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HEADQUARTERS OFFICE
180 HOWARD STREET
SAN FRANCISCO, CALIFORNIA U.S.A. 94105
TELEX (WUI) 677058, (ITT) 470040, (RCA) 278362, (WUD) 34376
PHONE (415) 442-7300

Mr. John D'Antonio
Department of Energy
Uranium Mill Tailings Project Office
c/o Jacobs Engineering Group, Inc.
5301, Central Ave N.E., Suite 1700
Albuquerque, New Mexico 87108

Subject: UMTRA PROJECT: Review Comments on Documents for Working Group 2 - Geotechnical Stability

Dear Mr. D'Antonio:

Enclosed are copies of the following draft doucments distributed at the 2 July 1985 meeting of the DOE, NRC, TAC and RAC at NRC Headquarters in Silver Springs, MD.

1. For the "Draft Standard Review Plan (SRP) for Geologic - Seismologic Reviews of UMTRAP Documents prepared by the NRC", I have included my review comments in separate sheets instead of on the body of the SRP. I have confined my comments and discussion on four open issues, and presumed that TAC will provide comments on the entire SRP as it has primary responsibility for seismic hazard investigation including the preparation of the report.

2. For the TAC documents cited below I have provided marginal comments as agreed upon during the meeting. For the ultimate benefit of the users, I have also taken the liberty to expand the content of some of the documents:

   a) Site Characterization - Geotechnical (Preliminary Draft).

   b) Seismic Hazard Assessments for the UMTRAP (Preliminary Draft).

   c) Slope Stability (Preliminary Draft).

   d) Ground Settlement (Preliminary Draft).

   e) Liquefaction (Preliminary Draft).

Main open issues as below are primarily related to Seismic Hazard Assessments for the UMTRA facilities.

(i) Develop unified accepted terminology/definition of all key terms used.

(ii) Level of field and office efforts considered adequate for each UMTRA site.

(iii) Design life of UMTRA facilities, 200 or 1000 years (interpretation of 40 CFR Part 192).

(iv) Selection of acceleration attenuation function.

(v) Acceptable level of maximum earthquake acceleration (mean or 84 percentile).

(vi) Selection of design earthquake and related parameters.

(vii) Methods of Seismic Stability and Liquefaction analysis.

(viii) The need for a complete seismicity study report, containing conclusion, recommendations, and sufficient back-up data.

Regarding TAC's design manual, it appears as a step in the right direction, but it will need additional work to serve as an effective design guide.

Sincerely,

N. G. Banerjee
Technical Review Engineer

NGB:afp

cc: Mike Blackford/Benard Jagannath, NRC
Ron Rager, TAC

SHIPPING/MESSENGER and U.S. MAIL REQUEST FORM

ADDRESS:

TO: Nuclear Regulatory Commission
Waste Management Group
7915 Eastern Avenue
Silver Springs, MD 20915
Attn: Banad Jagannath

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180 Howard Street
San Francisco, CA 94105

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Date: 16 July 1985

Document Title: SRP for Geologic & Seismologic Reviews for UMTRAP Documents.
Doc. originated by: NRC.
Review Comments by: Nani G. Banerjee, MI Member, Workgroup 2

Review Comments of Nani G. Banerjee, MKE, Member, Workgroup 2 on Part of "Draft Standard Review Plan (SRP) for Geologic-Seismologic Reviews of UMTRA Documents - prepared by The NR

- I have not intentionally made any para by para comments on the draft SRP prepared by NRC and referred above.
- In Attachment-A I have discussed several open issues which need to be resolved so that we could develop seismic design criteria and procedures which meet the requirements of 40 CFR 192 as interpreted by me as well as complies with the NRC requirements stipulated in the S.R.P.
- I am also enclosing as Attachment-B, an abstract from 40 CFR Part 192; I have also included some short comments on various sections for the convenience of the reader. The purpose of this abstract is to bring into focus the spirit and intent of the regulation which require understanding for developing rational design procedures and also may find NRC approval.
- Also included as Attachment C are two tables developed from the published papers by Newmark and Seed which show the design horizontal seismic coefficients recommended for the pseudo analysis of some critical structures.

- Attachment-D includes a note on the Seismic Coefficient method of design of dams in Japan. It also includes a table showing the design seismic coefficients used in various parts of Japan.

- Attachment E: NRC's Draft SRP for Geologic-Seismologic Reviews of UMTRAP Documents.

Enclo: 5 Attachments A through E.

ATTACHMENT - A

OPEN ISSUES WHICH NEED NRC'S CONSIDERATION FOR INCLUSION IN THE S.R.P.

① Design Life: In 40 CFR Part 192, Section III, P for protection against flood at some UMTRA sites, the design life of 200 years is stated to be acceptable. In the same token, for the protection against seismic hazards, whether the design life of the UMTRA remedial designs could be assumed as 200 years without undue risks needs consideration.

It may be pointed out that the proposed EPA standard required at least 1000 years design life, which has been relaxed in the final Standard (Design life: 200 to 1000 years) due to the following considerations

a) So that it is practical for agencies to certify that the Standards are implemented in all cases (Section III, P597).

b) At least 50 million dollars may be unnecessarily spent (Section III, P595) in the design and construction of the tailings piles to meet the proposed requirement of at least 1000 years design life.

c) Refrain from actions undertaken merely in the name of necessarily artificial levels of statistical certainty (Section III, P590)

Considering the nature, location, geometeric configuration and characteristics of tailings piles, 200 years design is considered adequate by MKE for most UMTRA sites, not associated with undue hazards.

② Selection of Earthquake Acceleration Attenuation Relationship with Distance:

Because of the interaction of various complex parameters, no single attenuation equation is considered most suited for all sites so MKE believes, that mean earthquake acceleration values computed from 4 or 5 selected attenuation relationships which have found wide acceptance by by The professional community may provide the best answer: Alternatively, we recomend one or more than one of the following three earthquake acceleration attenuation relationships:

① Seed and Idriss (1982
② Joiner and Boore (1981.)
③ Iwasaki et al (1978).

③ Maximum Earthquake Acceleration

Two earthquake motions designated Safe Shutdown Earthquake (Equivalent to the term MCE used for UMTRA sites) and the Operating Basis Earthquake are used for the design of nuclear power plants. 10 CFR Part 100, Appendix A, p 823 states that "The maximum vibratory ground acceleration of the Operating Basis Earthquake shall be atleast one-half the maximum vibratory ground acceleration of the Safe Shutdown Earthquake.

• Considering the fact that we can not think of any mechanism of earthquake failure of the tailings piles, which could be remotely comparable with the health and safety hazards associated with the earthquake damage of nuclear power plants, we believe, the the choice of mean maximum rock acceleration of the MCE event may be adequate for the UMTRA sites; these mean acceleration values may need some correction based on site soil or rock conditions and the height of the embankments.

• It may be pointed out that spectral acceleration values are not significantly influenced by a substantial reduction in peak acceleration values. For example, a reduction in peak acceleration of 30% would appear to reduce the spectrum intensity by only 5% (Schnabel, P. B and Seed, H. B, 1972).

• There is also evidence that earth dams are damaged more severely by ground movements with longer periods (lower frequencies) than those with high frequencies which cause greatest damage to building structures. For this reason dams located very near the earthquakes may not be as badly cracked as dams many miles distant (Sherard, J. L et al, 1967).

• The need for the selection of 84 percentile peak earthquake acceleration as stipulated in the S.R.P may therefore please be reexamined.

(4) Earthquake Design Parameters & Method of Analysis:

The choice of the earthquake design parameters will depend not only on the characteristics of the MCE event but also on various other factors including the type of analysis (performed). Therefore, it will be helpful if you also discuss in the S.R.P NRC's approach towards the selection of Design Earthquake, design earthquake parameters and methods of analysis which will be acceptable to NRC.

We believe pseudo-static slope stability analysis with appropriate design horizontal seismic coefficient (a fraction of maximum earth-quake acceleration of the MCE event), involving minimum number of assumptions, should be adequate for the design of UMTRA facilities. In 10 CFR Part 100, Appendix A, p825, the above method is stated to be acceptable provided it is demonstrated that the results obtained by this method provides adequate conservatism. If required, we will also perform at some sites earthquake-induced permanent deformation analysis, also known as Newmark type analysis. We may examine the merits of various simplified methods of dynamic analysis and select the one which provides conservative but realistic answer.

We will perform simplified liquefaction analysis at all sites. The type or types of analysis will depend on the level of seismic risks and site conditions. The methods of analysis are described in MKE Design Procedure Manual, Chapter 11.

ATTACHMENT-B

(

Extracts from 10 CFR Part 192

with short notes & comments by Nani G. Banerjee, MKE
Member, Workgroup 2.

This extract is prepared first of all to better understand the meaning and spirit with which the EPA Standards were formulated; what rights responsibilities, flexibilites in the interpretation and implementation of the standards are vested in various Federal Agencies (DOE, NRC and EPA). How RAC could act in concert with these Agencies in developing design procedures for successful implementation of the remedial action program.

ABSTRACTED FROM 40 CFR Part 192

From page 590

. . . .

These standards are established to satisfy the purposes of the Act to ". . . stabilize and control . . . tailings in a safe and environmentally sound manner and to minimize or eliminate radiation health hazards to the public." The Act does not provide specific criteria to be used in determining that these purposes have been satisfied. We have therefore made it our objective to establish standards that take account of the tradeoffs between costs and benefits in a way that assures adequate protection of the public health, safety, and the environment; that can be implemented using presently available techniques and measuring instruments; and that are reasonable in terms of overall costs and benefits. We have been especially cognizant of the need to differentiate what would be desirable from what we believe to be necessary to achieve the purposes of the Act.

Note: The Act does not provide specific Criteria

Trade off between costs and benefits

Need to differentiate what would be desirable from what we believe to be necessary to achieve the purposes of The Act

From Page 591

The final standards provide for:

(1) *Control systems for tailings piles*—Control and stabilization which will ensure, to the extent reasonably achievable, an effective life of 1000 years, and in any case, for at least 200 years.

Expected life 200 to 1000 years.

From Page 592

SUMMARY COMPARISON OF PROPOSED AND FINAL STANDARDS

	Proposed	Final
Control of Tailings Piles:		
1. Longevity	At least 1000 years	Up to 1000 years, to the extent reasonably achievable, but at least 200 years.
2. Radon emissions from disposal site.	2 pCi/m^2s; equivalent to about 99.8% reduction.	20 pCi/m^2s, or 0.5 pCi/1 in air outside the disposal site; equivalent to about 96% reduction.
3. Water protection	Specific limits for a number of toxic and radioactive contaminants in ground-water; nondegradation of surface water.	Use existing State and Federal standards, apply site-specific measures where needed.
Cleanup of Buildings:		
1. Indoor radon decay products	Shall not exceed 0.015 WL	Shall not exceed 0.03 WL; to the extent practicable, achieve 0.02 WL.
2. Indoor gamma radiation	20 microR/hr	Unchanged.
Cleanup of Land:		
1. Surface	5 pCi/g in any 5 cm layer within one foot of surface.	5 pCi/g in the 15 cm surface layer.
2. Buried	5 pCi/g in any 15 cm layer below one foot.	15 pCi/g in any 15 cm layer below the surface layer.
Exceptions:		
1. Procedure	Site-specific exception procedures	Supplemental standards (may be applied on generic or site-specific bases).
2. Applicability	Where health and safety would be endangered, or where costs clearly outweigh benefits.	Same as proposed; criteria also provided to avoid cleanup of small amounts of tailings and inaccessible tailings posing minimal hazards.

Design Life

Supplemental Standards

From Page 492

"The Committee does not want to visit this problem again with additional aid. The remedial action must be done right the first time." (H.R. Rep. No. 1480, 95th Cong., 2nd Sess., Pt. I, p. 17, and Pt. II, p. 40 (1978).)

The remedial action must be done right the first time

From Page 593

The Act gives other agencies of the Federal Government the responsibility to decide how to satisfy these standards at specific sites.

** Responsibilities of other agencies.

From Page 595

The longevity (i.e., long-term integrity) of control is particularly important. This is affected by the potential for disruption by man; by the probability of occurrence of such natural phenomena as earthquakes, floods, windstorms, and glaciers; and by chemical and mechanical processes in the piles.

Potential sources of disruption of long term integrity.

From Page 595

Methods to prevent misuse by man and disruption by natural phenomena may be divided into those whose integrity depends upon man and his institutions ("active" controls) and those that do not ("passive" controls). Examples of active controls are fences, warning signs, restrictions on land use, and inspection and repair of semi-permanent tailings covers, temporary dikes, and drainage courses. Examples of passive controls are thick earthen covers, rock covers, massive earth and rock dikes, burial below grade, and moving piles out of locations highly subject to erosion, such as unstable river banks.

Examples of Active control

Examples of passive control.

From Page 596

III. Resolution of Major Issues Raised in Public Comments

A. The Basis for the Standards

From Page 597

3. Cost-Benefit Analysis. Commenters expressed the view that the cost of implementing the proposed standards will be high compared to the benefits, that we failed to carry out a cost-benefit analysis for these standards, or that we did not adequately consider alternatives to the standards proposed.

. . .

From Page 597

Based on these analyses, we have made a number of changes (described in Sections B and C, below) to make the standards more cost-effective and easier to implement.

Standards (more) cost-effective & easier to implement

One notable conclusion from our analysis is that providing tailings piles with thick, durable covers costs surprisingly little more than applying minimal covers that will require maintenance [illegible] last a much shorter time. This conclusion follows from the large start-up expenditures related to managing the remedial program and undertaking any significant level of remedial work at mill sites. Thick covers offer greatly increased benefits from inhibiting misuse, controlling radon emissions, and increased longevity of the covers' effectiveness. For example,

Thick covers offer greatly increased benefits

From Page 596

we estimate that the final control standard provides about ten times greater overall benefits than the lowest cost alternative standard, for only about 25 percent greater cost. Therefore, given that tailings piles will be stabilized under any of the alternatives we considered, we find it cost-effective to stabilize them well.

← Cost-effective to Stabilize them well

P.596 & 597

Benefits: We estimate benefits under the assumption, when appropriate, that tailings pile control systems will be partially effective longer than the standard requires. Control systems are required to be effective for as long as reasonably achievable up to 1000 years, but for not less than 200 years. Under this standard most of the 24 tailings pile will be stable against erosion and casual intrusion for misuse for much longer than 1000 years. Those few piles that are susceptible to flood damage will be protected for at least 200 years, and might not suffer real damage for much longer

** ← **? Therefore it is implied that 200-year design life criteria could also (be used) for earthquake damage

P597

B. The Standards for Control of Tailings Piles

1. Longevity of the Control.

P597

We considered longevity requirements ranging from 100 to 10,000 years and have concluded that existing knowledge permits the design of control systems for these tailings that have a good expectation of lasting at least for periods of 1000 years. We recognize that it may not always be practical, however, to project such performance with a high degree of certainty, because of limited engineering experience with such long time periods.

High degree of certainty may not be practical always for 1000-year design life.

P597

Section 104 of the Act requires the Federal Government to acquire and retain control of these tailings disposal sites under licenses issued by the Nuclear Regulatory Commission (NRC). The NRC is authorized to require performance of any maintenance, monitoring, and emergency measures that are needed to protect public health and safety.

** NRC may require performance of any maintenance, monitoring and immergency measures

P597

In the final standards we have modified the requirement for longevity of control so as to assure that it is practical for agencies to certify that the standards are implemented in all cases. The proposed standard required a longevity of control of at least 1000

P598 → years. The final standard requires that control measures be carried out in a manner that provides reasonable assurance that they will last, to the extent reasonably achievable, up to 1000 years and, in any case, for a minimum of 200 years.

** Note: Why in the final Standard the minimum design life requirement were relaxed from the proposed Standard requirement of 1000-years.

P. 598

(We estimate up to 50 million dollars might be unnecessarily spent to move piles under the proposed requirement for a longevity of at least 1000 years.) The change does not signify that there are circumstances under which the term of protection contemplated by the proposed standards is not appropriate. The change merely acknowledges that implementing agencies may in some cases have difficulty certifying that control measures that are appropriate can reasonably be expected to endure without degradation for 1000 years. Man's ability to predict the future is notoriously limited. That fact, which on the one hand warrants our making responsible societal efforts to limit risk to future generations, also warrants our refraining from actions undertaken merely in the name of necessarily artificial levels of statistical certainty.

← Estimated additional expense to assure atleast 1000-year design life: 50 million dollars.

Artificial levels of Statistical Certainty unwarranted

P601

A. Guidance for Implementation

P601

We are therefore providing ... "Guidance for Implementation" to avoid needless expense which may result from uncertainty or confusion as to what level of protection the standards are intended to achieve.

** Avoid needless expense.

P601

Implementation will require a judgment that the method chosen provides a reasonable expectation that the provisions of the standard will be met, to the extent reasonably achievable, for up to 1000 years, and, in any case, for at least 200 years. This judgment will necessarily be based on site-specific analyses of the properties of the sites, candidate control systems, and the potential effects of natural processes over time, and, therefore, the measures required to satisfy the standard will vary from site to site. We expect that computational models, theories, and expert judgment will be the major tools in deciding that a proposed control system will adequately satisfy the standard

**
Use site specific data, analyses & Judgement for implementing the Standard.

P601

We are confident that DOE and NRC, in consultation with EPA and the States, will adopt implementation procedures consistent with our intent in establishing these standards.

** DOE, NRC and EPA will develop implementation procedures.

P 601

B. Supplemental Standards

The varied conditions at the designated sites and limited experience with remedial actions make it appropriate that EPA allow adjustment of the standards where circumstances require. We believe that, in most cases, our final standards are adequately protective and can be implemented at reasonable cost. However, the standards could be too strict in some applications. We anticipate that such circumstances might occur. We originally proposed to deal with this through an "exceptions" procedure which would relax standards when certain criteria were satisfied. We agree with the comments, however, that the proposed procedure was unnecessarily burdensome to apply.

In the final regulations we have eliminated this procedure and replaced it with a simplified procedure for applying "supplemental standards." This is a more effective means of accomplishing our original purpose. An additional significant change in the proposed criteria for exceptions is the addition of criterion 192.21(c), which relaxes the requirement for cleanup of land at off-site locations when residual radioactive materials are not clearly hazardous and cleanup costs are unreasonably high. This category of contamination was not adequately addressed in the proposals.

ATTACHMENT-C 1/2

SELECTION OF DESIGN EARTHQUAKE PARAMETERS FOR CRITICAL STRUCTURES
FOR CRITICAL STRUCTURES

Peak acceleration of Maximum Credible Earthquake (MCE)/Seismic Safety Evaluation Earthquake	Recommended Peak Acceleration of Seismic Engineering Design Earthquake	Comments & References
0.50g ~ 0.60 (Use for liquefaction analysis)	0.10g ~ 0.15g (Use for pseudo-static analysis)	According to Seed (Ref. 14, p.59): ". . . Both theory and experience show that this is perfectly reasonable procedure."
1.0g[a]	0.30g ~ 0.60g (For the design of earth structures)[b] 0.18 ~ 0.35g (For the design of steel structures)	[b]Specified by Newmark and Hall (1973) (Ref. 15)

a. Specified by Page R.A et. al., US&GS (1972, Ref. 17).

2/2

DESIGN CRITERIA FOR THE PSEUDO-STATIC ANALYSIS OF EMBANKMENTS

[After Seed (1979), Ref. 16, p. 236]

Earthquake Magnitude	Design Criteria	Comments & References
6-1/2	FS = 1.15 for Seismic Coefficient = 0.1	a) Applicable for embankments constructed of soils which do not build up large pore pressures due to earthquake shaking nor show more than 15% strength loss (usually).
8-1/4	FS = 1.15 for Seismic Coefficient = 0.15	

A Note on the Seismic Coefficient Method of Design of Dams in Japan

In the so-called seismic coefficient method of dam design, the weight of the dam body itself and a part of reservoir water determined by the formula of dynamic water pressure are multiplied by the seismic coefficient, and the value obtained is treated as the earthquake force. These forces of inertia are applied horizontally to the dam body to calculate stresses and stability. This method has been in use in Japan since the time high dams began to be constructed in that country.

This method has been improved based on studies and research on earthquake phenomena and behavior of dams during earthquakes. The seismic coefficients selected for dam design are at present determined by various factors such as the type of dam, geological conditions, and occurrence of historic earthquakes in the vicinity of the proposed dam. Table-B1 shows the design seismic coefficients established by the Japanese National Committee on Large Dams (Ref. 18, p. 20). The values are classified by the types of dams and the regions in which the proposed dams are to be located.

