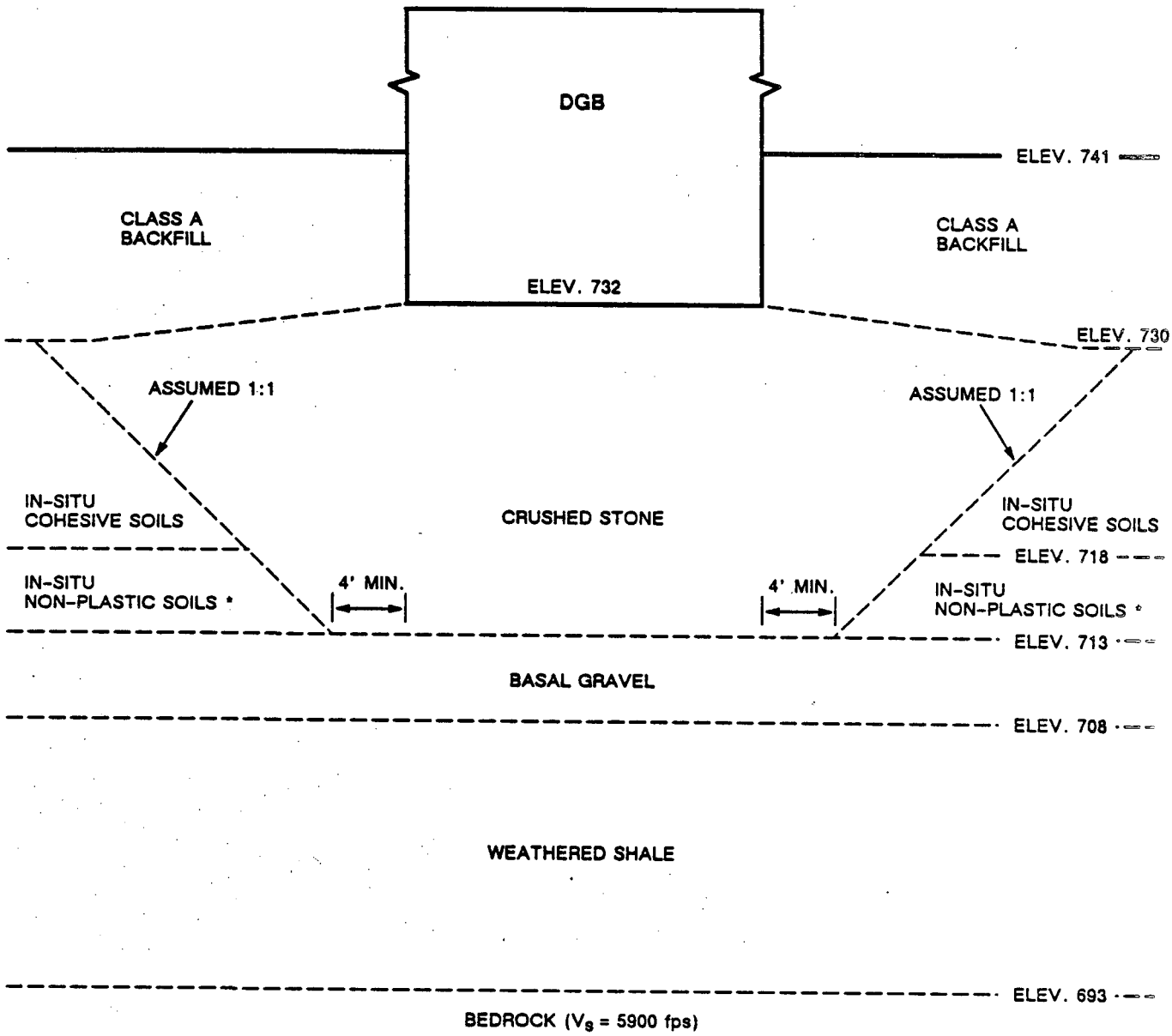
* Includes CL-ML material with N values less than 10 blows per foot. For discussion of liquefaction potential for Watts Bar soils, see FSAR Section 2.5.4.8. Note: Design groundwater level = Elev. 727

TABLE O-2-4

DYNAMIC SOIL PROPERTIES - NORTH STEAM VALVE ROOM

Elevation	Material	Total Weight	Poisson's Ratio	Low Strain Shear Modulus	Shear Modulus Vs. Strain	Damping Ratio Vs. Strain	Remarks
728 to 683	Class A Backfill	120 pcf	0.35	$G_{max} = 4.5 \times 10^6$ psf	Appendix O Figure O-3	Appendix O Figure O-4	Vary $G_{max} \pm 50\%$
728 TO 716	In-Situ Cohesive Soils	120 pcf	0.35	$G_{max} = 4.5 \times 10^6$ psf	Appendix O Figure O-8	Appendix O Figure O-9	Vary $G_{max} \pm 50\%$
716 to 698	Basal Gravel	143 pcf	0.46	$G = 1000\ k_2\ (\bar{\sigma}m)^{1/2}$ (k_2) max = 365 $ko = 1 - \sin \phi' = 0.4$	Appendix O Figure O-13	Appendix O Figure O-14	Vary (k_2) max from 200 to 500
698 to 683	Weathered Shale	127 pcf	0.35	$G_{max} = 12.8 \times 10^6$ psf (Vs = 1800 fps)	Appendix O Figure O-15	Appendix O Figure O-15	Vary $G_{max} \pm 50\%$