TABLE B-1 - DESIGN SEISMIC COEFFICIENTS FOR DAMS IN JAPAN*

(Ref. 18, p. 21)

Type of Dam	Part of Tohoku region; Kanto region; Chubu region; Kinki region; Southern Shikoku region	Hokkaido region; Hokuriku regions Other part of Tohoku region; Chugoku region; Northern Shikoku region: Kyushu region
Concrete Dams and Rock-Fill Dams	$k = 0.12 \sim 0.20$	$k = 0.10 \sim 0.15$
Earth Dams	$K = 0.15 \sim 0.25$	$k = 0.12 \sim 0.20$

(A) (1)

PRELIMINARY DRAFT

SITE CHARACTERIZATION - GEOTECHNICAL

1.0 INTRODUCTION

materials extracted from
extracted from and observed in test pits and by the examination of the results of

The purpose of geotechnical site characterizations for UMTRA sites is to define the physical conditions of the existing tailings piles, foundation soils and proposed borrow sources. These conditions include the stratigraphy and physical properties of individual materials composing the stratigraphic units. Stratigraphy is determined by visually logging boreholes and ~~by~~ static-cone penetration tests. Material~~s~~ properties are determined by laboratory testing and field tests. Ground water site characterization, which is closely related to geotechnical site characterization is presented in another position paper.

and other types of

Site reconnaissance survey will be followed by detailed field and laboratory investigation at each UMTRA site, to supplement the data available from previous studies. Therefore the nature and extent of investigation required will vary from one site to the othe The objective will be to collect sufficient field and laboratory test data so that the site and soil conditions could be characterized with reasonable degree confidence; this is best accomplished by developing several geologic Cross-Sections/Soil profiles through the Site area. Such profiles will show the existing ground surface and or the tailings surface, location and log of all borings along the profile, show major soil layers description & classification, bedrock surface profile, bedrock type/description and ground water level.

This chapter describes the method of exploration and the type of tests proposed to be performed at sites marked for in place stabilization, relocated sites and the borrow areas.

Document Title: SITE CHARACTERIZATION - GEOTECHNICAL
Doc. Originated by: TAC (Jacobs- Weston)
Marginal Review Comments by: Nani G. Banerjee, MKE
Member, Workgroup 2

2.0 ARCHIVED DATA

Several of the UMTRA sites have geotechnical site characterization data available from previous studies. This data varies in type, quality and quantity and must be assessed for appropriateness on an individual site basis. This data may include boring logs, penetration test data and laboratory test data. Depending on the present concept toward stabilization at a site and the type and suitability of the available data toward use in this concept, a more limited program may be required than is outlined in this paper. A review of all data available will be made at each site and the generic site characterization program will be appropriately modified at that point.

To optimize the use of available data, and avoid the duplication of efforts, all the available data for a particular site should be collected and compiled in the form of soils and foundation report. Such reports should contain location map showing all exploration work, list of borings with their x, y and z coordinate soil test results results summary including simple statistical data (Range, Mean value, Standard Deviation and data population, etc) Typical format for compiling data summary are included in MKE'S UMTRA DESIGN PROCEDURES, Chapter 12. Early collection, compilation, and reduction of available data will be helpful for TAC to develop a cost effective, efficient and adequate additional exploration program; it also could be used with advantage by RAC for final analysis.

* The Standard Penetration Test procedures should be closely observed by knowledgeable Geotech. engineers. When carelessly performed the SPT data could be very misleading due various errors contained in them

3.0 STABILIZATION IN PLACE SITES

3.1 FIELD STUDIES

Is not this ASTM Standard for Cone Penetrometer?

The nature and material properties of the tailings piles must be determined if SIP is to take place without recompaction of the pile. In addition, the behavior and stratigraphy of the foundation soils must be determined in order to assess the long term stability of the pile. A series of piezocone penetration tests (ASTM D3441) are performed at a density of one (1) per acre to cover the tailings pile. Each test will penetrate the entire depth of the pile and up to 5 to 10 feet into the foundation soils. Output from those tests is in the form of a continuous profile of the stratigraphy (as shown on Figure 1) and a digital tape of the data collected. Interpretation of data collected from these probes will be used to: 1) derive the stratigraphy of the pile in detail, thus locating all significant layers, zones, and pockets of slimes within the embankment; 2) determine dissipation rates of induced pore pressures (Figure 2) which are used to estimate in-situ permeability and consolidation characteristics of the materials; 3) obtain the penetration resistance of the tailings as related to their strength and bearing capacity characteristics, and 4) determine the ground water level.

Is there yet an ASTM standard for piezocone?

May need borings at closer spacings if any soft slimy material is encountered.

Shelby tube wall should satisfy the thin walled tube criteria so that the samples are undisturbed.

Based upon interpretation of the stratigraphy, selected borings will be located and conducted to obtain undisturbed samples for laboratory testing. Approximately one (1) boring will be required for every 4 acres. At least two (2) Shelby tube samples and numerous standard penetration test * (SPT) samples will be obtained form each boring. In the event the soils prove too stiff to obtain Shelby tube samples, split barrel samples will be obtained instead. The Shelby tube and split barrel samples will provide undisturbed samples for laboratory testing while the standard penetration tests will provide a correlative basis for interpretation of the data with other published or unpublished data and will be used in liquefaction analysis. Ground water levels at the time of drilling will be determined, but no piezometers will be installed. These borings will extend a minimum of 20 feet below the tailings-natural soil interface. One of the borings will extend to bedrock or up to 250 feet below the interface if foundation stratigraphy and material properties require definition.

At what intervals?

At shallow depths the intervals should not exceed 3' to 5'.

On small piles, the piezocone data will not be required and borings, conducted on a similar grid as described will be substituted.

Test pits will be excavated on the pile to obtain representative sand, sand-slime and slime tailings samples for laboratory testing. This data will be used for determining the remolded geotechnical properties of fill soils.

If the soil is clayey and bentonite is used as the drilling mud, the water level in the hole may not reflect the true Ground Water Table. It may take several days for the water level to stabilize to the true G.W.T. How you plan to resolve this problem?

General Comment

- If any soft material is encountered within the pile it will be necessary to establish the boundary of the soft material by boring at closer spacing, as otherwise the computed differential settlement may be misleading.

- Where the foundation soil on the tailings are essentially very loose to loose sand, perform continuous SPT tests than try to retrieve Shelby Tube Samples.

3.2 LABORATORY TESTING

This includes residual moisture content. How about Diffusion Coefficient

Laboratory testing of undisturbed and SPT samples for correlative material properties will be conducted (see Attachment A). In addition, strength (triaxial compression) compressibility (one-dimensional consolidation), permeability, capillary moisture, and other tests will be conducted on the undisturbed soil samples. The results of this testing will be correlated to the piezocone data. A relationship will be developed between the field data and the tested properties which will ultimately reduce the need for laboratory strength testing.

* If the soft material — within the pile occupy significant volume, adequate number of representative samples should be tested for consolidation properties.

- At some sites where the (borrow) soil (or the tailings) are of marginal or doubtful quality some special tests may be necessary, such as swelling and shrinkage test, sensitivity, mineral composition, etc.

When strength test data is determined by indirect methods, it should be clearly spelled out in the soil test results summary, so (that) some degree of conservatism is used in developing parameters for design or analysis.

(5)

4.0 RELOCATED SITES (INCLUDING STABILIZATION ON SITE)

4.1 FIELD STUDIES

4.1.1 Disposal area

Grid spacing?

Borings at closer spacing are desirable along the deepest/critical section.

In order to determine the foundation soil and/or bedrock characteristics at the disposal sites, a series of borings will be required. The density of borings will be approximately one (1) boring for every 3 acres. A sufficient area will be covered to allow repositioning of the pile within the general area. Shelby tube, 2-1/2 inch diameter split barrel and standard penetration test (SPT) samples will be collected to aid in classifying the soils, correlating data, and for further laboratory testing. Ground water levels at the time of drilling will be determined, but no piezometers will be installed. Field packer tests will be conducted where applicable in order to determine in-situ permeability characteristics. These borings will extend at least 20 feet below grade, with at least 2 borings extending up to 50 feet below grade or into bedrock.

Please see note on page 3

One of the borings may extend as deep as 250 feet if a deep soil site is encountered.

4.1.2 Tailings pile

Piezocone penetration tests (ASTM D3441), or borings with SPT sampling, will be conducted for full thickness of the pile at a density of one (1) test per 4 acres in order to define the sand-slime makeup of the pile. Test pits will be required (these may be combined with the rad waste group work) in order to obtain bulk samples for laboratory testing of remolded material properties.

4.2 LABORATORY TESTING

A full range of material property, strength, and compression tests will be required on the disturbed and undisturbed samples obtained from each area, similar to what is outlined in Attachment C.

To incorporate appropriate corrections please ask the inspector in the field to record the test procedures in details, such as, type of hammer used, method of dropping hammer, type of split spoon barr used (with or without inside liner), total length of drill rods, type of drill rods used, etc.

- drillrods should be pulled out very slow, otherwise quick condition could be created which will disturb the strata.

Very useful soil data also could be obtained from the nearest U.S. Soil Conservation Service Office/ US Forest Dept. or the County Office

(6)

5.0 BORROW AREA SITES

5.1 GENERAL

Borrow areas will be identified by performing a preliminary borrow assessment. This study will be performed by a TAC geologist or geotechnical engineer and shall consist of a review of any pertinent data and a site visit. Local materials contractors will be contacted in an effort to obtain information on the availability of local borrow sources.

5.2 RADON COVER

5.2.1 Field program

(1) Check the Ground water Table.
(2) Insitu density and moisture, so that realistic Shrinkage & Swelling factor could be determined; also the need for moisture conditioning during compaction could be assessed.

Following preliminary borrow assessment, a limited number of areas will be investigated by excavating 8 to 12 test pits at each area. The test pits will be spaced in an effort to define the limits of suitable borrow material. Both large and small bulk samples will be obtained in order to perform classification and material properties tests. A field log of each test pit will also be maintained.

• Take close up photo of the test pits showing the soil profile (DOE/NRC engineers may like to see.)

5.2.2 Laboratory testing

• Not always possible.
• Preferable to select/ examine atleast two prospective sources.
• Need to make Comparative study of Quality & Cost.

By visual examination of the soil samples and review of test pit logs, the most suitable borrow area will be selected and samples from this area will be subjected to laboratory tests. If all sites appear equal, the most economical to develope will be used. Selected samples obtained from the field program will be tested for their mechanical characteristics, strength, compressability, permiability, capillarity, radon diffusion, and erodability. Depending on the nature of the borrow source individual or mixed samples will be tested. In addition, soil amendments may be added to certain soils in order to alter their behavior.

5.3 ROCK ARMORING

• Most desirable will be to use the native soil available (in abundance) in the vicinity.

5.3.1 Field program

Following the preliminary borrow assessment several areas will be investigated in order to define the limits and quality of Rock Armor borrow material. Six to eight test pits will be conducted at each area. Both large and small bulk samples will be obtained in order to perform classification and material properties tests. A field log of each test pit will also be maintained.

Not realistic in rock quarry.

(1) Check the rock type. for US&GS maps.
(2) Find out the nearest Commercial Rock Quarry. Request for Test data & Unit cost.
(3) First step is reconnaissance Survey by a Geologist/ Geotech Engineer
(4) Nearest County Offices could be very helpful.
(5) Borings or Blast Test, if necessary. ~~(6) Load Test~~

7

5.3.2 Laboratory Testing

?

By visual examination of the soils obtained from the field program and a review of the test pit logs, the most suitable borrow area will be selected for further testing. The samples obtained from the test pits will be subjected to minimal durability and soundness testing.

Are you not discussing in this section laboratory testing of armor rock for slope protection.

Proposed Rock Tests

1. Specific Gravity
2. Absorption. Test.
3. Los Angeles Abrasion Test.
4. Soundness Test in Sodium Sulfate soln.
5. Freezing and Thawing test.
6. Porosity.

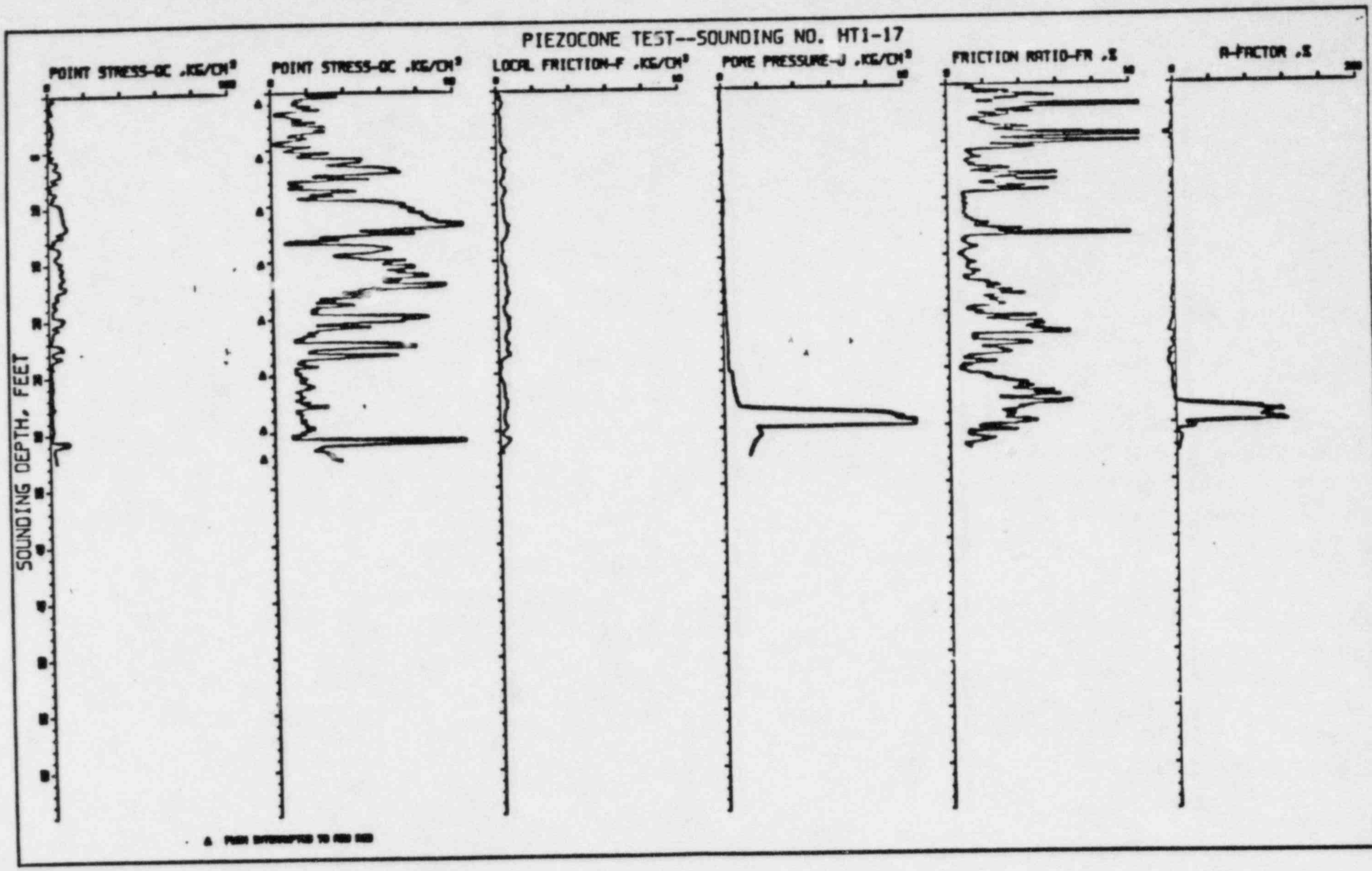
PIEZOCONE TEST--SOUNDING NO. HT1-17
POINT STRESS-QC ,KG/CM²
POINT STRESS-QC ,KG/CM²
LOCAL FRICTION-F ,KG/CM²
PORE PRESSURE-U ,KG/CM²
FRICTION RATIO-FR ,%
A-FACTOR ,%
SOUNDING DEPTH, FEET

FIGURE 1

8

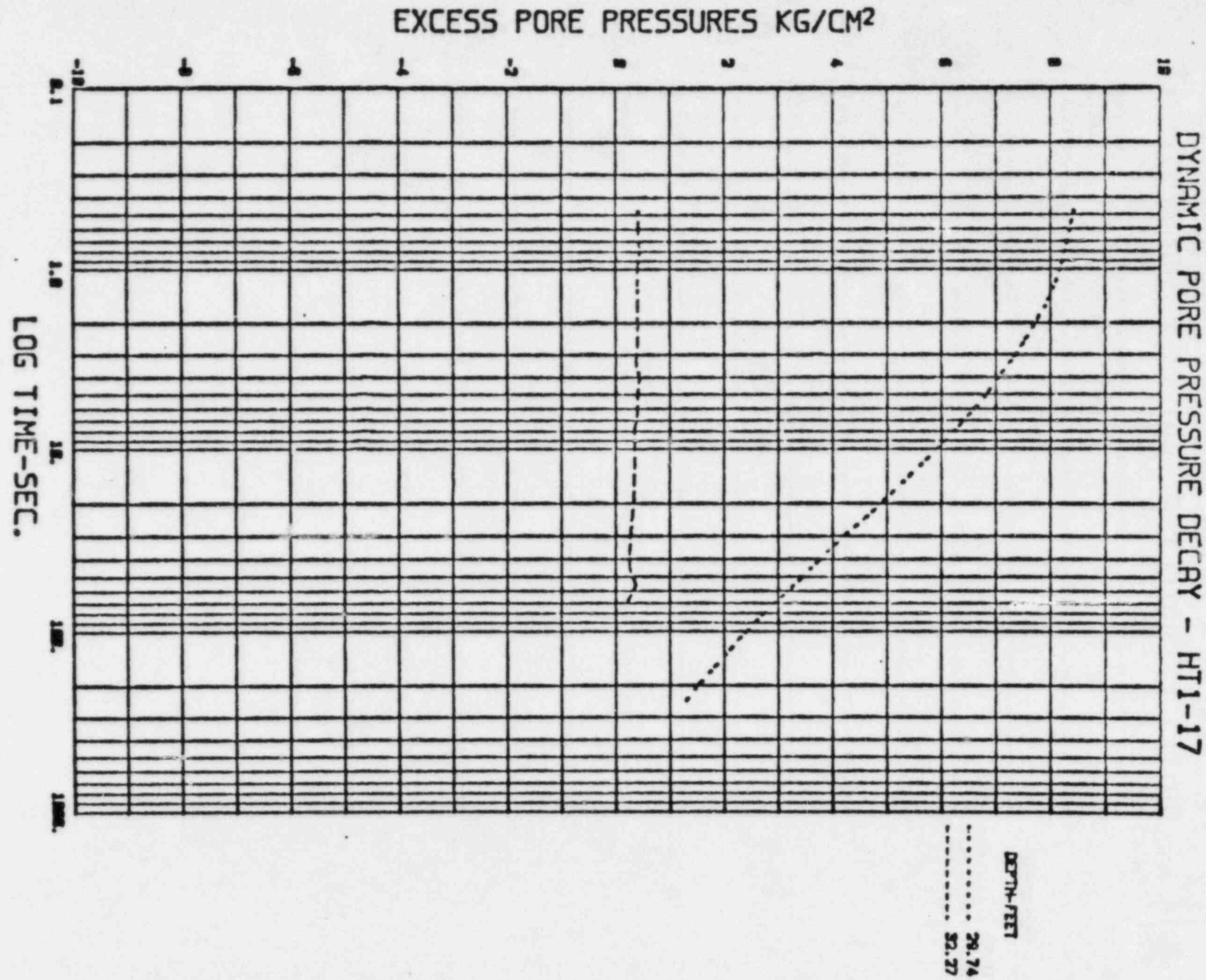
EXCESS PORE PRESSURES KG/CM2
DYNAMIC PORE PRESSURE DECAY - HT1-17
LOG TIME-SEC.
DEPTH-FEET

Figure 2

9

(10)

CONTRACT ASD-34-6703-S-84-0025

EXHIBIT A
REVISION C
DATED 5/28/85

SCOPE OF WORK

Basic Ordering Agreement for Geotechnical Laboratory Testing

Jacobs Engineering Group Inc. (Jacobs) requires the Subcontractor to provide the services and supplies required for geotechnical laboratory testing of soils in support of Jacobs' prime contract to the U.S. Department of Energy under the Uranium Mill Tailings Remedial Action (UMTRA) Project.

It is intended that this program will provide a detailed evaluation of the soils at 20 to 30 potential borrow sites, about 10 uranium mill tailings sites to be reclaimed under the UMTRA Project and as many as 5 selected disposal sites should stabilization of the mill tailings in place prove impractical.

The following outlines the proposed scope of services that may be required at each site to obtain geotechnical information on the proposed materials. Sample collection and delivery will be done by others.

1. Soil Sample Containers and Labeling

Soil samples contained in large and small plastic bags, Shelby tubes, and 2 to 3 inch diameter tube (ring) samples will be provided by Jacobs' Technical Representative (TR). The samples will be independently labeled by the TR for adequate identification as to site, location, sample number, depth, etc.

2. Laboratory Testing

Laboratory testing of retained soil samples will be required. All testing shall be performed in conformance with the latest edition of the appropriate ASTM Standard or other specified standard. Tests which are anticipated to be required, but for which no standard exists, will require the Subcontractor to present, in writing, his test procedures to the TR. These methods will then be approved, disapproved or approved with modification to the satisfaction of the TR and the Subcontractor prior to performing any testing. Tests which may be necessary on each site include, but are not limited to, the following:

ATTACHMENT C

(11)

Sieve analysis without hydrometer (ASTM C136)
Sieve analysis with hydrometer (ASTM D422)
Atterberg limits (ASTM D4318)
Moisture content (ASTM D2216)
Moisture density (ASTM D698)
Moisture density (ASTM D1557)
Capillary moisture relationships (ASTM D3152 and ASTM D2325)
Specific gravity (ASTM D854)
Triaxial permeability (EM 1110-2-1906)
Constant head permeability (Army Corp of Engineers EM 1110-2-1906)
Falling head permeability (Army Corp of Engineers EM 1110-2-1906)
Three point sets Triaxial (R) (Army Corp of Engineers EM 1110-2-1906)
Tnree point sets Triaxial (Q) (Army Corp of Engineers EM 1110-2-1906)
Three point direct shear test (CD) (ASTM D3080)
Three point direct shear test (CU)
Three point direct shear test (UU)
Dry density
Slake durability(International society for rock mechanics, suggested method for determination of the slake-durability index)
Three point sets California Bearing Ratio (ASTM D1883)
One-dimensional consolidation (ASTM D2435)
Increments of secondary consolidation (per machine day)
Crumb tests (ASTM Proceedings STP623)
Pinhole (ASTM Proceedings STP623)
Double hydrometer (ASTM D422)
Aggregate specific gravity and absorption (ASTM-C127)
Aggregate soundness (ASTM C88)
Los Angeles abrasion (ASTM C131 and C535)
Remolding of samples per test sample
Rock crushing in preparation of samples per bulk sample

Other tests that may be requested from time to time include:

Unconfined compression (ASTM D2166)
Relative density (ASTM D4253 & D4254)
Shrinkage limit
Percent passing No. 200 sieve
Expansion, shrinkage and uplift (ASTM D3877)
Falling head permeability conducted in association with consolidation tests (per load increment)

3. Testing Procedures

a. General

All testing shall be performed in conformance with the latest edition of the appropriate ASTM Standards or other standard as indicated by the type of test performed.

b. Compacting Samples of Cohesive Soil

Samples of compacted soil shall be prepared in a split mold having inside dimensions equal to the dimensions of the desired sample. The method of compacting this soil into the mold should duplicate as closely as possible the field compaction method. The soil should be compacted into the mold in 6 equal layers using a pressing or kneading action of a tamper having a contact area with the soil of less than one-sixth the area of the mold. The surface of the layer should be thoroughly scarified before placing the next layer. Under no circumstances shall standard impact types of compaction be acceptable.

The sample shall be prepared according to the ASTM D-698 test procedure using an appropriate amount of water to produce the desired water content.

The desired density shall be produced by either kneading or tamping each layer until this accumulated weight of the soil placed in the mold is compacted to a known volume or by adjusting the number of tamps per layer and the force per tamp. For the latter method of control, special constant force tampers are necessary. After each sample has been compacted to finished dimensions and removed from the mold, the appropriate laboratory test may be performed. Input parameters such as moisture content at compaction, etc., will be provided by the TR.

Preparation of compacted granular soils should be performed as outlined in the U.S. Army Corp of Engineers' "Laboratory Soil Testing," publication EM1110-2-1906.

c. Consolidation Testing

Consolidation tests must include time-rate of settlement plots of all load increments. These plots will be either log-time or square root of time plots, whichever best defines the end of primary consolidation. Falling head permeability tests may be required on certain consolidation tests.

d. Falling Head Permeability Tests

Testing procedures for falling head permeability tests shall be conducted according to EM 1110-2-1906. Input parameters such as confining pressures will be provided by the TR. At least 95% saturation of the samples is expected. In addition, falling head tests, conducted during consolidation testing may be required.

e. Constant Head Permeability Tests

Testing procedures for constant Head Permeability Tests shall be performed according to EM 1110-2-1906. Input parameters such as confining pressures will be provided by Jacobs Engineering Group Inc. At least 95% saturation of the samples is expected. Pressure can be applied to achieve saturation, but shall not exceed an equivalent position head of 15 feet of water.

f. Triaxial Testing

Triaxial testing of select undisturbed or compacted samples may include permeability tests, unconsolidated undrained tests (Q), consolidated undrained tests with pore pressure measurements (R). All testing shall be conducted according to procedures outlined in EM 1110-2-1960. A "B" parameter of 0.97 or higher is expected on all test samples prior to shearing, unless otherwise indicated. Input parameters such as confining pressures, etc., will be provided by the TR. Photographs of the sample shall be included in the test data, showing the condition of each sample at failure. These should show an external view(s) and a cross section view of the sample.

g. Capillary Moisture Relationships

Capillary Moisture relationships shall be determined for a specific soil sample using a combination of ASTM D3152 and ASTM D2325 test methods to produce a series of moisture contents at tension values ranging from 0.1 to 15 bars. The increments used should be 0.1, 0.3, 0.5, 0.7, 1.0, 2.0, 4.0, 7.0, 10.0 and 15.0 bars.

4. Project Schedule

A site specific workplan (Delivery Order) will be sent to the Subcontractor along with the samples to be tested. All analyses for each phase must be completed within four weeks after receipt of the samples unless otherwise specified in the Delivery Order as issued. For selected specific gravity, moisture density gradation and Atterberg Limits tests a two week completion will be required.

5. Quality Assurance

All laboratory testing shall be performed by experienced and qualified personnel in conformance with the applicable ASTM test procedures. Any deviation from these procedures or any analytical procedures that are not available from ASTM shall be submitted in writing to the Jacobs Contract Representative (CR), who, after consulting Jacobs' Quality Assurance Manager will provide approval of any such procedure, prior to performing the test. These deviations shall be carefully documented and included on the typed laboratory report. The laboratory which is to perform the testing, including equipment, shall be available to the QAM's representative prior to and during the testing for inspection.

The laboratory must have a Jacobs' approved Quality Assurance (QA) Program in affect to assure that the data transmitted is correct and that the lab tests were run according to the required standard. The Subcontractor's QA program shall provide a designated person as the primary contact person should any questions arise as to the reliability of transmitted data.

6. Contract Performance

All testing is subject to review and acceptance by Jacobs. Acceptance or non-acceptance of a deliverable, will be made by the TR within 14 days after receipt of test data. Tests improperly or inadequately performed will be retested at no cost to Jacobs.

All testing must be performed by the Subcontractor. No tests are to be further subcontracted without prior approval by the Jacobs CR. If different tests are to be run at various laboratory's within the same company, the tests which will be run at each lab must be specified on Attachments to Exhibit C, and the reason given for more than one lab. Shipment of samples will only be paid by Jacobs to the Subcontractor's laboratory nearest Albuquerque, New Mexico.

Any discrepancies in data must be identified and explained on the "Comments" section of the forms attached under Exhibit C; as to the unusual nature or reason for apparent invalid test results.

7. Deliverable Quality Assurance

Results of all analyses shall be submitted on the specified reporting forms (Exhibit C) and accompanied by legible copies of all associated laboratory work sheets. Reporting forms shall be typewritten with all lines on the form being completed. The letter designation "N/A" for not applicable or "N/K" for not known

will be used in all blank spaces. If some steps or procedures were not performed as specified by delivery order requirements, the reasons must be stated on the appropriate reporting form or submitted as an attachment thereto. All laboratory worksheets shall provide objective evidence that the data has been checked by qualified personnel other than those performing the tests.

8. Sample Storage and Shipment

Any remaining portions of samples shall be retained until direction for disposal is received from Jacobs. Boxing and shipping will be the responsibility of the Subcontractor. Shipping charges when authorized by Jacobs, will be billable to Jacobs. Shipping charges will be paid at cost only.

Once testing of samples are completed they shall be re-sealed and stored for up to a period of six months, at which time the TR shall be notified before the samples are disposed of.

9. Health and Safety Requirements

Some of the samples received will be uranium mill tailings. These samples shall be handled in accordance with OSHA, DOT and any other applicable safety standards, such that contamination of equipment and personnel is minimized.

Uranium mill tailings are a low level hazardous waste.

PRELIMINARY DRAFT

D

GROUND SETTLEMENT

1.0 INTRODUCTION

Ground settlement at UMTRA sites will be assessed in order to evaluate the long term stability of the tailings piles. Settlement, especially differential settlement, can lead to flow concentrations during rainfall runoff events. If these flow concentrations exceed those assumed during design, erosion of the pile cover can occur. In addition, severe differential settlement could lead to cracking within the radon cover.

Assessment of both total and differential settlements are somewhat imprecise but are indicative of the performance that can be anticipated at the piles. Settlements occurring during placement of contaminated soils will have no effect on the life of the radon barrier or rock erosion layers.

This section has only touched the surface of the settlement problem. To be effective as a design guide, this section needs to be further expanded ~~providing~~ in the manner presented in MKE ~~DDD~~ UMTRA Design Procedures, Chapter 7.

Document Title: GROUND SETTLEMENT
Doc. Originated by: TAC (Jacobs - Weston)
Marginal Review Comments by: Nani G. Banerjee, MKE
Member, Workgroup 2

2.0 DATA COLLECTION

Details of data which will be collected are described in the geotechnical site characterization position paper. As indicated in that paper the level of data collected depends upon the previous data collected at a site. In addition site conditions as indicated by field explorations will be considered in the assignment of laboratory tests. Specific areas of consideration include:

- One dimensional consolidation tests on all major material types encountered.
- Saturated and unsaturated tests as indicated by field data.
- Use of pore pressure decay from piezocone data.
- Actual input parameters used in the analysis will vary depending on actual site conditions. Engineering judgement will be used to assess t these conditions and determine appropriate input parameters.

3.0 ANALYSIS

3.1 MATERIALS AND METHODS

3.1.1 Nonplastic soils

Not of great significance relating to the performance of the structure.

Nonplastic soils will be analyzed using elastic theory. The method presented in NAVFAC DM7.1-211 and Figure 7 (Attachment A). The modulus of elasticity will be developed using cone penetration and/or standard penetration test data. Where possible this value shall be checked against the modulus of elasticity developed from the triaxial compression tests. Also, if appropriate, one-dimensionsal consolidation tests will be performed and conventional consolidation theory will be used to calculate the settlement using actual strains measured from the test. Settlement of nonplastic soils will be assumed instantaneous or to occur during construction.

3.1.2 Plastic soils (clay)

Plastic soils will be analyzed using conventional consolidated theory and laboratory one-dimensional consolidation tests conducted on undisturbed soils. Where data is lacking or as a check of the consolidation test data, consolidation properties derived from literature and mechanical properties correlations will be used.

Conventional consolidation theory is used in conjunction with the following equation (Lambe and Whitman, 1969):

$$S_c = \frac{C_c H \text{ Log}}{1 t e_o} \quad \frac{P_o + P}{P_o}$$

S_c = consolidation settlement (feet)

C_c = compression index

H = height of layer (feet)

e_o = initial void ratio

P_o = initial effective stress at midpoint of layer (psf)

P = change in effective stress at midpoint of layer (psf)

. Need to include (method of computation of) more complex settlement (and rate of settlement problems) such as layered system, with one-way or two-way drainage, recommended remedial measures, etc.

Secondary or long term settlement, where considered significant will be analyzed. Time-rates of consolidation will be determined from field and laboratory tests and used to estimate the length of time for settlement to occur where cohesive soils are saturated.

3.1.3 Cover cracking

Should not the settlement cracks be modelled, and of loss of effectiveness of the cover be checked for acceptability?

When appropriate cracking of the radon cover due to differential settlement of the underlying soils will be calculated using methods developed by Lee and Shen (1969) Attachment A. Typically these methods will only be applied at stabilization in place sites where large localized differential settlements are likely to occur.

3.2 APPLICATION

3.2.1 Stabilization in place

Soil Profile?

Where stabilization in place is being considered the site stratigraphy will be combined with determined material properties and a contour plot of resultant settlements will be developed. Depending on the results of this analysis, and the maximum differential settlement obtained from this plot, the potential for cover cracking may be analyzed. If appropriate, long term settlement contour maps will also be prepared.

3.2.2 Relocated piles

Where piles are to be relocated settlement of the foundation and the pile itself will be calculated. It is assumed that since the tailings will be blended and compacted, that differential settlement will not be critical unless foundation soils are extremely variable. Also settlement of the pile will be considered nearly instantaneous since the tailings will be unsaturated.

3.3 FINAL CONDITION

The large scale of the tailings embankments, complexity of the subsurface stratigraphy within the tailings piles and foundation soils and the limited data available from which to derive design parameters make the prediction of total and differential settlements inexact at best. In order to reduce the uncertainty and raise the reliability of long-term stabilization, several construction related design features will be required:

- Monitoring of embankment settlement for completion prior to placement of a majority of the radon cover material. This will lessen the chance of cover cracking due to sharp differential settlement.

- Monitoring for completion of settlement after placing and before final grading of the radon cover. This will lessen the chance of unanticipated flow concentrations of storm water runoff.

- Placement of cover material at 2 to 3 percent above the optimum moisture content, this makes the material more pliable and less likely to experience cracking due to settlement.

Provide details of type of settlement gages ~~you plan to~~ recommend for installation, their program of installation, frequency of measurement. According to 40 CFR Part 192, these control measures will have to be discussed also with the NRC.

Lambe and Whitman, 1969. Soil Mechanics, Massachusetts Institute of Technology, Boston, Massachusetts.

Lee, K.L., and C.K. Shen, 1969, "Horizontal Movements Related to Subsidence," Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 95, SM.1, pp. 138-166.

NAVFAC (U.S. Department of Navy, Naval Facilities Command), 1982. DM - 7.1, Alexandria, Virginia.

ATTACHMENT A

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

HORIZONTAL MOVEMENTS RELATED TO SUBSIDENCE

By Kenneth L. Lee,[1] A. M. ASCE, and C. K. Shen,[2] A. M. ASCE

INTRODUCTION

Considerable progress has been made in the techniques of settlement calculations since Terzaghi introduced his theory of consolidation to the profession over 40 yr ago. On present day jobs engineers routinely proceed with some confidence to predict the amount of total and differential settlements likely to develop at a particular site. During recent years there has been an ever increasing recognition and concern for horizontal movements that are often observed to develop in conjunction with vertical subsidence. Many structures are more susceptible to damage from differential horizontal movements than from differential settlements, and examples of horizontal movement measurements and structural distress due to these movements are being reported in ever increasing numbers. The nature and causes of settlement induced horizontal movements are understood in a general way. However, methods of predicting horizontal movements are still very much in their infancy.

This study was conducted in order to better understand and define some of the mechanisms by which horizontal movements can accompany subsidence, and to investigate some techniques for predicting the nature and magnitude of the total and differential horizontal movements that are likely to develop under given conditions. Examples of a number of actual cases are briefly reviewed to illustrate the extent and nature of the problem. Analytical methods and experimental studies were used to investigate the basic mechanism of horizontal movements and to check hypotheses for predicting the magnitude of horizontal movements likely to develop under particular conditions. The study concludes with an example of the predicted nature, distribution, and magnitude of hori-

Note.—Discussion open until June 1, 1969. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 94, No. SM6, January, 1968. Manuscript was submitted for review for possible publication on January 25, 1968.

[1] Asst. Prof. of Engrg., Univ. of Calif., Los Angeles, Calif.
[2] Asst. Prof. of Engrg., Univ. of Calif., Davis, Calif.

zontal movements likely to develop in an earth dam constructed on a compressible foundation.

EXAMPLES OF HORIZONTAL MOVEMENTS

Some of the earliest recognized examples of horizontal movements related to subsidence were associated with mining areas. In some mining operations the mine tunnels are allowed to collapse after the ore has been removed. Although some arching may occur, a considerable amount of total and differential subsidence is often observed at the ground surface. Furthermore, it has been found that horizontal movements inevitably accompany the ground subsidence. When this type of mining operation is employed, structures on the ground surface may be subjected to considerable differential vertical and horizontal movements. Many structures are more sensitive to horizontal than to vertical movements, and mining engineers have made many studies of the nature of both horizontal and vertical surface movements caused by mining operations (8, 11, 16, 18, 20, 23, 25, 37).[3]

Another type of serious horizontal movement is related to ground subsidence caused by pumping oil, gas or water from underground natural reservoirs. The best known and best documented example of this phenomena has occurred at Long Beach, Calif., which is founded above the Wilmington oil field. Pumping has produced a settlement basin encompassing 50 sq miles with settlements at the center of approximately 29 ft. The slope of the subsidence profile toward the center of the basin has been found to be as much as 0.25%. The development of this basin has been accompanied by horizontal movements as great as 6 ft and differential horizontal movements or surface strains as great as 0.35%. The damage to structures in the Long Beach Harbor area resulting from the movements associated with this subsidence basin has exceeded $60,000,000 (1, 18, 39, 40). Several studies of the basic nature of these movements were made by Grant and others. They suggested that the horizontal movements could be explained by assuming that the ground above the oil-bearing sands deformed like a thick plate to conform to the subsidence which developed as the oil was removed (12, 13, 14), and other theories have also been suggested (29).

Although Long Beach is perhaps the best known example, subsidence and horizontal movements are known to have developed over other oil fields (40). In Los Angeles, the Baldwin Hills dam was located on the edge of the subsidence zone caused by pumping oil from the Inglewood oil field. Settlement as great as 9 ft, and horizontal movements exceeding 2 ft have been measured over the lifetime of the oil field. The dam was unfavorably situated on the edge of the subsidence area where maximum horizontal extension strains were likely to develop. The catastrophic failure of this dam in 1963 was probably related to these movements (17, 21, 22).

An example of a subsidence and horizontal movement problem associated with water pumping has been reported at Houston, Texas where a 30-mile diameter subsidence bowl has developed. The maximum settlement is reported to be about 3.5 ft and absolute horizontal movements as much as 16 in. have been measured (7,26).

There are numerous other areas in the world where pumping has caused large settlements such as at Lake Maracibo, Venezuela; Mexico City, (5, 27, 45); Tokyo (30); and California's Central and Santa Clara valleys (34, 36). However, horizontal movements at these areas have not been reported, possibly because they are relatively small or occur in areas where the buildings and other structures are relatively flexible, and therefore horizontal movements have not been measured or observed. However, the examples of cases where horizontal movements are known to occur illustrate the serious consequences of this type of deformation.

[3] Numerals in parentheses refer to corresponding items in Appendix I.—References.

Another example of horizontal and vertical movements occurred where a large industrial plant in Northern British Columbia was founded on 30 ft of fill overlying a 100-ft layer of gravel which in turn rested on a layer of deep compressible clay. The maximum settlement was about 4-1/2 ft and the resulting slope of the subsidence bowl was as much as 0.5%. However, the most serious damage to some of the industrial buildings was due to horizontal movements which in some areas caused buildings which were 1000 ft long to shorten as much as 6 in. This net movement was made up of stretching in some sections and compression in others (15).

Davis and Taylor (6) have also examined the problem of horizontal movements which may develop as a result of placing large fills on compressible soils.

It has been known for some time that horizontal movements may cause cracks to develop in earth dams as the fill settles (34, 38). Many examples have been reported which illustrate the widespread nature of this problem (10, 28, 32, 33, 35, 41). The settlement may be due to an underlying compressible foundation or partial collapse of the embankment soil upon wetting when the reservoir is filled. In most cases where observations have been made it has been found that differential vertical settlements along the crest of the dam were accompanied by horizontal movements directed toward the zone of maximum settlement. These horizontal movements create a favorable situation for the formation of transverse tension cracks near the abutments, and many such examples have been reported.

Cappleman (2) describes still another closely related problem. Horizontal movements due to differential settlement of earth dams on compressible foundations have lead to a number of conduit failures in which the joints have been pulled completely open.

A study of the available data regarding examples of ground movements suggests that there are two fundamental factors which are important in the problem of horizontal movements related to ground subsidence:

1. The seat of settlement must be located at some depth below the ground surface.

2. There must be differential vertical movement with the gross pattern of settlement approximating a subsidence bowl or subsidence trough in which the settlements increase from zero at the outer edges to a maximum at the center.

Both of these features appear to have been present in every case in which horizontal movements were observed in subsiding areas.

Rellensman (37), Grant (1, 13, 18), and others have observed certain correlations between the horizontal movements and other features of subsiding areas. The correlations, along with the two factors mentioned are illustrated

Ripley, and Lee (15) found a reasonably good correlation between observed movements and those predicted by Eq. 1 using $h = H$, the thickness of the upper stiff gravel layer.

The assumptions of the simplified beam theory which lead to Eq. 1 neglect the effect of shearing stresses, and these may be important especially for beams with deep sections. For example, if the beam shown in Fig. 2 were to be deformed under a state of pure horizontal and vertical shear, and section AB would remain vertical, and the horizontal movement at point A would = 0. Thus the inclusion of a shear stress would lead to a computed value for horizontal movement somewhat less than given by Eq. 1. In addition, for most real soils the modulus of deformation increases with confining pressure, and will therefore increase with depth below the surface.