NOTE: Design groundwater level = Elev. 727

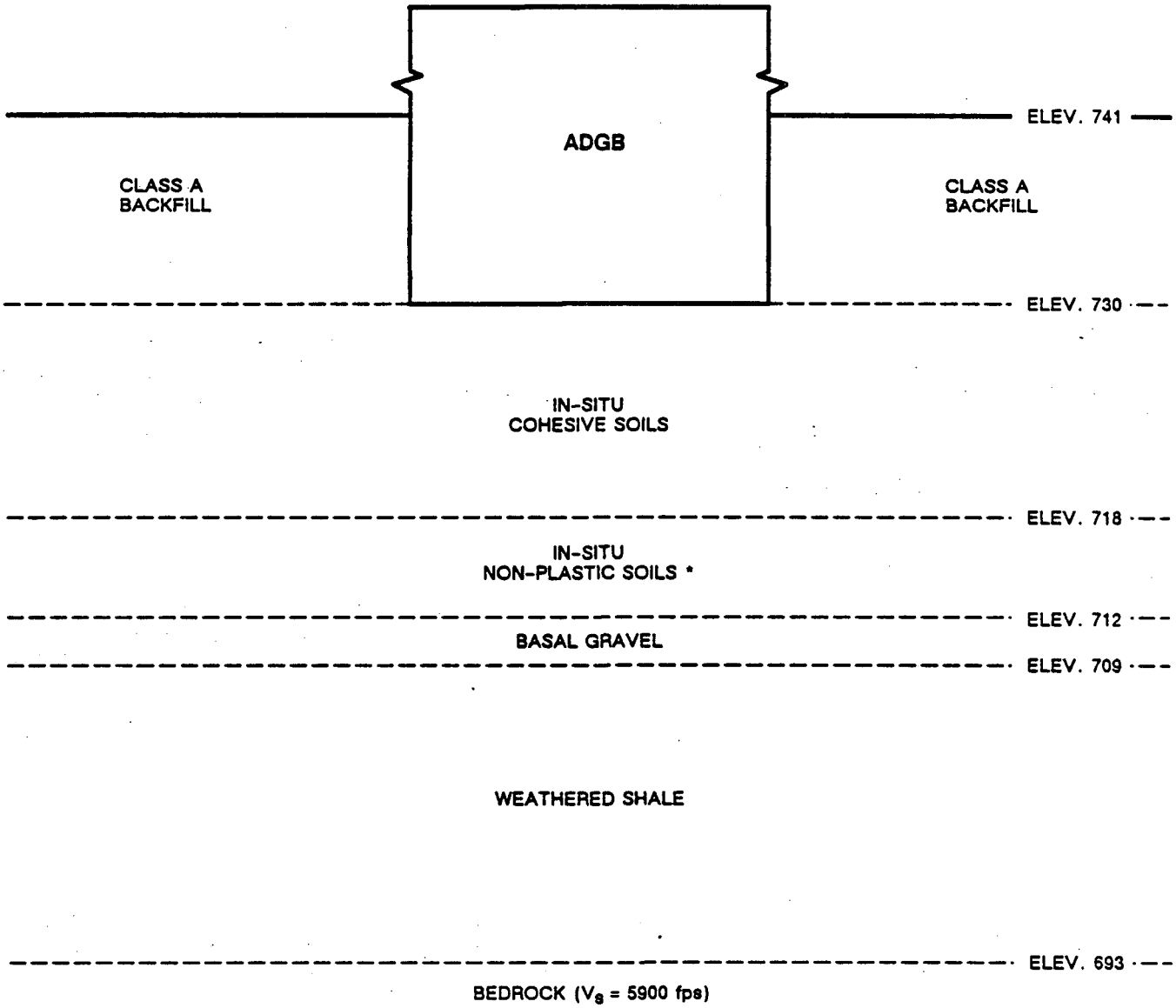


* - INCLUDES CL-ML MATERIAL WITH N VALUES LESS THAN 10 BLOWS PER FOOT

NOTE: DESIGN GROUNDWATER LEVEL = ELEV. 727
NOTE: GROUND SURFACE SLOPES AWAY AT 3:1 ON SOUTH SIDE OF BUILDING TO ELEV. 728