The addition of these factors to the analysis will in effect raise the neutral axis and result in computed horizontal movements somewhat less than given by Eq. 1 for $h = H$, the thickness of the upper stiff stratum. Although it may be difficult to account for these two factors by a precise analytical method,

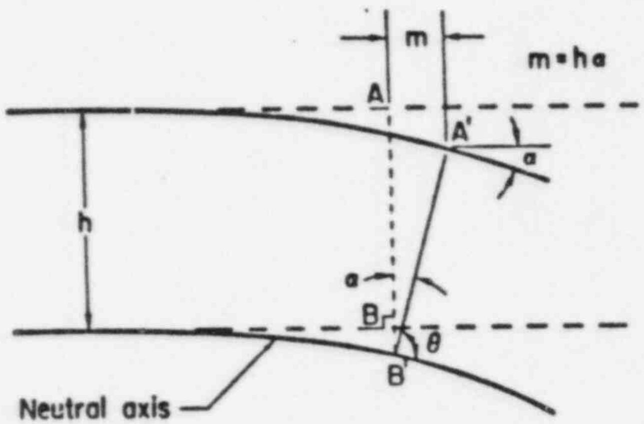

FIG. 2.—BEAM ANALOGY OF HORIZONTAL MOVEMENTS

it seems reasonable to expect that a more realistic expression for the horizontal movement of a surface point A shown in Fig. 1 would be

$$m = kHa \qquad (2)$$

in which k = a factor to correct Eq. 1 for the effect of shear and a variable modulus. For the ideal assumptions of the simple beam theory and constant modulus $k = 1.0$ and for pure shear $k = 0$. These probably represent extreme limiting values so that for any particular case k would lay somewhere inbetween, possibly closer to 1.0 than to 0.

Although Eq. 2 may be considered as only an empirical expression, nevertheless, its use with any reasonable value of k between zero and one will lead to predicted values of horizontal movements of the same general form as those illustrated in Fig. 1. Therefore, provided that the settlement profile can be predicted with some confidence, Eq. 2 could be useful in predicting at least the nature of horizontal movements likely to develop at a particular site.

Therefore, it is of interest to investigate the type of horizontal movements which do develop at subsiding areas in order to establish guide lines as to the reliability of Eq. 2, and to obtain reasonable values for the factor k.

MODEL STUDIES

To study in some detail the development of horizontal movements due to subsidence of a relatively stiff layer of soil, tests were concluded using a model (Fig. 3) consisting of a 93 in. × 24 in. × 6 in. wide beam of granular soil confined in a wooden box. The bottom of the box was adjustable so that any desired subsidence profile could be imposed. The object of these model tests was to induce certain predetermined settlement profiles at the bottom of the soil beam, and to measure the resulting vertical and horizontal movements developed at the surface. These measured movements were then compared to theoretically predicted values.

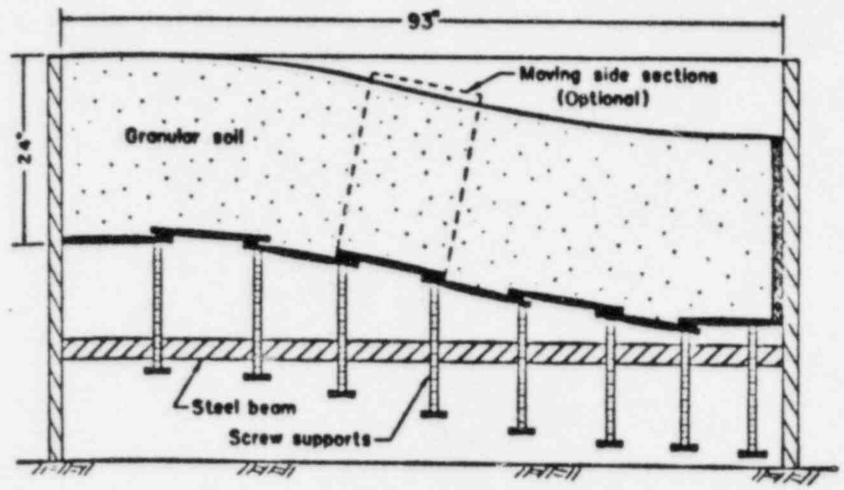

FIG. 3.—SCHEMATIC DRAWING OF MODEL TEST APPARATUS

There was some concern that friction between the soil and the wooden walls of the box might lead to arching and thus retard the free movement of the soil in the beam. To overcome this difficulty thin wooden movable sections of 1/8-in. thick masonite were placed between the soil and the fixed wooden sides of the box. These thin sections were fixed to the adjustable base sections of the box so that they tilted in direct proportion to the induced differential settlement as shown in Fig. 3. Thus, using the movable sides, arching between the sides and the soil was expected to induce excessive horizontal movements, whereas the effect of arching without the movable sides was expected to result in horizontal movements which were too small. Therefore, two tests were performed for each deformation condition; one with moving sides and one with no sides. The data from the two tests were averaged in each case, and are believed to be sufficiently accurate for the purpose intended.

Based on the summary of previous field data shown in Fig. 1 it was assumed

in Fig. 1. In this figure, the seat of settlement is located at a depth H below the surface, and the settlement profile through the loaded area has the typical saucer shape with zero settlement at the outer edges, and a maximum settlement in the center. The horizontal movements and horizontal strains likely to develop at different points along this subsidence profile are also shown. These movement curves are related to each other as follows:

1. There is no horizontal movement at the point of maximum settlement nor are there any horizontal movements at considerable distances beyond the subsidence zone.

2. At each location the direction of the horizontal movement is toward the point of maximum settlement.

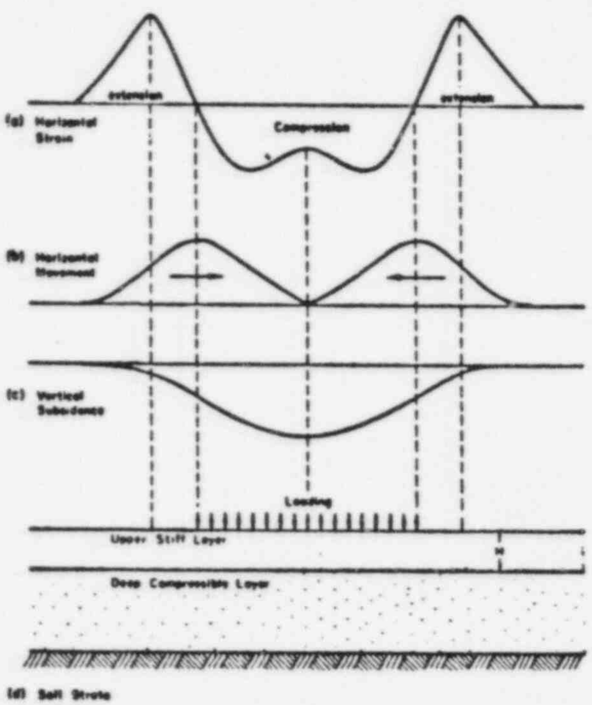

FIG. 1.—IDEALIZED SURFACE MOVEMENTS AT A SUBSIDING AREA

3. The point of maximum horizontal movement corresponds to the point of steepest slope of the vertical subsidence profile and the horizontal strain at this point is zero.

4. The horizontal strain over much of the central part of the subsidence area is compressive, and extension strains develop near the outer edges of the subsidence zone.

5. The point of maximum horizontal strain is located at the point of steepest slope of the horizontal movement curve.

The shapes and relative positions of the movement curves shown in Fig

agree with the observed data from a large number of field cases, as well as with the other theories presented herein.

Note that for the idealized foundation and loading conditions illustrated in Fig. 1, the point of maximum absolute horizontal movement coinsides roughly with the edge of the loaded area, and the point of maximum horizontal extension strain lies somewhat outside of the area of applied surface loading. For more complicated foundation and loading conditions such as an earth dam in a rock gorge, etc., the details of horizontal movements may be somewhat altered from those illustrated in Fig. 1, but the available data strongly suggest that the general pattern of these movements is the same in all cases.

HORIZONTAL MOVEMENT PREDICTIONS

A number of attempts have been made to predict on a theoretical or semitheoretical basis the magnitudes of horizontal movements likely to develop at a particular site (19, 25, 37). The theories developed for mine subsidence have stressed the importance of the thickness, H, of the upper stiff layer which does not participate directly in the subsidence, but simply moves down as the lower layers compress. Grant and his colleagues (1, 13, 18) have also recognized the importance of the thickness of the upper stiff layer, and have suggested that this material behaves like a thick plate which bends to conform to the vertical movements necessary to maintain continuity with the lower layers as they compress. In a vertical section through the subsiding area the stiff upper layer resembles a continuous beam.

This beam analogy is illustrated in Fig. 2. Line AB represents a vertical transverse section in the beam before any subsidence takes place. After some subsidence has occurred this line will have moved to a new position A'B' resulting in some horizontal movement of point A on the surface. According to one of the fundamental assumptions in the simplified beam theory, plane sections before bending remain plain after bending. From this assumption it follows that for a constant modulus of elasticity, the line A'B' should be straight, the angle, θ, should be 90°, and the slope, α, at the surface should be the same as the slope of the neutral axis. The horizontal movement, m, of the point A on the surface would then be defined by

$$m = h\alpha \quad (1)$$

in which h = the depth to the neutral axis, and α = the slope of the subsidence profile at the point.

In applying these concepts to the subsidence problem at Long Beach, Gilluly and Grant (11, 13) proposed that the neutral axis would be in the center of the stiff upper layer, at depth $H/2$ from the surface. This is the case for a normal symmetrical beam of thickness H, and there was some field evidence in the form of measured depths to sheared oil well casings to indicate that it was also a reasonable approximation for the field. However, this assumption was criticized by McCann and Wilts (29) who pointed out that a neutral axis at mid-depth would require horizontal movements at the bottom of the stiff layer which were equal and opposite to the horizontal movements observed at the top, and they reasoned that friction at the contact zone would probably prevent this from occurring. In an analysis of the horizontal movements at an industrial site founded on a 100-ft layer of gravel overlying compressible clay, Hardy,

that symmetrical boundary conditions would lead to a symmetrical distributio of movements, and that all points on the center line would move verticall down. On this basis it was decided to model only one-half of a symmetrical subsidence profile. The maximum vertical subsidence which would really occur in the center of the profile was modeled at one end of the beam as shown i Fig. 3.

Two types of granular soil were used for the model tests; a clean unifor medium-grained quartz sand, and expanded styrofoam. The styrofoam wa in the form of uniform spheres about 1 mm in diam. In a dense condition th unit weight was only about 17 lb per cu ft. The test results reported here wer obtained with this material because it was easier to handle than the sand, an also because of its low density it was felt that side friction effects would b

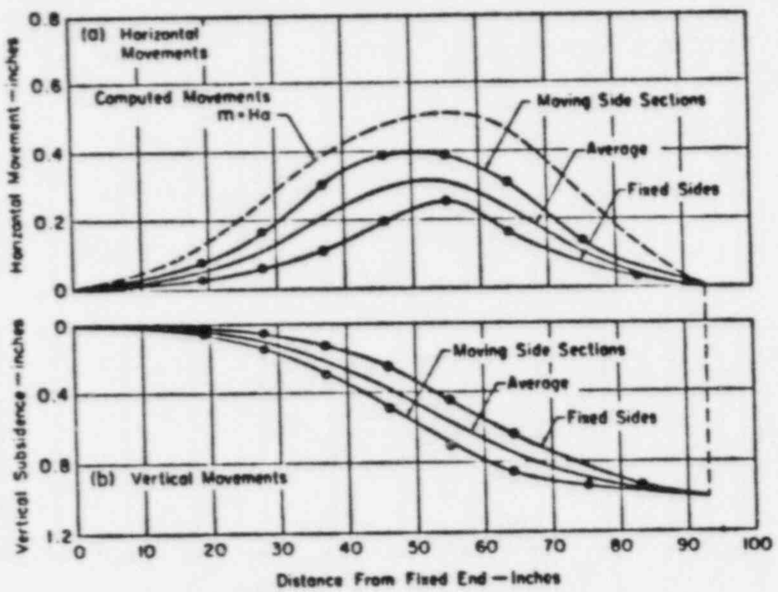

FIG. 4.—TYPICAL MEASURED MOVEMENTS AT THE SURFACE OF MODEL BEA

minimized. However, a limited number of tests on the quartz sand gave simil results.

To prepare the model for testing, the soil was placed in thin layers in t wooden box and each layer was vibrated with a pneumatic vibrator until reached approximately 100% relative density. Small markers were embedd in the surface of the soil and their positions accurately determined with r spect to a fixed reference. The bottom sections of the box were supported adjustable screws, and the test was performed in a series of stages. At ea stage of the bottom supports were simultaneously lowered a small dista in proportion to its ultimate movement. After the final positions were attaine the vertical and horizontal positions of the surface markers were again a curately measured.

To be assured of reasonably accurate measurements, it was necessary use magnitudes of movements somewhat greater than might be expected i

typical field situation. The smallest total vertical subsidence used corresponded to 1 in. at the end of the beam. This represented an average slope over the 93-in. length of about 1.0%, and a maximum slope of the subsidence profile of 2.0%. Other tests were performed using a maximum vertical movement at the end of the beam of 2 in. and 3 in. respectively. These were somewhat greater than the magnitude of differential movements which have been observed to cause damage to structures (9) or earth dams (24). However, it was felt that if consistant correlations could be made between observed and predicted move-

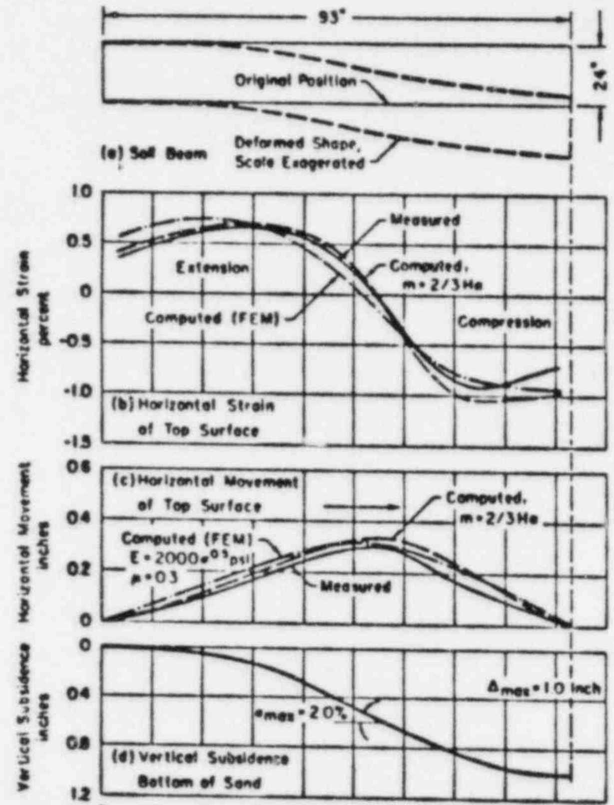

FIG. 5.—MEASURED AND COMPUTED HORIZONTAL MOVEMENTS IN MODEL BEAM

ments for several cases, the results could be extrapolated back to the smaller field movements.

The results of a typical test are shown in Fig. 4. The amount of arching indicated by the magnitude of the difference in surface movements measured in tests conducted with and without movable side plates was small. The average of the observed surface vertical movements for these two cases was, within the accuracy of the measurements, almost identical to the vertical movements imposed at the bottom. The nature of the horizontal movements is identical to

that shown in Fig. 1, and, therefore, it was felt that the average movements from the two tests were sufficiently accurate for comparison with predicted values. The measured values of horizontal movements are compared with the movements predicted from Eq. 1 for the case of $h = H$ shown by the dashed line in Fig. 4. This case corresponds to no-slippage and therefore a neutral axis at the bottom of the beam, which might logically be expected to apply for these tests because of friction between the soil and the wooden bottom of the box. However, it is clear that Eq. 1 with $h = H$ predicts excessively large horizontal movements, even compared to the case with moving sides where arching would lead to excessively large horizontal surface movements.

The average of these test data is reproduced in Fig. 5 along with the average horizontal strains determined from this data. The nature of the measured data is similar to that shown in Fig. 1, which represents the form of movements observed from many field cases.

An attempt was made to fit Eq. 2 with the average values of observed horizontal movement. As shown in Fig. 5 it was found that the observed movements were very closely represented by Eq. 2 using a value of $k = 2/3$, i.e.,

$$m = \frac{2}{3} H\alpha \quad \text{(3)}$$

THEORETICAL STUDIES BY THE FINITE-ELEMENT METHOD (FEM)

The finite-element method of analysis is a powerful analytical method of extreme generality which has been applied to a variety of problems in soil mechanics. Clough (3) and Wilson (43) describe the method and its formulation for use with high speed digital computers, and an excellent example of its use for soil mechanics problems has been described by Clough and Woodward (4).

The values of soil modulus used in the finite-element analysis were determined by a series of triaxial tests on the sand used in the model. The soil was compacted to 100% relative density corresponding to the density to which it was compacted in the model beam, and tested at confining pressure ranging from 5 psi to 25 psi. The modulus values, defined as the slope of the initial tangent to the stress strain curve, were found to be related to the confining pressure by

$$E = 2000\sigma^{0.5} \quad \text{(4)}$$

in which E = the modulus value in pounds per square inch and σ = the value of the confining pressure in pounds per square inch. This relationship is similar to those determined for other granular soils and it was assumed that a relationship of this same form could also be used to represent the modulus of the styrofoam. The analytical results indicated that the computed horizontal movements were not particularly sensitive to modulus. Therefore, Eq. 4 was used to calculate the modulus for various depths below the surface of the model beam. For these calculations the confining pressure was taken as one-half of the vertical overburden pressure. Thus, the value of E used in the analysis increased in direct proportion to the square root of the depth. For most of the studies the value of Poisson's ratio was assumed equal to 0.3, which agrees with measured values from other granular soils.

The boundary conditions used in the FEM studies were as follows: (1) All points on the bottom surface moved vertically down a prescribed amount to

conform to the imposed subsidence profile of the test; (2) all points on each end were prevented from any horizontal movements, but were free to move vertically; and (3) all points on the surface and in the interior were given no restrictions. No external loads or body forces were applied. The only source of deformations was the induced subsidence at the bottom of the beam.

For most of the analyses the beam was divided into 248 elements each 3 in. × 3 in. square. However, analyses performed using twice as many elements gave results which were the same, indicating that the computed values of stress, strain, and displacement were not influenced by the finite-element representation chosen for the analyses.

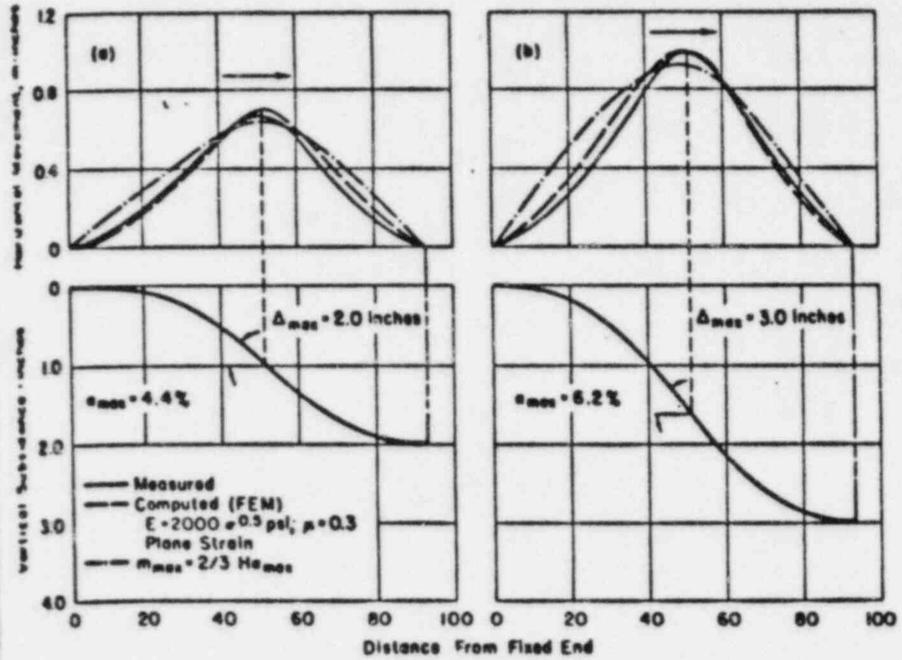

FIG. 6.—MEASURED AND COMPUTED SURFACE HORIZONTAL MOVEMENTS

The horizontal surface movements and horizontal strains computed by the FEM for a maximum 1-in. vertical deflection are shown in Fig. 5. It may be noted that these movements are remarkably similar to the measured movements, and to those computed by Eq. 3. Similar comparisons for the data from the tests for 2-in. and 3-in. maximum deflections are shown in Fig. 6. Again there is good agreement between the measured values and the two methods of computation.

EFFECT OF DIFFERENT ELASTIC PROPERTIES

The good agreement between the FEM and the experimental data suggested that the FEM would be useful in investigating other more complicated situa-

tions. Before doing so, however, it was desirable to determine the effect of variations in assumed elastic properties on the FEM results. Accordingly, a number of FEM studies were made for the model beam using the 1-in. maximum subsidence profile, but with different assumed elastic properties for the granular soil.

The first step was to perform a number of computer analyses using a constant value of E throughout the entire beam as well as a number of studies for a modulus which varied with confining pressure. A summary of these analyses is presented in Fig. 7. Identical movements were computed for all constant values of modulus. Similarly, identical movements were predicted for all analyses in which E increases proportionally to the square root of the depth. However, the movements predicted using a constant E were slightly less than those predicted using values of E which increase with depth.

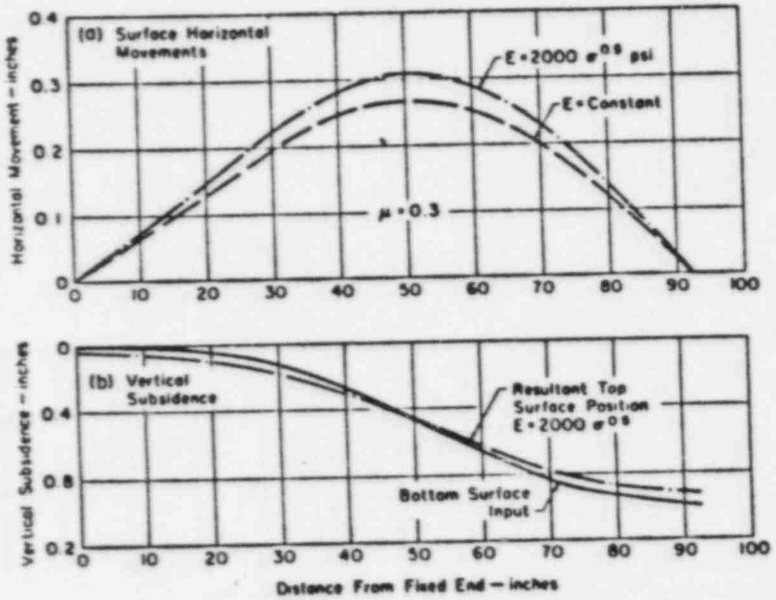

FIG. 7.—EFFECT OF SOIL MODULUS

The second step was to investigate the effect of using different values of Poisson's ratio in the analysis. The results of analyses using three different Poisson's ratios are shown in Fig. 8. A summary of the effect on the maximum horizontal movements for different values of Poisson's ratio and for two methods of specifying modulus is presented in Fig. 9. These data show that Poisson's ratio has only a small effect. The lower values of Poisson's ratio result in the largest values of horizontal movement.

Eq. 1 was derived on the basis of a fundamental assumption in the derivation of classical beam theory that plane sectors remain plane during bending and that there was zero movement of the neutral axis. The data obtained from the model studies, and confirmed by the FEM analyses in which the boundary conditions fixed the neutral axis at the base of the beam, indicated that the

equation should be slightly modified, in favor of an expression such as Eq. 2. The results of the model study and the FEM analysis both suggest that the constant, k, in Eq. 2 may be of the order of 2/3, and therefore Eq. 3 presents a more accurate representation of the horizontal movements which develop at the surface of a subsiding layer.

Because the FEM gives movements at all points within the body, and not just those at the surface, it was of interest to use data obtained by this method to study the variation of movements along an originally vertical plane section. The movements along a typical section computed by various methods are shown in Fig. 10. The horizontal movement at the bottom of the beam = 0, because this was the boundary condition employed in both the model studies and the finite-element analysis. The analytical results from the FEM of analysis using either method of specifying modulus indicate that originally plane

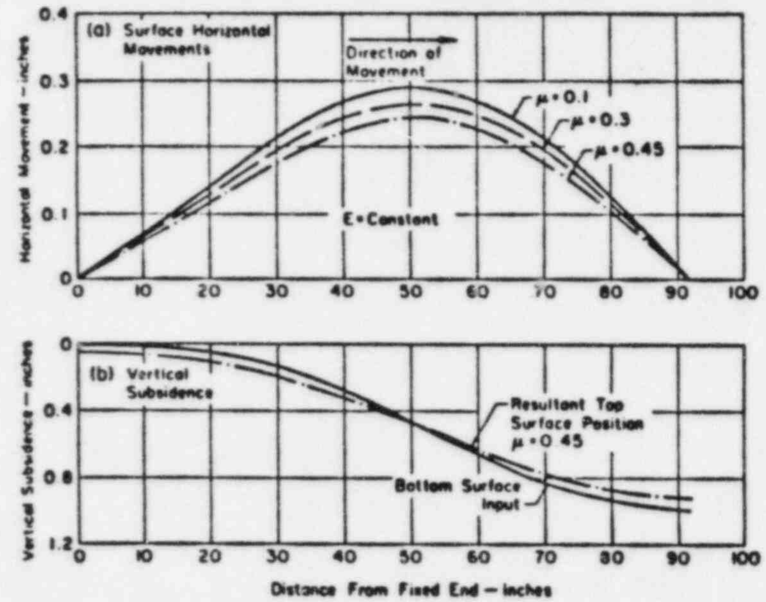

FIG. 8.—EFFECT OF POISSON'S RATIO

sections become slightly curved during the deformation. The angle θ at the neutral axis does not appear to remain at 90° during bending as is assumed by the simple beam theory, indicating an appreciable influence of shear on the resulting deformations. However, the data in Fig. 10 indicate that throughout the beam the horizontal movements approximate the linear relation suggested by Eq. 3.

THREE-DIMENSIONAL PROBLEM

The model beam experiment produced a plane strain deformation condition, and a plane-strain FEM computer program was used to obtain the comparative analytical values shown in the previous figures. The computer program con-

verts values of modulus and Poisson's ratio to appropriate values for the plane-strain analysis.

However, many field problems are three-dimensional in character. It is, therefore, of interest to investigate the nature of errors that might be involved in using the plane-strain analysis for a nonplane-strain condition. For this purpose a FEM computer program for axisymmetric structures (44) was used, and the results of the axisymmetric analyses were compared with those already considered.

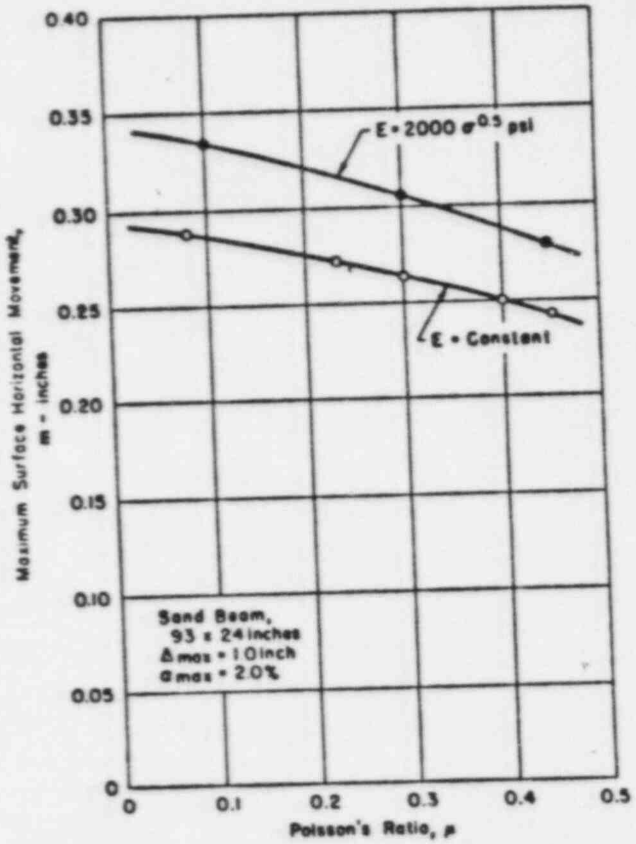

FIG. 9.—EFFECT OF POISSON'S RATIO ON THE COMPUTED MAXIMUM SURFACE HORIZONTAL MOVEMENTS

The difference between plane strain and axisymmetric conditions for the type of deformations being considered here is shown in Fig. 11. In using the axisymmetric program it was assumed that the deformed shape of the model beam represented one radial section of an axisymmetric subsidence bowl. In other words, the shape of the deformed vertical section shown in Fig. 11(b) for the axisymmetric case was made to be identical to the deformed shape of the model beam.

The comparison of the resulting horizontal movements for these two cases

for the same input subsidence profile corresponding to 1 in. maximum deflection is shown in Fig. 12. The axisymmetric problem leads to slightly smaller horizontal movements than determined by plane strain analysis. However, the difference between the movements computed by these two methods is no greater than the difference computed by choosing different values of Poisson's ratio or different distributions of modulus. Most field problems will be somewhere between the extremes represented by the plane strain and the axisymmetric cases. It is doubtful that at the present state-of-the-art the inaccuracies that might be involved in using either solutions will be of great significance. Errors in the choice of the distribution of E, or of Poisson's ratio could lead to as great a difference in the computed horizontal movements as the use of a plane-

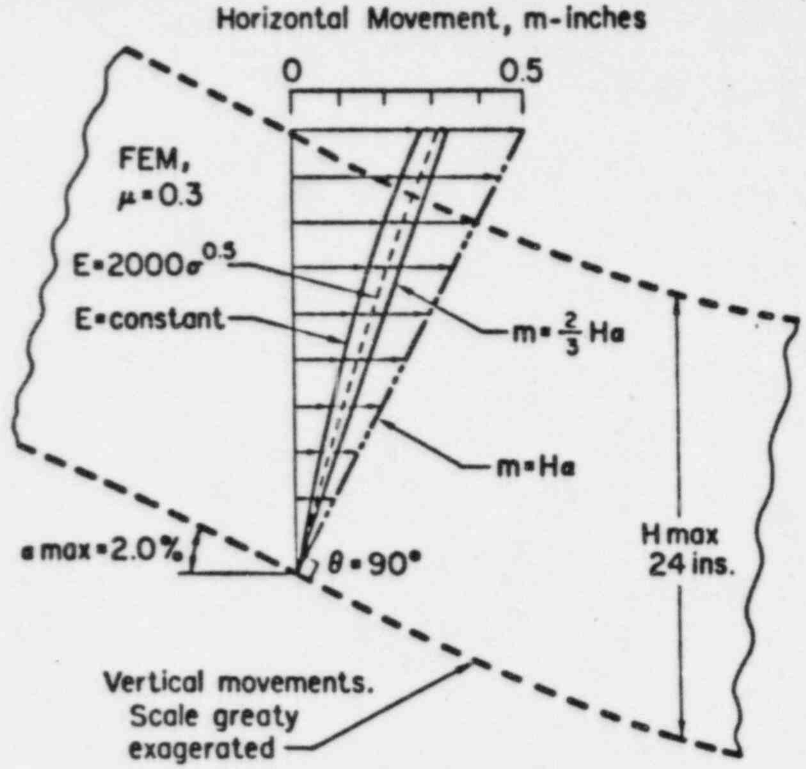

FIG. 10.—DISTRIBUTION OF THEORETICAL HORIZONTAL MOVEMENTS WITHIN SUBSIDING SOIL LAYER AT POINT OF STEEPEST SUBSIDENCE PROFILE

strain FEM program to obtain the horizontal movements for a three-dimensional case. No matter what choice is made for E, μ, or type of deformation the correct nature of movements will be computed, and the values of these computed movements will be of the right order of magnitude. Furthermore, the simple semi-empirical Eq. 3 will also give a good first approximation to both the nature and magnitudes of horizontal movements likely to develop.

In all of these methods the most important factors required are the thickness of the stiff material above the zone of compression, and the vertical

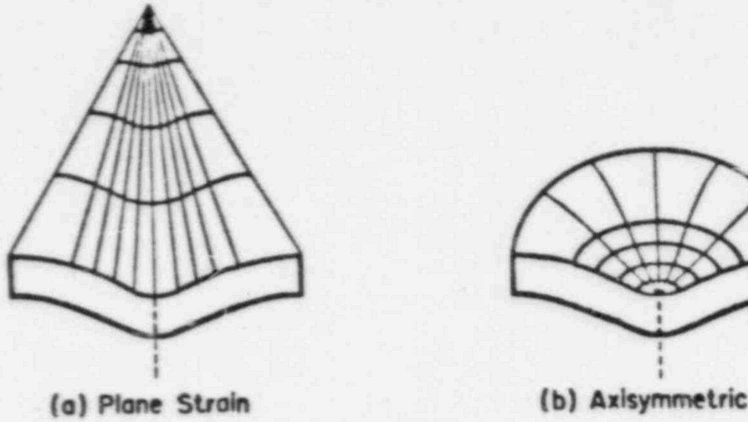

FIG. 11.—PLANE STRAIN AND AXISYMMETRIC DEFORMATIONS

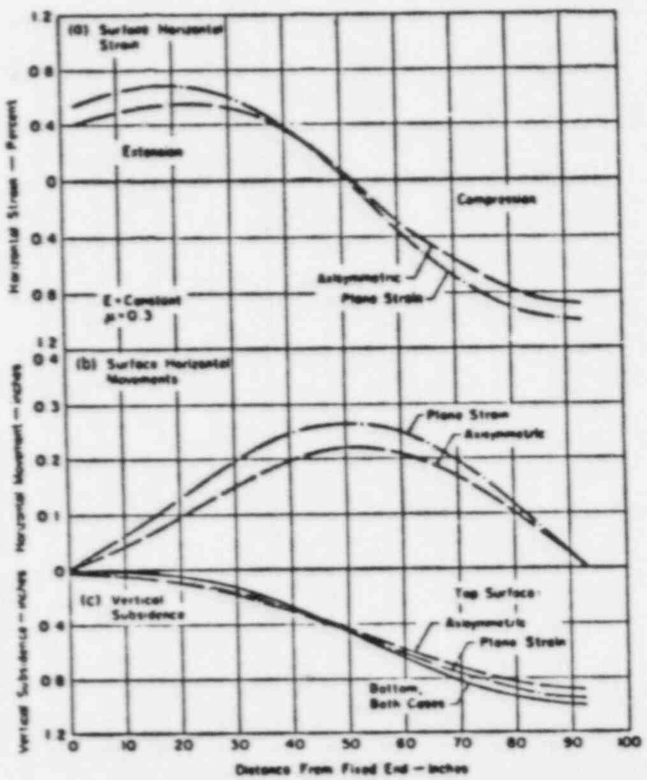

FIG. 12.—EFFECT OF THREE DIMENSIONAL DEFORMATIONS ON THEORETICAL HORIZONTAL MOVEMENTS

subsidence profile. Compared to these two factors, all other refinements are of secondary importance.

DAM ON COMPRESSIBLE FOUNDATION

One area in which horizontal movements are being observed with increasing care is in connection with earth dams. At the 1966 ASCE Slope Stability Conference in Berkeley there were no less than five papers which dealt with some aspects of horizontal movements and cracking of earth embankments associated with differential vertical settlements (10, 28, 32, 33, 35). Sherard et al. (38) and Leonards and Narain (24) describe other examples. Cappleman (2) describes

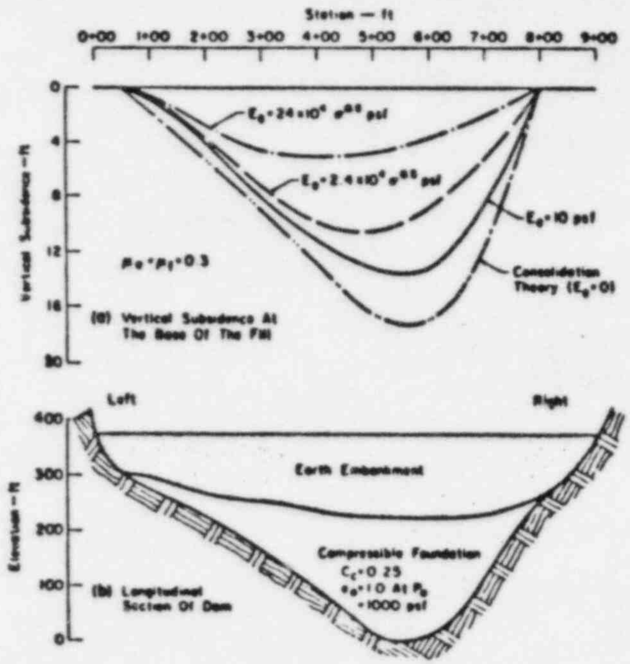

FIG. 13.—INFLUENCE OF FILL STIFFNESS ON THE VERTICAL SUBSIDENCE

examples of 13 dams on compressible foundations where horizontal movements resulting from vertical settlement have seriously damaged pipe conduits running under the dams.

To illustrate the possible applications and limitations of the FEM and Eq. 3 in predicting horizontal movements for earth dams, a hypothetical example of a 125 ft high dam resting on a foundation of compressible clay with a maximum depth of 225 ft was studied. The longitudinal section through this dam and foundation is shown in Fig. 13(*b*). The clay was assumed to be normally consolidated and saturated with a buoyant unit weight of 60 lb per cu ft. It was assumed to have a compressive index C_c = 0.25, and an initial void ratio e_o = 1.0 at an overburden pressure P_o = 1000 psf. The fill for the earth embank-

for the consolidating clay foundation were computed from

$$E = \frac{\Delta P(1 + e_0)}{C_c \log\left(\frac{P_0 + \Delta P}{P_0}\right)} \quad \text{(5)}$$

in which P = the change in applied load and the other terms were assigned the values mentioned previously. Although this definition does not exactly cor-

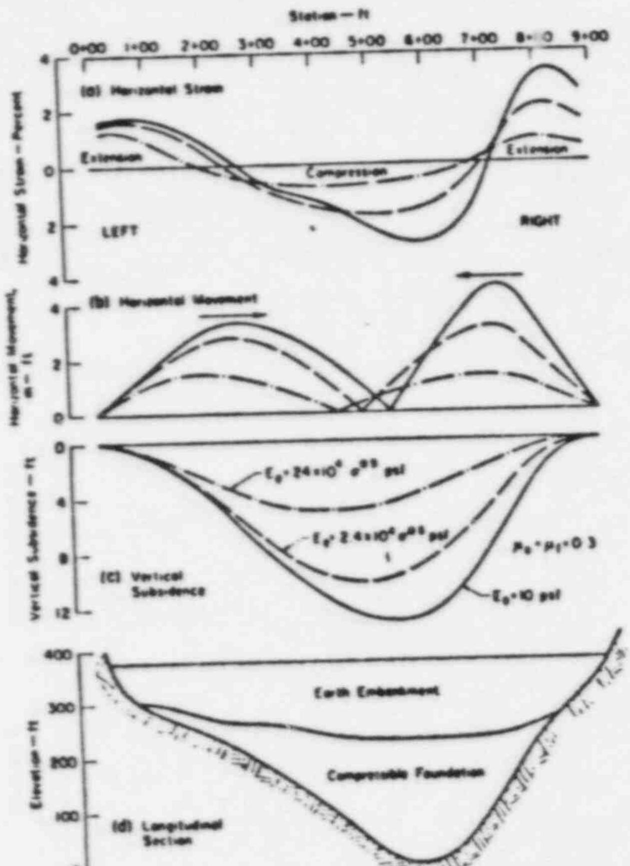

FIG. 14.—INFLUENCE OF FILL STIFFNESS ON HORIZONTAL MOVEMENTS

respond with the classical definition of Young's modulus, nevertheless it wa felt that the values obtained from Eq. 5 were satisfactory for the intende purpose of providing a realistic order of magnitude of the variation of modulu with depth. The values of E determined by Eq. 5 ranged from 11×10^4 ps near the surface of the compressible foundation to 27.5×10^4 psf in the deeper zones.

$$E = 2.4 \times 10^4 \sigma^{0.9} \text{ psf} \quad \text{(6)}$$

in which σ = the confining pressure in lb per sq ft. This relation is the same as Eq. 4 when the units are converted from pounds per square inch to pounds per square foot, and is similar to other data for modulus of compacted granular soil such as granular glacial till. By this equation the value E near the base of the embankment was 230×10^4 psf. The embankment was thus much stiffer than the underlying foundation. A value of Poisson's ratio equal to 0.3 in the embankment and in the foundation was used for most calculations, but the effect of using other values was also investigated.