Figure O-2-1

SUBSURFACE PROFILE - DIESEL GENERATOR BUILDING
(REFERENCES: 1, 2, 3, 5, AND 7)

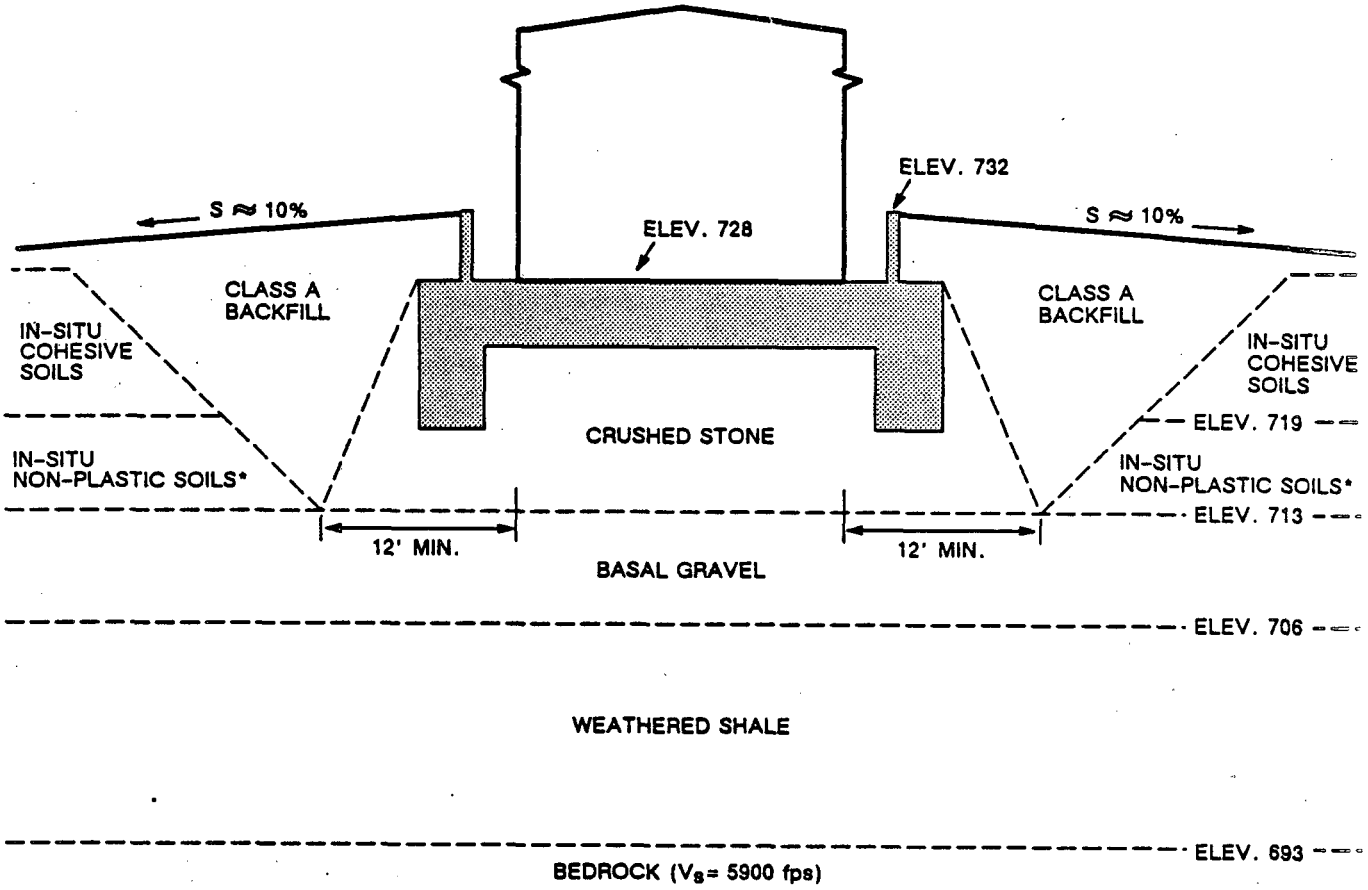


* - INCLUDES CL-ML MATERIAL WITH N VALUES LESS THAN 10 BLOWS PER FOOT

NOTE: THIS STRUCTURE SUPPORTED ON H PILES DRIVEN TO BEDROCK
NOTE: DESIGN GROUNDWATER LEVEL = ELEV. 727
NOTE: GROUND SURFACE SLOPES AWAY AT 3:1 ON SOUTH SIDE OF BUILDING TO ELEV. 728

Figure O-2-2

SUBSURFACE PROFILE - ADDITIONAL DIESEL GENERATOR BUILDING
(REFERENCES: 1, 2, 5, AND 6)