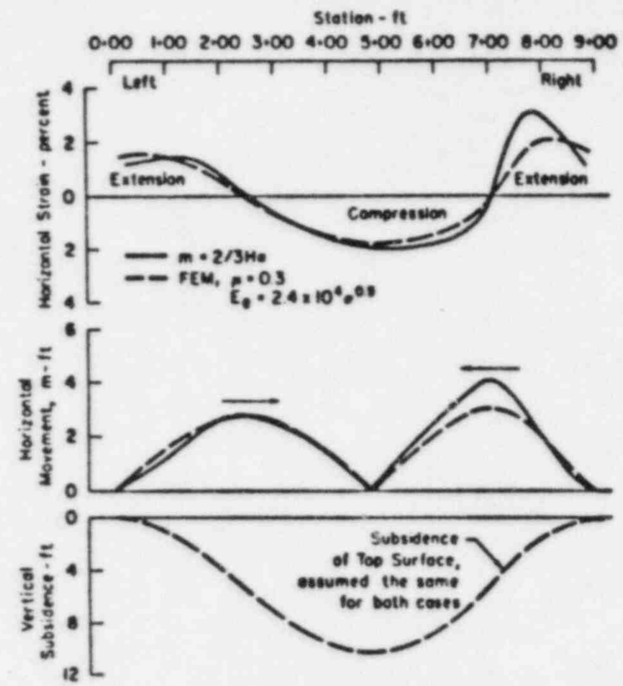

FIG. 15.—COMPARISON OF COMPUTED MOVEMENTS BY APPROXIMATE AND FINITE ELEMENT METHODS

For the sake of consistency and simplification it was assumed that any consolidation within the embankment itself would occur very rapidly in comparison to the settlements due to consolidation in the foundation. This was simulated with the FEM computer program as follows. The unit weight of the foundation clay was made equal to zero so that no body forces acted within this layer. The correct unit weight of the embankment was maintained and the entire fill and foundation were allowed to settle under the weight of the embankment. Most of the vertical settlement was due to compression within the foundation, but a small amount of settlement also occurred within the embankment. To eliminate this, another analysis was conducted, this time with the

unit weight of the embankment also equal to zero, but specifying the vertical deformation of each point at the base of the embankment to be identical to that determined by the previous analysis. The differences in movements at the fill surface for these two cases were small for all cases where the modulus of the embankment E_e was fairly large. However, for consistency this procedure was used in all cases.

Vertical Subsidence of the Hypothetical Dam.—Trollope (42) has pointed out that because of arching, the magnitude and distribution of vertical stresses below earth embankments may not be the same as the weight of the overburden material, and Nonveiller (31) presents an example of arching between the clay core and rock fill of an earth dam. If the vertical stress is affected by arching, the resulting subsidence would also be affected.

Subsidence profiles computed for different values of assumed embankment modulus E_e are shown in Fig. 13(*a*). This data illustrates the influence of the

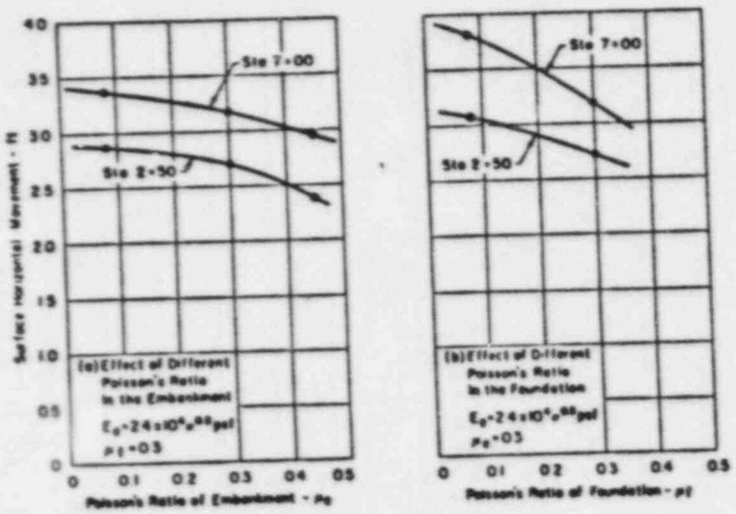

FIG. 16.—EFFECT OF DIFFERENT VALUES OF POISSON'S RATIO ON THE MAXIMUM HORIZONTAL MOVEMENTS

stiff embankment in arching between abutments to reduce the total settlemen[t] The settlement computed by consolidation theory makes no account for th[e] stiffness of the applied fill load. This is equivalent to assuming the modulus [of] the embankment equal to zero. For this relatively narrow valley the settlemen[ts] computed by conventional consolidation techniques are comparatively larg[e]. Subsidence profiles computed for different values of E_e show a progressi[ve] decrease in the amount of settlement for increasing values of embankme[nt] modulus.

Horizontal Movements of the Hypothetical Dam.—The differences in th[e] amount of settlement due to the arching effect were large, and could be signi[f]icant in many cases. One of the aspects in which this difference was significa[nt] was in the computed horizontal movements.

Values of horizontal surface movements and strains determined by the FE[M] for three different embankment stiffnesses are shown in Fig. 14. Note th[at]

the absolute values of these movements depend on the vertical subsidence, which in turn depends on the stiffness of the embankment fill. However, the general nature and trend of these movements is the same for all cases: compression toward the center, extension at the edges, zero movement at the point of maximum subsidence, and zero strain at the point of maximum movement.

It is of interest to examine the accuracy with which Eq. 3 was able to predict the horizontal movements for this earth dam. A comparison of the horizontal movements and strains determined by Eq. 3 and by the FEM for one

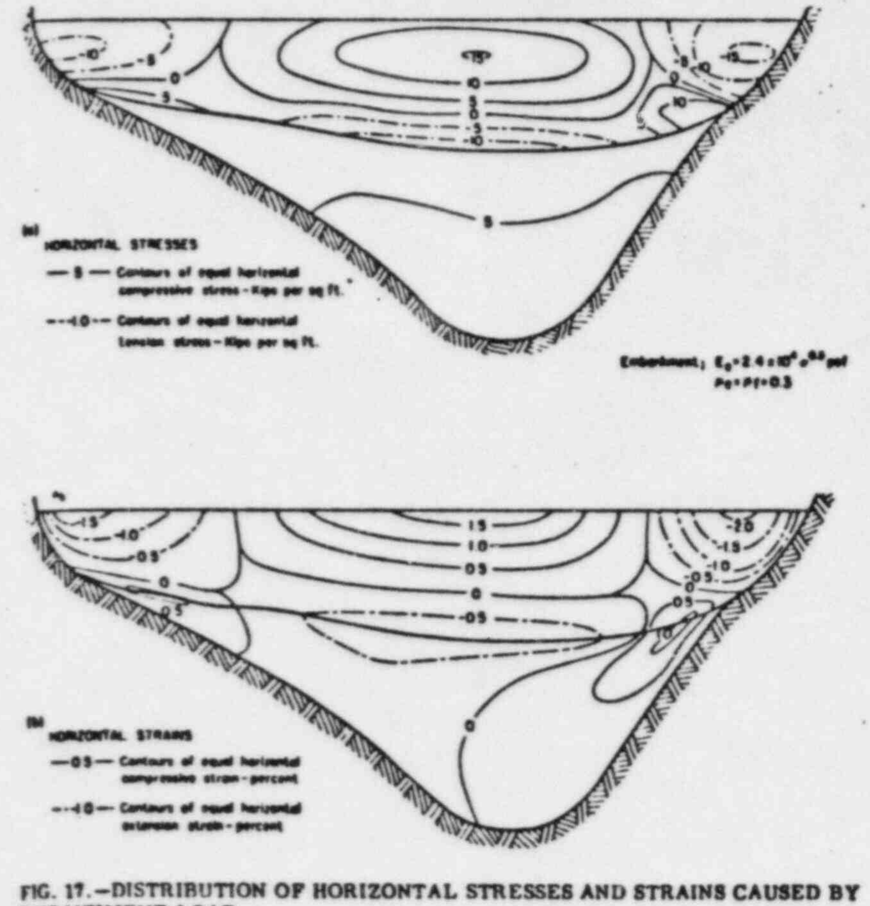

FIG. 17.—DISTRIBUTION OF HORIZONTAL STRESSES AND STRAINS CAUSED BY EMBANKMENT LOAD

value of embankment modulus is shown in Fig. 15. Again, the general nature and trend determined by the two methods are identical. The position of the maximum extension strains, and of the zero strains is predicted to be the same by either method. The absolute values of movements on the left side are also the same for both methods. However, on the right side the FEM predicts movements about 25% less than the movements predicted from Eq. 3.

The foregoing FEM calculations for this dam were made assuming Poisson's

The dam was completed and the reservoir began to fill on June 15, 1964. At this date, the water level in the reservoir was at about el. 80. Immediately the water level in the reservoir began to rise; and by June 23, 1964, it had reached el. 120 as shown in Fig. 18. The vertical subsidence of the surface of the dam and the surface horizontal strains measured between June 15 and June 23 are also shown. Measurements within the embankment fill indicated that during this period most of the vertical subsidence was occurring within the lower portion of the fill which was wet from the rising reservoir water. Thus, this situation is typical of those mentioned previously; i.e., a stiff thick upper layer resting on deep compressible material.

Using Eq. 3, and a value of H equal to the depth to the reservoir on June 23 (or to the rock abutment, as appropriate for the particular section), the absolute horizontal surface movement was calculated for each point across the crest of the dam. From these horizontal movements, the horizontal surface strains were then determined. These computed movements are also shown in Fig. 18 where they can be readily compared with the actual measured movements. Considering that the computed movements were in effect obtained from a second differentiation of the subsidence profile, the close agreement between the measured and computed horizontal strains, both in magnitude and distribution, is very encouraging.

During the period of time that the reservoir was filling to this height, one small transverse crack developed near the left abutment and three developed near the right abutment as shown. Note that the position of these cracks corresponds closely with the position of maximum horizontal extension strain predicted from Eq. 3.

It seems significant that horizontal movements and strains sufficient to cause cracking could develop for a vertical settlement of only 11.5 cm in this 148 m high dam. The maximum vertical strain was only about 0.7% of the height of the embankment and the maximum slope of the subsidence profile was only about 0.1%.

An attempt was also made to check the ability of the FEM to predict the movements which developed during this period of time. For this analysis values of the soil modulus above the water level were computed from

$$E = 530\sigma^{0.5} \text{ kg per sq cm} \quad \text{(7)}$$

in which σ = the confining pressure in kg per sq cm. This relation is the same as Eqs. 4 and 6 when expressed in the appropriate units.

To simulate an increasing compressibility as the water table rose, somewhat lower values of soil modulus were used for all elements below the water table. After a few trials, values of soil modulus were found which gave theoretical vertical settlements of approximately the same order of magnitude as the measured values. The modulus was defined by

$$E = 340\sigma^{0.5} \text{ kg per sq cm} \quad \text{(8)}$$

in which σ = the confining pressure in kg per sq cm computed on an effective stress basis taking the position of the water table into account. Thus, at the same confining pressure the assumed modulus below the water table was only about 65% of the modulus assumed for the dry soil. A value of Poisson's ratio equal to 0.3 was used for all elements of soil above and below the water table.

The FEM calculations were made in two steps. For the first step, the total settlement of the dam was computed for the condition of gravity loads through out the embankment and the water level at el. 80, which was the approximate position at closure on June 15. The same type of calculations were repeated for the second step; this time with the water table at el. 120, corresponding to June 23. The difference in movements computed by these two steps was taken as the movements due to the compression in the lower layers caused by the reservoir level rising from el. 80 to el. 120.

The theoretical vertical subsidence profile determined by this FEM approach is also shown in Fig. 18. At all locations it is close to the subsidence profile which was actually measured. Also shown in Fig. 18 are the distributions of horizontal movements and horizontal strains computed by the FEM. Again there is reasonably good agreement with the observed data. Thus, this example illustrates that by using a realistic and an appropriate choice of values for modulus and Poisson's ratio, the FEM can be used with confidence to predict some types of vertical and horizontal movements within earth dams.

CONCLUSIONS

These studies suggest the following conclusions regarding the nature and distribution of horizontal movements which have often been observed to accompany ground subsidence.

1. It appears that a major and possibly a necessary requirement for horizontal movements to develop in conjunction with subsidence is the existence of an upper layer of material which does not contribute directly to the settlement but which lies over a deeper layer of material in which most of the compression develops.
2. Horizontal movements are related to the vertical subsidence profile as illustrated in Fig. 1.
3. The finite-element method of analysis appears to offer a useful solution to the problem of both vertical and horizontal movements, especially in cases where arching may develop because of a relatively thick and stiff upper layer.
4. Provided the subsidence profile can be estimated, Eq. 3 appears to be a satisfactory approximate method of computing horizontal movements. In spite of its empirical nature, it appears to predict both the nature and magnitude of horizontal movements to a useful degree of accuracy which may be satisfactory for many cases.

ACKNOWLEDGMENTS

The model tests were performed by the following students as a senior undergraduate project in the Department of Engineering, UCLA; J. A. Fitton, W. E. Simpson, L. I. Seals, J. D. Allison, R. K. Shanman, and J. S. Bianchi. The computer programs and valuable assistance in their use was provided by Stanley Dong, Dept. of Engrg., UCLA, and J. Lysmer and E. L. Wilson, Dept. of Civil Engrg., U.C., Berkeley. Several of the writer's colleagues read the manuscript and offered many helpful suggestions. Part of the work described herein was supported by Grant No. W144-4U-66/68, University of California Water Resources Center. Grateful appreciation is expressed for all of this assistance.

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LIQUEFACTION POTENTIAL

- Document Title: LIQUEFACTION POTENTAL

Doc. Originated by: TAC (Jacobs - Weston)

- Marginal Review Comments by: { Nani G. Banerjee, MKE (Member Work Group 2)

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E

PRELIMINARY DRAFT

1

LIQUEFACTION POTENTIAL

1.0 INTRODUCTION

Liquefaction potential will be assessed at UMTRA sites under Maximum Credible Earthquake conditions in order to evaluate the long term stability of the tailings pile.

Liquefaction and/or cylic mobility can only occur in saturated cohesionless soils (sands and silts) due to cyclic loading usually caused by earthquake induced ground motions. Liquefaction occurs when effective stress is reduced to zero by earthquake induced pore water pressure buildup. When this occurs the soil loses shear strength, becomes essentially a viscous fluid, and thus fails catastrophically. Cyclic mobility, on the other hand, occurs in denser soils. The pore pressure buildup causes loss of shear strength but results in a limited amount of shear strain (generally not greater than 15 percent) before pore pressures are reduced and shear strength is regained.

There are several factors which are important in assessing the potential for liquefaction and/or cyclic mobility. Of these the most important are: 1) the ratio of earthquake induced shear stresses in the soil to the vertical effective stress; and 2) the relative density (D_r).

Since there is a practical maximum acceleration and thus a maximum shear stress that can be produced by the largest earthquakes, this means that as the soil in question gets deeper the ratio of maximum possible shear stress to effective stress tends to become smaller. This factor generally precludes liquefaction and/or cyclic mobility at depths greater than approximately 50 feet below the ground surface. True for moderate Sha[king]

Also, most researchers agree that there is a relative density beyond which liquefaction cannot occur. Liquefaction can generally occur in soils with a relative density (D_r) less than 40 percent (Seed, 1976). Generally, beyond a D_r = 70 percent neither liquefaction nor cyclic mobility can occur (Casagrande, 1975).

Generally True except during very strong shaking

Does not seem quite right.
Please see attached Table 1 (Site Conditions and Earthquake Data For known Cases of Liquefaction and Non-Liquefaction. extracted from Seed's paper (1976), "Evaluation of Soil Liquefaction Effects on Level Ground Conditions", ASCE Annual Convention and exposition; Liquefaction Problems in Geotechnical Engineering, Philadelphia (See Insert-A)
The summary of This table shows that material with relative density in the range of 30 to 72%. (Avg: 52.5%) developed liquefaction; material with relative density in the range of 53 to 100 (Avg: 69.5%) did not develop liquefaction.

Please see insert A for Table-1

INSERT-A

TABLE 1 SITE CONDITIONS AND EARTHQUAKE DATA FOR KNOWN CASES OF LIQUEFACTION AND NON-LIQUEFACTION

Earthquake	Date	Magnitude	Site	Distance from Source of Energy Release (miles)	Soil Type	Depth of Water Table (feet)	Critical Depth (feet)	Effective Overburden Pressure, p.s.f. σ'_v	Correction Factor for Penetration Resistance, C_N	Average Penetration Resistance at Critical Depth N	Corrected Average Penetration Resistance N_1	Relative Density (%)	Maximum Ground Surface Accn. (g)	τ/σ'	Duration of Shaking (secs)	Field Behavior	Reference
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SUMMARY

LEGEND	Relative Density, Dr (%) Range	Remarks
Develop liquefaction	30 ~ 72 (Avg: 52.5)	Observed data reported by different researchers.
No Liquefaction	53 ~ 100 (Avg: 69.5)	

~~Table 4~~

~~Factors Affecting Soil Liquefaction Characteristics and Penetration Resistance~~

Factor	Effect on Stress Ratio Required to Cause Cyclic Liquefaction	Effect on Penetration Resistance
~~Increased Relative Density~~	~~Increases Stress Ratio for Liqn.~~	~~Increases Pen. Resistance~~
~~Increased Stability of Structure~~	~~Increases Stress Ratio for Liqn.~~	~~Increases Pen. Resistance~~
~~Increase in Time under Pressure~~	~~Increases Stress Ratio for Liqn.~~	~~Probably Increases Pen. Resistance~~
~~Increase in K_o~~	~~Increases Stress Ratio for Liqn.~~	~~Increases Pen. Resistance~~
~~Prior Seismic Strains~~	~~Increases Stress Ratio for Liqn.~~	~~Probably Increases Pen. Resistance~~

Table above extracted from Seed's paper (1976), "Evaluation of Soil Liquefaction Effects on Level Ground Conditions", ASCE Annual Convention & Exposition: LIQUEFACTION PROBLEMS IN GEOTECHNICAL ENGINEERING, Philadelphia.

(2)

2.0 ANALYSES TO BE USED IN UMTRA SITE DESIGN PROCESS

2.1 GENERAL

The analysis of liquefaction will be performed in a phased manner. Basically there will be three phases each depending upon the results of the previous phase.

2.2 PHASE I - ANALYSIS

The first phase involves using the simplified liquefaction analysis as developed by Seed and Idriss (1982), Attachment A.

Describe methods for determining relative density, Dr (See RAC's Design Procedures Manual, Chapter 11) of in place material from the SPT Data.

In this phase, and all that follow, it is explicitly assumed that no liquefaction will occur above the water table, even if there are zones of saturated or nearly saturated soils. It should be noted that this assumption may not hold in the case of an extensive saturated zone associated with a perched water table. These cases would be analyzed for liquefaction. For this phase it will further be assumed that only sands (SP, SW), silty sands (SM), and low plasticity silts (ML) with a relative density less than 70% are capable of liquefying.

Generally true, however this statement apparently contradicts [illegible]

The Seeds and Idriss simplified method is based on the empirical correlation of documented cases of liquefaction as related to measured ~~Richter~~ *R* Richter magnitude (M), maximum horizontal ground acceleration at the site and the standard penetration test (SPT) blow count (N) of the soil prior to liquefaction.

The SPT data needs (N) various corrections including normalization (N_1).

In order to determine the maximum horizontal ground acceleration at the site, the maximum credible earthquake (MCE) and the distance to the causitive fault will be used in conjunction with the attenuation curve as developed in the seismic position paper to estimate the maximum acceleration in rock below the site. Since this motion will be either attenuated or amplified by the foundation soils at the site, the appropriate curve, as presented in the previously mentioned paper by Seed and Idriss, will be used to estimate the maximum horizontal ground acceleration at the surface of the site. This acceleration will then be used to determine the shear stresses developed by the earthquake in the various soil layers below the site.

The value for shear stress required to cause liquefaction in a particular layer. The shear stress required to cause liquefaction is found by entering a family of curves (See and Idriss, 1982), for the given magnitude and the SPT blow count appropriate to the layer in question. Full accounting of the grain size distribution will be taken as indicated in these curves. *where?*

Seed and Idriss state that a factor of safety against liquefaction in a given soil layer can be calculated by dividing the shear stress required to cause liquefaction in the layer by the shear stress generated in that layer. They state that a factor of safety between 1.25 and 1.5 should be taken as the minimum. Since UMTRA sites are generally in rural

adopt

Describe method for the normalization of SPT data (From N in Blows/Foot recorded, determine N_1 in Blows/Foot) (See RAC's Design Procedures Manual, Chapter 11).

- *How about the influence of clay content on liquefaction resistance? This should be considered*

Because of many simplified assumptions in the analysis, the required minimum factor of safety should be close to 1.5

and the consequences of liquefaction failure are generally considered minimal the lower value (or 1.25) will be taken as the minimum acceptable factor of safety for design purposes. (3)

Should be close to 1.5

Should the Phase I analysis show that significant areas of the tailings and/or foundation materials may liquefy (i.e., have a factor of safety less than 1.25) then a more detailed analysis is called for and will proceed as outlined in the following section. If only limited layers indicate a potential for liquefaction, then further analysis may not be warranted. If the hazard is considered minor, i.e., no extensive flow resulting from liquefaction, the potential for liquefaction will be noted and surveillance following earthquake events will be required.

2.3 PHASE II - ANALYSIS * (Please see general comments at the bottom of the page).

In the above analysis the least accurately known factor is the actual shear stress developed within any given layer in the soil profile. Thus the second phase will undertake to more accurately calculate the shear stresses developed within the soil profile.

Need to develop the design accelerogram in early stages of this project

In order to do this, the program "SHAKE" (Schnadel et al., 1972), will be utilized. In using this program a digitized earthquake record from a previously recorded earthquake in a similar tectonic setting will be appropriately modified to better approximate the site MCE and will be mathematically input at bedrock level within the program. In order to better calculate the shear stresses in each soil layer of the profile it is also necessary to determine the shear modulus and damping characteristics of the soil layer under consideration as a function of shear strain. These material properties will be taken from curves presented by Seed and Idriss (1970). The program then computes the shear stresses at various predetermined points within the soil profile using the one-dimensional wave propagation method.

The calculated shear stresses developed by the earthquake are then compared to the shear stresses causing liquefaction (as determined in Phase I) and, since more accurate values of the developed shear stresses are used, a factor of safety of 1.0 will be considered the minimum value for design purposes.

• Suggested minimum FS ≥ 1.25

If it is found that the factor of safety is less than 1.0 a sensitivity analysis will be performed to determine if the soil damping and shear modulus values are critical. If so, a limited amount of dynamic testing will be performed to establish some benchmark values for shear modulus and damping. These values will then be used to appropriately to shift the modulus and damping curves. These modified curves will then be input and a second analysis performed. If the factor of safety for a given layer is still below 1.0, the Phase III analysis will be initiated.

* ① Our greatest concern in suggesting any sophisticated dynamic analysis procedure is that without adequate site specific data, the level of confidence in the results may be uncertain

② For sites where Phase II analysis is anticipated a suitable accelerogram should be selected, representative of the design earthquake, during the Site Characterization and or the preparation of the RAP.

③ We also suggest some resonant column tests (be done) on representative soil sampl

(CONTINUED ON NEXT PAGE)

(4) The tasks described under items (2) and (3) are slow and time consuming, therefore should be initiated in the early stages of the project. Any attempt to generate this data during the final design phase may result in schedule conflict.

2.4 PHASE III - ANALYSIS

The Phase III analysis will involve an extensive lab testing program to determine the dynamic properties of the soil deposit for input into a 2 or 3 dimensional computer model (i.e., Dynamic Finite element modeling). (4)

This process is extremely site specific and thus no detailed procedures will be developed here. This process will also be very expensive and will only be undertaken for cases where a failure in the tailings embankment would result in very serious consequences.

I don't visualize the need for this method. Three-dimensional effect is significant in narrow valleys. Three dimensional effect also could be incorporated (with reasonable degree of accuracy) in a two-dimensional program by stiffening the soil by adjusting the input soil parameter, (K2) max.

General Comments on This Chapter

- For some sites where the seismic risks are low and the site and soil conditions are favorable, some very simple methods available should be adequate. Please see chapter 11 of RAC's Design Procedures Manual.
- It is extremely important to have adequate, high quality SPT data covering the site area; with corrected SPT data, the liquefaction analysis could be done fast and will ensure acceptable level of confidence in the results.

(5)

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Document Title : SEISMIC HAZARD ASSESSMENTS FOR THE UMTRAP

Doc. Originated by: TAC (Jacobs-Wes-

Marginal Review Comments by } Nani G. Banerjee MKE Member, WorkGroup-2

SEISMIC HAZARD ASSESSMENTS for the URANIUM MILL TAILINGS REMEDIAL ACTION PROJECT.

prepared by

JACOB WESTON TEAM.

May, 1985

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ADVANCED SYSTEMS DIVISION, ALBUQUERQUE OPERATIONS

(B)

Seismic Hazard Assessments for the Uranium Mill Tailings Remedial Action Project

Note

Besides the comments on the body of the text, please see also Attachment - A included at the end of this text.

Attachment - A includes discussion on the following:

- Development of MCE,
- Design Earthquake and Related Parameters.
- Some suggestions relating to seismicity study & Report Contents.
- A note on earthquake acceleration attenuation after Iwasaki et al (1978).
- A note on Seismic Coefficient Method of Design of Dams in Japan.
- Table B-1: Design Seismic Coefficients for Dams in Japan.
- Table - 2: Selection of Design Earthquake Parameters for Critical Structures (After: Newmark & Seed)
- Table-3: Design Criteria for Pseudo-Static Analysis of Embankments [From Rankine Lecture, 1979, by H. B. Seed]

June, 1985

Final Draft

40 CFR Part 192 Specifies the design life upto 1000 years, to the extent achievable, but atleast 200 years. It must also be pointed out that atleast 1000-year design life specified in the proposed Standard Page 1 was changed from the consideration of cost and benifit and problem of certifying implementation of the Standard. THIS IS AN OPEN ISSUE.

1.0 INTRODUCTION

As an integral part of conceptual design development for the reclamation of the 24 UMTRAP sites, a seismic hazard evaluation is completed. This evaluation provides an estimate of potential earthquake-induced ground motion which has a reasonable chance of occurring during the 1,000-year design life of the reclaimed facility. Once this estimate is provided and the geographic distribution of potentially active faults is ascertained, the potential for damage to the structure is assessed. This damage could result from liquefaction of soils underlying the tailings or the tailings proper, on-site fault rupture, and/or slope failure of the containment due to excessive ground motion. If the potential for damage is distinct, remedial actions to mitigate the resulting damage are considered during design development.

How about earthquake induced Landslide potential?

The phase of seismic studies discussed herein is restricted to a seismotectonic characterization of each UMTRAP site, and development of initial on-site acceleration values, duration of shaking, distance to the primary causitive fault or faults and other parameters required to characterize the seismic hazard.

The Uranium Mill Tailings Project office of the Department of Energy (DOE) has recently received comments by the Nuclear Regulatory Commission (NRC) on seismic studies completed for the Salt Lake City (SLC) and Shiprock (SHP) UMTRAP sites. These documents contain both generic and specific comments concerning the technical approach, the scope of previous studies, the definitions of several key terms, and acceptable methods for developing parameters characterizing the seismic hazard.

Not clear will this not involve performing all tasks (field & office) evaluating site Seismicity including Active Fault Studies, developing MCE, Design Earthquake, and the seismic design parameters?

Within the context of this submittal and ensuing discussions with NRC staff, we wish to clarify our position relative to the scope and content of seismic hazard evaluations completed for the UMTRAP sites. We trust that a standardized, technically sound approach to estimating seismic hazard and developing associated characteristic parameters will result from these communications.

Based on the NRC comments regarding the content of the SLC and SHP seismic hazard evaluations, there are several specific issues to be addressed. In summary, these issues are as follows:

- Explanation of phased investigative program and

Also particulars of field and office studies performed, findings and analysis, leading to logical conclusions recommendations with back up exhibits (Aerial Photos, Field Geology Map, ...

specific methodologies applied to the assessment of seismic hazard for each UMTRAP site.

- Explicit definition of "active fault" which satisfies the requirements of 40 CFR 192. The selection of this definition should consider the long exposure time of the UMTRAP sites and the impact on public health and safety should the tailings sites experience earthquake damage.

- Standardization of seismic acceleration terminology.

?

- Use of deterministic attenuation relationships which are acceptable to all parties concerned.

- Use of a percentile bound in applying attenuation relationships which represents a reasonable level of conservatism.

or Attenuation relationship or relationships developed based on Western United States earthquake data and found wide acceptance by the professional Community?

Each issue listed above will be discussed separately in the text presented below. Our recommendations related to each subject are presented at the end of the separate report sections. The appendices contain bibliographies of literature reviewed during the compilation of this paper and annotated discussions on several specific issues.

2.0 PHASED APPROACH TO ASSESSING SEISMIC HAZARD

The following outlines a sequential, phased approach which we propose to follow in all future seismic investigations for UMTRAP sites. We recognize the validity of NRC comments regarding the need for a more complete literature review, a greater effort on-site to evaluate the disposition of known faults, and a more thorough search for previously unrecognized features.

Phase I

- All pertinent geological, geophysical, geomorphological (including soils) mapping and data will be acquired and interpreted. These data will include existing maps which delineate active faults and bedrock faults of any age in the site region, and published discussions pertaining to the regional seismotectonic setting. Computerized data bases such as Georef will be employed to assure acquisition of all pertinent information.

We suggest the follow sources be tapped for existing information in addition to others you have in mind:

1. U.S. Army Corps of Engineers.
2. U.S. Bureau of Reclamations.
3. Various Light and Power Companies, Owners of Nuclear Power Plants.
4. and U.S.G.S.

Some critical analysis of the historic earthquake data is extremely important, such as (a) Frequency of Occurrence Vs Earthquake Magnitude plot of Earthquakes within 200 miles radius from the site may provide very meaning clue to future earthquake activity in the site area.

- An assessment of instrumentally and historically recorded earthquake files will be completed. An epicentral map showing the geographic distribution of all known earthquakes within 120 miles of the sites will be compiled. The 120-mile radius has been selected so that any seismic event which could have caused detectable on-site ground motion will be included in the compiled record. The NOAA and all earthquake data files, epicentral listing from state-maintained seismic nets, and available micro-seismic data will be obtained and evaluated.

We suggest 200-mile radius cover. This will not involve any extra time & cost but provid better picture of the historical seismicity of the region.

These should be included in your Final Report

- A selected suite of remote sensing imagery and conventional aerial photography at suitable scales will be acquired and analyzed. All photogeologic lineaments or geomorphic features indicative of an active seismic setting will be plotted. Specific attention will be paid to any active fault or bedrock fault traces identified by previous investigators. The photo coverage and analysis will encompass an area within a 12-mile radius of all sites and alternative disposal areas, plus selected strip coverage of terrain which may contain active faults, up to a distance of 40 miles from the sites. We maintain that the photogeologic analysis and subsequent reconnaissance of an area within a 12-mile radius of the site, and selected areas within 40 miles, will adequately identify any active faults or systems which could pose a surface rupture hazard. The terrain study completed in selected areas up to 40 miles from the site will normally be sufficient to characterize the distribution of active faults which could control the development of the on-site acceleration value.

The Office Studies should cover atleast 65 miles radius area from the site; the extent of field investigation will be influenced by the findings of Office Studies. For example a 50-mile long active fault located at distances greater than 40 miles could cause significant shaking at the site.

- Utilizing the findings of the previous efforts, ground and aerial observations of known faults and suspect terrain indicative of an active seismic setting will be performed. All aerial reconnaissance missions will be completed under low-sun-angle conditions. The areas within a 12-mile radius of the sites will be thoroughly reconnaissanced and any outlying features which could influence the derived regional maximum credible earthquake (MCE) or influence the earthquake design parameters will be studied in the field.

- The findings of the efforts discussed above will be compiled into a series of maps which depict the distribution of active faults and known earthquake

I believe, the details of field investigation (aerial and ground) and the findings of studies will be included in your report along with the exhibits (ERTS Imagery, Aerial Photos, and field geology maps).

How about the relationship between fault rupture length (L) and magnitude (M) suggested by Mark, R.K. (1977)?

epicenters. A detailed discussion of each seismotectonic setting will be developed. If specific seismogenic sources can be identified, an MCE for each fault system will be estimated using the fault length versus magnitude relationships developed by Slemmons (1977, 1982)*. By applying deterministic and/or probabilistic methods, an initial estimate of on-site ground motion will be developed. Currently acceptable attenuation relationships will be applied. A discussion of these relationships is presented in Section 4.2 of this submittal.

- Decision by DOE with concurrence by NRC on the adequacy of the initial phase of work, and a determination as to what additional evaluations, if any, are required. The various questions which need to be answered at this stage of the investigative process include: (1) will further investigations lead to more critical design parameters?; (2) is there a liquefaction hazard?; (3) using the initial, conservative design parameters, what damage to a well engineered facility could occur?; and (4) if deemed necessary, what additional studies are needed?

If the current field investigation program is likely to affect the schedule, priorities in investigating the sites will have to be established in consultation with the DOE

Phase II

- Applying methods developed by Glass and Slemmons (1978), an LSA aerial photo mission of the area within a 12-mile radius of site, in addition to strip areas which contain known or suspected potentially active lineaments, would be completed.

- A subsequent photogeologic critique of LSA photographs would be completed. Any geomorphic features indicative of active faulting would be identified.

- A detailed field study of suspect features using scarp morphology, trenching, and radiometric age dating of surficial materials would be performed.

- The seismic design parameters would be reestimated.

---

*References are listed at end of report.

Phase III

° If necessary, i.e., where specific suspect features play an outstanding role in assessing the seismic hazard, a more detailed investigation of the specific features would be performed. This phase could include additional trenching, radiometric age dates, test borings and geophysical surveys, where applicable.

° The seismic design parameters would be reestimated in light of all data gathered to date.

For a vast majority of the UMTRAP sites, the Phase I program discussed above will probably be more than sufficient in adequately evaluating the seismic hazard. For each seismic hazard evaluation, the decision to proceed with additional work should be based on a perception of the risk of earthquake damage, and the impact this damage would have on public health and safety. It should be recognized that in cases where a low liquefaction hazard exists, the actual damage which could be caused by strong ground motion would not have an adverse impact. The geometry and ultimate siting of the UMTRAP reclamations are tailored to minimize long-term impacts on groundwater quality and to enhance the erosional characteristics of the encapsulation. The above-grade configurations being proposed for all reclaimed sites inherently minimize the risk of earthquake damage. The reclaimed side slopes will not exceed 5:1 (horizontal to vertical), and sites which are not underlain by shallow groundwater are favored. In some cases, reclamation plans include the engineered compaction of the loose tailings. All these design or siting features enhance the seismic safety of the reclamation action.

How you are planning to assess the liquefaction potential?

Irregardless of the inherent seismic stability of the proposed actions, we intend to evaluate each disposal area on a site-specific basis. Once the possibility of liquefaction is eliminated, the only seismically-induced damage that could be anticipated is disruption of the radon/erosion-protection covers. In the seismotectonic settings of a vast majority of the UMTRAP sites, we believe that the possibility of cover and underliner damage is quite remote. Due to this low risk, it appears that repair of such damage could be instigated as a contingency response measure. We believe this basic approach is in keeping with the objectives of the EPA standards.

As a starter, the results of studies published by the following may be used with great advantage:

1. Open File Report 76-159: Colorado
2. " " " 75-278: Idaho
3. " " " 75-279: Wyoming

by Witkind et al, USGS.

4. Earthquake Studies in UTAH by Arabasz, W.J. et al (1979)
5. Earthquake Potential in Colorado by Kirkham & Rogers (1981)

Are you suggesting that an earthquake event smaller than MCE should be selected as the design earthquake? But UMTRA document specifies that the remedial designs should be safe against an MCE event!

3.0 DEFINITION OF AN ACTIVE FAULT

As evident from the multiple definitions presented in Appendix A, there is no standardized definition of an active fault. As stressed by Nichols and Buchanan-Banks (1974), the definition of what constitutes an active fault varies greatly according to the type of land use contemplated. For extremely critical structures, such as nuclear reactors and natural gas facilities, conservative definitions are employed. These studies define faults with very low rates of activity and very long recurrence intervals as active because of the adverse impact to public health should an earthquake seriously damage the structure. For a less critical structure, such as well-designed tailings pile, a less conservative approach is justifiable.

The definition which has been applied during our previous studies is provided by Slemmons and McKinney (1977) and is as follows:

"An active fault is a fault that has slipped during the present seismotectonic regime and is therefore likely to have renewed displacement in the future. The fault activity may be indicated by historic, geologic, seismologic, geodetic, or other geophysical evidence. The most widely used definition in current engineering practice is for faults with evidence of Holocene displacement (approximately the last 10,000 years)."

The Holocene represents the period of time which has elapsed since the last Pleistocene glaciation (i.e., about 10,000 to 12,000 years). It therefore may not accurately represent the duration of the "present seismotectonic regime". However, surficial deposits of Holocene age can often be distinguished from older units in the field, and reasonably accurate estimates of their ages can be made. Without extremely elaborate field studies, age estimates of Pleistocene and older units are generally subject to greater uncertainty. The Slemmons and McKinney definition quoted above provides a criterion which is applicable within the context of the UMTRAP seismic studies. These studies normally will not include the level of effort required to fully document the Pleistocene displacement history of a particular fault.

We have recognized in our previous reports that fault movement in the area of many UMTRAP sites (i.e., in the Colorado Plateau) may have recurrence intervals on the order

It's true Holocene faulting is most significant, Pleistocene less so, and older faulting progressively less important (Allen, 1975). Since an MCE event comes under the category of long recurrence interval earthquake, we should look for late Quaternary faulting.

According to Smith (1978), seismic zoning based solely upon epicenter maps of historic earthquakes or solely on evidence of Holocene faulting should not be done.

of tens of thousands to hundreds of thousands of years. In such a case, a fault which is accumulating stress under the present seismotectonic regime and may produce earthquakes in the future may not be identified as active under our definition. However, when such a condition is recognized, an attempt will be made to estimate the age of the last movement on the fault.

It would be convenient if a perception of risk could be ascertained by associating evidence of fault displacement in the past 10,000 to 12,000 years with the 1,000-year design life of the UMTRAP reclamation plans. In simple terms, it appears that this comparison implies that there is a 10 percent chance that a fault, classified as active under this criterion, will experience renewed displacement in the next 1,000 years. This statement is not true because the occurrence of future fault activity is dependent upon the tectonic flux or rate of strain being imposed, and the associated recurrence interval of surface rupture of the fault system. It is entirely possible that a fault in the stable interior of the Colorado Plateau which displays Holocene activity may not experience movement for another 100,000 years or longer. It is also plausible that faults which would not be recognized as active under any conservative definition and level of study may be the seismogenic source of an earthquake within the next 1,000 years.

We recognize the difficulties in fully documenting the tectonic history of an area, and the displacement history of faults. We will qualitatively classify a fault as active if it is believed that the structure has experienced displacement during the present seismotectonic regime. This approach is quite conservative. In the seismotectonic settings of most UMTRAP sites, this approach would group all faults which have displaced Quaternary geologic units as active.

In conclusion, we recommend that the definition of an active fault quoted above be applied to all seismic hazard evaluations for UMTRAP. We believe this definition is in keeping with the objectives of the EPA standards.

4.0 ACCELERATION & ATTENUATION

4.1 Terminology

It is agreed that the terms "sustained peak acceleration", "peak horizontal acceleration" and "effective

I believe we should go along with NRC and adopt their definition of Capable/Active Fault.

peak acceleration" have distinct meanings, and that they should be explicitly and consistently used. Since the acceleration is determined from an attenuation relationship based on distance and magnitude, the appropriate term is also dependent on the attenuation relationship. Typically, most relationships provide predictions of peak hori- zontal acceleration. In future work, usage of this term will be strictly adhered to, and use of any other terms will be preceded by a definition of the term.

Excepting very Soft Soil, site Conditions do not dramatically change the peak rock acceleration. For shallow heights Seed sometimes ignores the attenuation or amplification effects. However for liquefaction analysis we will have to determine the ground acceleration at the crest of the tailings embankments.

It is of some importance to note that the term "peak horizontal acceleration" is not always strictly defined. Joyner and Boore (1981), for example, explicitly define this term to be the larger peak value of the two horizontal components of a record, regardless of instrument location or site geology. Campbell (1981) defines it as the mean of the peaks of the two horizontal components, noting that if a peak horizontal acceleration independent of horizontal direction is required, the mean should be multiplied by a factor of 1.13. As with Joyner and Boore (1981), the term is explicitly defined with reference to the record of the motion. Donovan (1973) uses the term "peak ground acceleration", however, it is not fully defined. This term could be interpreted to be the peak of the horizontal and radial components, but to use the adjective "ground" is misleading, since most instruments are located in or on buildings, dams or other structures. Schnabel and Seed (1973) use the term "maximum rock acceleration" primarily because the records used were from sites underlain by rock and not soil. Donovan and Bornstein (1978) use the term "peak horizontal ground acceleration", but do not provide a complete definition.

4.2 Attenuation Relationships

There are a large number of acceleration attenuation relationships available for use. A listing of articles that present relationships is included in Appendix B.

To address the issue of which relationship should be used, we have selected eight for comparison. Selection of these eight does not imply that they should be applied to the UMTRAP sites; they only represent a cross section of available relationships. These are described in the annotated bibliography of Appendix C. Included

in the notes provided are the relationship, a definition of the terms used, and a discussion of the data base analyzed to develop the relationship. Source distance (near-field versus far-field events), earthquake location (California, worldwide or United States events), the sophistication of the regression analysis employed in analyzing the data, and the number of records used are key elements in selecting an attenuation relationship to be used.

A comparison of the eight relationships is provided in Table 1, which lists acceleration values predicted by each relationship for magnitude 6 through 7 events at distances of 5 to 100 kilometers. A comparison of the acceleration values indicates that the largest differences occur for larger magnitude events in the near-field. Bolt (1982) and Campbell (1981) both specifically attempted to consider the fault mechanism effect in the near-field in their analyses, the latter by developing an alternate constrained relationship. Since Campbell's data base is broad, and his analysis attempts to consider earthquake location, his constrained relationship might be most appropriate for use in the near-field.

-But this attenuation relationship has not been used in any of your report read so far.

Blume (1977) has considered a large data base, but it is limited to events in the United States, which would likely be biased by California events. The only relationships that do not consider mostly California events are those of Campbell (1981) and Donovan (1973). These relationships predict very similar acceleration values, which may be a reflection of their nonbias as regards to event location. Since the UMTRAP sites are located outside the California-Nevada area, either of these relationships might be most appropriate for use.