* - INCLUDES CL-ML MATERIAL WITH N VALUES LESS THAN 10 BLOWS PER FOOT

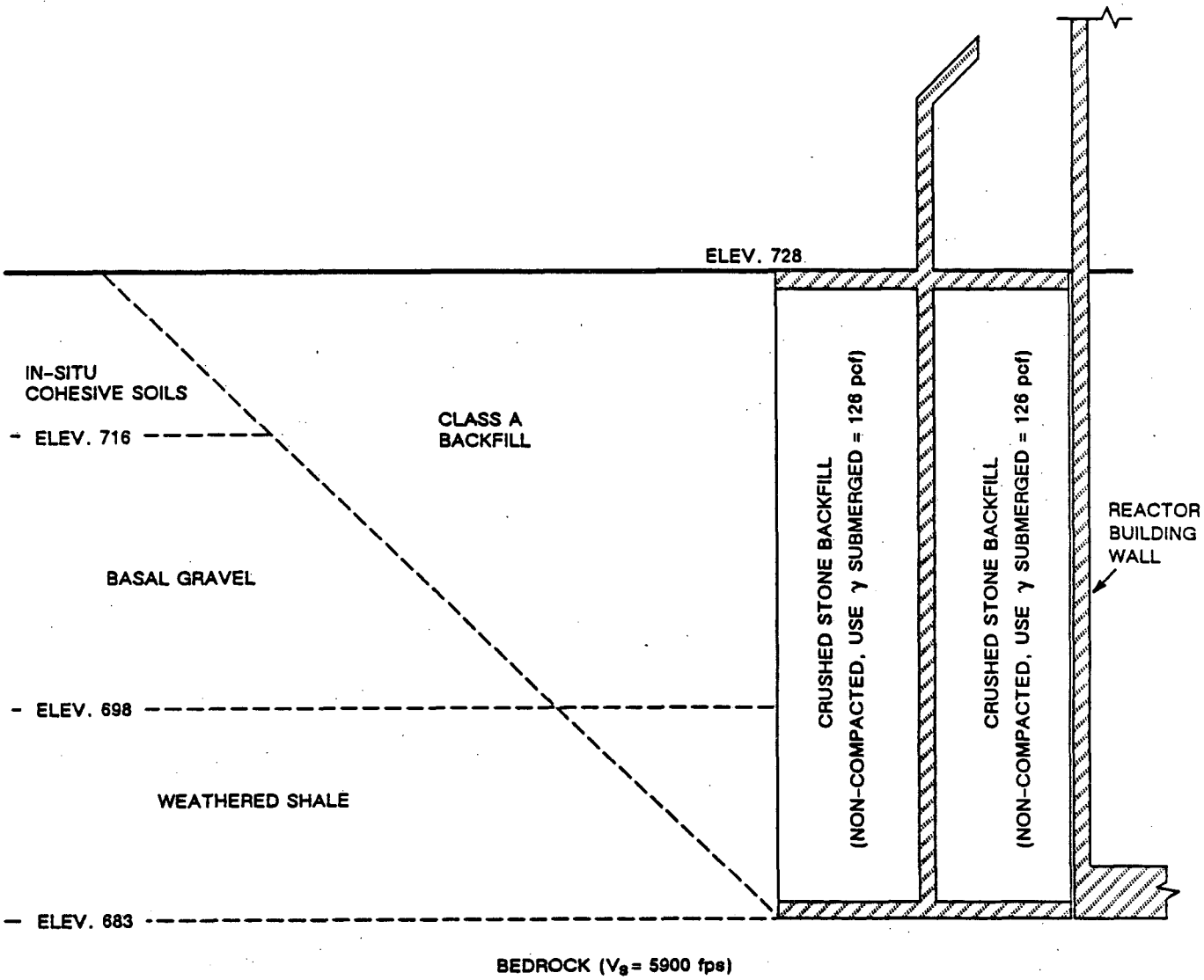
NOTE: PIPE TUNNELS NOT SHOWN

NOTE: DESIGN GROUNDWATER LEVEL = ELEV. 727

Figure O-2-3

SUBSURFACE PROFILE - REFUELING WATER STORAGE TANKS

(REFERENCES: 1, 5, AND 7)



NOTE: THIS IS A REINFORCED CONCRETE STRUCTURE FOUNDED ON BEDROCK. THE BELOW GROUND PORTION IS BACKFILLED WITH DUMPED STONE.

NOTE: DESIGN GROUNDWATER LEVEL = ELEV. 727

Figure O-2-4

SUBSURFACE PROFILE - NORTH STEAM VALVE ROOM

(REFERENCES: 1, 2, AND 4)

ENCLOSURE 1

CORRECTIVE ACTION PROGRAM PLAN

MATRICES