Considering that Campbell (1981) also has provided a more complete analysis of available data than the other articles, it is our position that either of his attenuation relationships should be used for the UMTRAP sites. It is noted that Campbell (1981) also considers in his analysis the variable location of the recording instrument and variable site geologic conditions.

4.3 Confidence Level

See also my computation on Table-1

Table 1 also addresses the issue of confidence

Campbell's attenuation relationships (are) relatively new. It tends to underestimate the peak acceleration; moreover, because of limited use so far, it will be difficult (to) verify how Campbell's acceleration values compare with the observed acceleration of Western United States earthquakes.

I believe the mean peak acceleration as obtained from 4 or 5 selected attenuation relationships will be more reliable than a single attenuation relationship. Alternatively, one out of the following three attenuation relations be selected:

(Please see next page)

MKE Recomm-endation

TABLE 1

Comparison of Acceleration Values From Selected Attenuation Relationships

Reference	M = 6.0					M = 6.5					M = 7.0					M = 7.5				
	5[1]	10	20	50	100	5	10	20	50	100	5	10	20	50	100	5	10	20	50	100
Joyner & Boore (1981)																				
50 percentile	0.32[4]	0.22	0.12	0.044	0.016	0.43	0.30	0.16	0.058	0.022	0.57	0.40	0.22	0.078	0.029					
84 percentile	0.58	0.41	0.23	0.080	0.030	0.78	0.54	0.30	0.106	0.040	1.03	0.72	0.40	0.14	0.053					
out **Campbell (1981)** (unconstrained)																				
50 percentile	0.26	0.16	0.091	0.036	0.018	0.34	0.22	0.13	0.056	0.028	0.42	0.29	0.18	0.083	0.042					
84 percentile	0.38	0.24	0.13	0.054	0.027	0.49	0.32	0.19	0.081	0.040	0.61	0.43	0.26	0.12	0.061					
Campbell (1981) (constrained)																				
50 percentile	0.29	0.18	0.096	0.029	0.010	0.34	0.24	0.14	0.048	0.018	0.38	0.29	0.19	0.074	0.031					
84 percentile	0.42	0.27	0.14	0.043	0.015	0.50	0.35	0.20	0.071	0.027	0.56	0.43	0.27	0.11	0.046					
out → **Donovan (1973)[2]**	0.25	0.20	0.13	0.062	0.028	0.33	0.26	0.18	0.083	0.038	0.44	0.35	0.24	0.11	0.051					
out → * **Schnabel & Seed (1973)[3, 2]**	0.49	0.35	0.20	0.07	0.02	0.57	0.40	0.25	0.09	0.025	0.63	0.48	0.32	0.15	0.035					
out → **Blume (1977)**																				
50 percentile	0.15	0.11	0.069	0.026	0.010	0.25	0.18	0.12	0.044	0.017	0.51	0.37	0.23	0.081	0.029					
84 percentile	0.37	0.28	0.17	0.066	0.025	0.62	0.47	0.29	0.11	0.043	0.92	0.68	0.41	0.15	0.052					
Bolt (1982)[2]	0.24	0.17	0.090	0.013	0.00	0.43	0.35	0.23	0.063	0.007	0.28	0.26	0.23	0.13	0.049					
Donovan & Bornstein (1978)																				
50 percentile	0.39	0.25	0.13	0.090	0.022	0.47	0.31	0.18	0.074	0.035	0.56	0.40	0.25	0.11	0.056					
84 percentile	0.53	0.34	0.21	0.081	0.036	0.63	0.42	0.27	0.12	0.057	0.75	0.54	0.35	0.17	0.091					
$\bar{x}$ =	0.299	0.205	0.116	0.042	0.016	0.395	0.283	0.174	0.065	0.024	0.474	0.355	0.233	0.102	0.040					
σ =	0.103	0.072	0.040	0.019	0.009	0.100	0.072	0.047	0.016	0.010	0.115	0.073	0.043	0.028	0.011	–	–	–	–	–
n =	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8					
include { Iwasaki et al (1978)	0.329	0.189	0.109	0.052	0.030	0.466	0.268	0.154	0.074	0.042	0.660	0.379	0.218	0.105	0.060	0.935	0.536	0.308	0.148	0.0[illegible]
" { Seed and Idriss (1982)*	0.451	0.309	0.177	0.058	0.014	0.545	0.402	0.252	0.093	0.023	0.579	0.454	0.305	0.135	0.047	0.599	0.495	0.359	0.18[illegible]	0.[illegible]

[1] Distance in kilometers
[2] Mean (50 percentile) value
[3] Graphical relationship
[4] Acceleration in gravity units

SERGENT, HAUSKINS & BECKWITH

* NOTE: Quite possible these are the same attenuation relationships which appear in two separate publications [illegible]

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① Joiner & Boore (1981)
② Seed & Idriss (1982)
③ Iwasaki et al (1978).
} Acceleration Attenuation Relationship after.

level by comparing 84 to 50 percentile acceleration predictions for several of the relationships considered. The differences are typically large, increasing with increasing magnitude and decreasing distance. Thus, as has been suggested, it might be most appropriate to use the 84 percentile value in design. However, it should be recognized that these relationships predict a peak acceleration value, which only represents one peak cycle of several hundred cycles recorded during an event.

To address the potential effect of using a peak value, Bolt (1982) analyzed 62 records, finding that the 90, 95 and 99 percentile levels of acceleration differed remarkably. For one given record, the associated acceleration values were 0.23g, 0.28g and 0.62g. Because of this degree of difference, Bolt suggests a percentile acceleration be used that is appropriate for the degree of risk involved.

Applying an 84 percentile bound in an attenuation relationship in conjunction with use of a peak value (which represents a 99+ percentile for a given record) is unduely conservative. It is our position that the mean value of a relationship based on peak values be used for the UMTRA project sites, as it represents a reasonable level of conservation.

I believe for UMTRA Project mean (peak) acceleration values should be adequate, even though for some critical structures 84 percentile acceleration value is sometimes used.

General Comments:

- Please read this marginal comments along with our written comments sent to TAC on 14 June, 1985
- Some material from the written report referred above is included as Attachment - A for the convenience of the reader.
- Are you considering the possibility of assuming the design life of the UMTRA facility as 200 years for assessing seismic risks
  (Refer 40 CRF Part 192, page 596 & 597: "Those few piles that are susceptible to flood damage will be protected for atleast 200 years, and might not suffer real damage for much longer".)

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APPENDIX A

APPENDIX A

Resume of Literature on the Definition of Active Fault

The meaning of active fault varies widely depending on how it is to be used. Most of the literature relates to engineering structures, and for these, definitions can vary widely with the importance of the structure as it relates to the effect that the structure may have on people if it failed during an earthquake. Because of the severity of the earthquake problem in California, much of the literature refers to that state. The only publication in which the definition of active faults is the subject is by Slemmons and McKinney (1977).

In addition to the literature summaries that follow, other summaries are given which are taken from references given in Slemmons and McKinney (1977). This was done because not all of their references were available in time to be reviewed for this report.

Summaries of Publications

Wood (1916) recognized for California that large earthquakes occurred along faults, and that earthquakes will occur along these faults in the future. He referred to the fault zones as "'living' zones of geological faulting . . . ." Topographic features along these zones are "small knolls, small pools, rush-ponds, marshes, small embankments and escarpments . . . ."

Louderback (1937) gives criteria for determining that a fault is active. These are geological, historical, and seismological. He states: "The best geological criterion is based on evidence of recent displacement along the fault, and especially on evidence of a series of displacements, running through a long period of time and coming down close to the present . . . If a fault shows evidence of repeated movements during Quaternary time, up to and including very recent offsets, as, for example, in very young alluvium, it is not likely that it died but yesterday, and we must believe that future movements are practically certain. The actual observational evidence includes fresh or youthful nonerosional scarps, offset streams and alluvial fans, longitudinal depressions and sag ponds, deformed and displaced recent deposits, and similar phenomena along the fault or shear zone . . . Historical evidence lies in the records of earthquakes the descriptive accounts of which

permit reasonable reference to a particular fault . . . A seismographic method of learning what faults in a region are active is that of determining the locations of centers of origin of recurring small earthquakes."

Louderback (1950) states that faults are often classified as active or dead. The faults "are considered active which are undergoing movement now or have undergone movements in recent geologic or in historic time and are considered liable to recurrent movements in the future. Dead faults are those which were active in some earlier period of diastrophism but show no sign of having been active in more recent geologic time. Criteria . . . that a fault is active are geological, historical, and seismological. The active group cuts all formations, including the recent, with which they come in contact, have direct expression in the topography, and a number have undergone movements accompanied by earthquakes in recent historic time."

Schultz and Cleaves (1955) state that active faults are "Fractures that are known to have experienced dislocation in historic time . . . The most direct and best evidence of activity is that furnished by seismographs and bench marks . . . Similarly, if accurately located bench marks exhibit horizontal or vertical displacements, any faults known to exist in the area should be regarded as active . . . The physiographic evidences of active faulting are (1) bold escarpments, (2) sag ponds, (3) offset streams, and (4) shutterridges . . . If a fault is known to be overlain by younger strata that are not displaced, it is permissible to regard it as inactive . . . ."

Trefethen (1959) refers to faults as either live or dead. He states: "A fault is considered live if displacements have occurred along it within historic time, whereas a fault on which no recent slipping has taken place is considered dead."

Sherard and others (1963) state: "By definition, active faults are those which either are clearly undergoing movement or have undergone movement in historical or recent geological time. Dead faults are those which show no sign of having been active in recent geological time."

Allen and others (1965) on geological criteria for activity of faults in southern California state: "In the absence of strain-accumulation data on historic records of

major earthquakes along a given fault, the only satisfactory criterion for activity lies in geological evidence that displacements have taken place along the fault in the recent geologic past. Faults that have had sufficiently recent movement to displace the ground surface are usually considered active by geologists simply because the ground surface is a very young and ephemeral feature. Such physiographic evidences of faulting (e.g. scarps, sag ponds, offset drainage lines) are powerful tools in identifying and studying active faults, but in practice it is difficult to use these features to compare degree of activity between different faults or to establish the time interval since the last major displacement. One principal problem is climatic: average annual rainfall varies by more than 25-fold within the area of this study, so that steepness and 'freshness' of scarps may be more a function of location than age . . . Offset drainage lines resulting from horizontal fault displacements are another very ephemeral feature of faults and therefore indicative of current activity . . . Most offsets have thus been considered of Recent age, although it is recognized that the ability of a stream offset to maintain itself will depend not only on age, but also on climate, rock type, depth of stream incision, regional gradient, and rate of fault movement." They further state that due to variations in climate, rock type, depth of stream incision, regional gradient, and rate of fault movement, the ages of such features as stream offsets and scarps in alluvium can vary greatly." Thus one is forced to the conclusion that if stream offsets and scarps in alluvium are to be used as criteria for activity of faults, then the term 'active' must apply to events dating well back into the Pleistocene epoch, perhaps as much as 100,000 years."

Albee and Smith (1966) state that a degree of activity should be assigned to most faults. They state: "the use of the term 'active fault' . . . reveals that most geologists use the term . . . in the sense expressed in Webster's (dictionary) definition that active means 'in action, moving' . . . Activity is not just a single state, but varies over a broad spectrum of degrees of activity. For example, some faults are exceedingly 'active', others are moderately 'active', and some are only slightly 'active' . . . There are similar difficulties in classifying a fault as 'inactive' or 'dead' . . . The more obvious evidences of fault activity are historic surface faulting, the occurrence of large earthquakes related to a fault, and measurements of accumulated strain . . . In addition to

the seismic record, geodetic measurements may provide a criterion for recognizing activity and estimating the degree of activity . . . Geologic evidence on 'the degree of activity' is a more powerful tool than seismic or geodetic evidence because the geologic evidence monitors a fault over a time period 1,000 or more times as long as the accurate seismic and geodetic records . . . One common criterion of action is that a fault has undergone displacement in Recent (10,000 years) time . . . Fault action may be indicated by displacement of the ground surface, which . . . suggest(s) recent movement. Physiographic features indicating ground displacement include fault scarps, sag ponds, and offset drainage lines. It should be clear . . . that physiographic features provide only a relative age for last movement on faults . . . Geologic criteria provided by inferred regional mechanics can give a broader view of the degree of fault activity."

Cluff and Bolt (1969) list topographic characteristics along active faults in the San Francisco Bay region. They state: "Distinctive 'rift topography' such as offset streams, side-hill ridges, scarps in alluvium, alignment of springs, shutter-ridges, and sag ponds are features common along most major active faults."

Bonilla (1970) states that an active fault can be defined as one that has moved in the recent past (about 10,000 years) and may move in the near future. Further, he states: "The determination . . . involves geology, geophysics, geodesy, and engineering. Some criteria currently in use are (1) the occurrence of earthquakes that can be related to the fault with reasonable assurance, (2) one or more episodes of surface rupture (including tectonic creep) or acute bending in the recent past as defined above, (3) instrumental evidence of elastic or inelastic strain, and (4) structural coupling to another fault (or other tectonic feature such as a monocline) that is active."

Flawn (1970) defines active and dead faults as follows: "Active faults are faults along which there has been movement in historic or Recent geologic time, or along which recurrence of movement is predicted or is likely to occur; dead faults are those along which there is no indication of movement in historic or Recent geologic time and no reason to predict a recurrence of movement. There are, of course, very subjective distinctions. Studies that demonstrate accumulation of strain in rocks in an area would justify reclassification of a dead fault to the active category.

Faults in seismic areas, even without a history of movement, are more likely to slip than faults in regions without a history of seismic activity."

Wesson and others (1972) list criteria by which active faults that may generate damaging earthquakes have been identified. These criteria have been given by decreasing certainity of activity: "1) historic and/or current seismicity or ground breakage, 2) physiographic faultline features and disturbances in Holocene sedimentary deposits which indicate displacement in the last ten thousand years, 3) structural geologic evidence for displacement in Quaternary time (last three million years), and 4) geophysical anomalies suggesting displacements in buried bedrock coincident with anomalous distribution of surficial deposits."

Cobarrubias and others (1973) classify faults as active, potentially active, and inactive. In addition, they divide potentially active into high and low potential. Active faults are "those which have shown historical activity." Potentially active faults are "those, based on available data, along which no known historical ground surface ruptures or earthquakes have occurred. These faults, however, show strong indications of geologically recent activity." High potential active faults have some or all of the following features: "a. Offsets affecting the Holocene deposits (age less than 10-11,000 years). b. A groundwater barrier or anomaly occurring along the fault within the Holocene deposits. c. Earthquake epicenters (generally from small earthquakes occurring close to the fault). d. Strong geomorphic expression of fault origin features." The low potential active faults are the same as high potential "with the exception that the indications of fault movement can be only determined in Pleistocene deposits (less than 1,000,000 years ago)." Inactive faults are "without recognized Holocene or Pleistocene offset or activity."

According to Krinitzsky (1974), faults are either active or inactive. He defines these as: "Active means that a fault may move at some time in the near future . . . Inactive means that it will not." In addition, he gives the criteria of the U.S. Nuclear Regulatory Commission with two additional criteria along with geomorphic criteria of the International Atomic Energy Agency (see another part of

this section for these criteria). He states that the criteria of the two agencies are "indicative of a present-day hazard from renewed movement along a fault, and they are sufficient to identify a fault as active."

Krinitzsky (1974) discusses a pertinent topic on the relationship between active faults and earthquakes in the Texas coastal area of the Gulf of Mexico. He states: "some faults that are moving at present, and therefore are active, need not be the cause of earthquakes." In this area, numerous Tertiary and younger faults of the Gulf Coastal Plain generally parallel the coast of the Gulf of Mexico, yet on the seismic risk map for conterminous United States (NOAA and USGS, 1982), this area has zero risk, which indicates little or no seismic activity. The mechanism for faulting (Bruce, 1972) indicates regional contemporaneous fault development through sedimentary processes that began in the Tertiary and are continuing to the present.

Nichols and Buchanan-Banks (1974) in defining an active fault imply that the definition should not be rigid, but should relate to the use and importance of structures. They state: "The definition of what constitutes an 'active fault' may vary greatly according to the type of land use contemplated or to the importance of the structure. For example, the Atomic Energy Commission regards a fault as active or 'capable' with respect to nuclear reactor sites if it has moved 'at or near the ground surface at least once in the past 35,000 years', or 'more than once in the past 500,000 years' . . . A definition for purposes of town planning in New Zealand defines as active, any fault on which movement has taken place at least once in the last 20,000 years . . . Commonly, faults are regarded as active and of concern to land-use planning when there is evidence that they have moved during historical time or, through geologic evidence, there is a significant likelihood that they will move during the projected use of a particular structure or piece of land. Because geologic evidence may be lacking, obscure, or ambiguous as to specific times of past movement, geologists may be able to estimate relative degree of activity only after a regional analysis that may extend far beyond the locality under consideration. Such analysis may be based on historic evidence of fault movement, seismic activity (occurrence of small to moderate earthquakes along the fault trace even though not accompanied by obvious fault movement), displacement of recent earth layers (those deposited during the past 10,000 years),

and presence of geomorphically young, fault-produced features (scarps, sag ponds, offset stream courses, and disruption of man-made features such as fences and curbs) . . . ."

<u>Sherard and others (1974)</u> define an active fault (or a potentially active fault) as "one on which there is sufficient evidence of displacement within the recent geological past to make it reasonable to anticipate that future surface displacements could occur within the lifetime of a dam (about 100 years)."

<u>Wesson and others (1975)</u> for the San Francisco region assume that "if a fault has been active over a considerable length of time (millions of years) and has been historically active or shows evidence of movement in the geologically recent past, it will most likely sustain movement in the future." These faults are commonly referred to as active faults, and are "characterized by at least one or more of the following features: (1) historical earthquakes with or without surface fault displacement, (2) ephemeral physiographic features such as sag ponds, offset streams, and linear ridges that suggest recent fault displacement, and (3) offset Holocene and Pleistocene deposits and geomorphic features."

<u>Slemmons and McKinney (1977)</u> summarize the geological literature on active, capable, and dead faults. They state: "Examination of the definitions . . . suggests the following general characteristics for <u>active fault</u>, <u>capable fault</u>, and <u>dead fault</u> . . . . An <u>active fault</u> is a fault that has slipped during the present seismotectonic regime and is therefore likely to have renewed displacement in the future. The fault activity may be indicated by historic, geologic, seismologic, geodetic, or other geophysical evidence of activity. The rates may vary from very low, with long recurrence intervals, to very high, with short recurrence intervals. The most recent offset along faults with long recurrence intervals may either be recent or ancient. Some workers compare active faults with active volcanoes, which show either historic or geologically recent activity, or are dormant, but with the potential for future activity. Definitions generally include a time indication of either the most recent offset or a recurrence interval. The most widely used definition in current engineering practice is for faults with evidence of Holocene displacement (approximately the last 10,000 years). Some definitions include the connotation that active faults may move or have

offsets during the life of man-made structures, but most workers prefer to define the term independently of applications or man-made structures . . . . The definition for capable faults is specified by the U.S. Nuclear Regulatory Commission . . . for siting nuclear reactors . . . The similar definition of the International Atomic Energy Agency . . . is summarized in Krinitzsky (1974). These definitions restrict this term to faults that have been displaced once during the last 35,000 years, or more than once during the last 500,000 years . . . . A dead fault is a fault that was active during an earlier orogenic period, but is not active within the present tectonic regime and accordingly does not offset late Cenozoic deposits or surfaces, and is not seismically active."

The following definitions of active, capable, and dead faults are from the Glossary of Geology (Bates and Jackson, 1980): "active fault - A fault along which there is recurrent movement, which is usually indicated by small, periodic displacements or seismic activity . . . capable fault - A fault defined by the Nuclear Regulatory Commission as one that is 'capable' of 'near future' movement; in general, a fault on which there has been movement within the last 35,000 years . . . dead fault - a fault along which movement has ceased."

The U.S. Nuclear Regulatory Commission (1984) does not refer to "active faults" but to "capable faults." A capable fault is defined as a fault "which has exhibited one or more of the following characteristics: (1) movement at or near the ground surface at least once within the past 35,000 years or movement of a recurring nature within the past 500,000 years, (2) macro-seismicity instrumentally determined with records of sufficient precision to demonstrate a direct relationship with the fault, (3) a structural relationship to a capable fault according to characteristics (1) or (2) of this paragraph such that movement on one will be reasonably expected to be accompanied by movement on the other."

In addition to the numerous publications used in this report, additional publications were summarized by Slemmons and McKinney (1977). These summaries and references are quoted from Slemmons and McKinney.

Willis (1923): ". . . two classes of faults are distinguished: active and dead. These terms are used very much in the sense in which we speak of active volcanoes or

dead volcanoes. An active fault is one on which a slip is likely to occur. A dead fault is one on which no movement may be expected . . . . Wood designated as active all faults on which there has been a movement within historical time, and also all faults upon which physiographic evidence of recent surface dislocation--'trace' phenomena--could be obtained . . . . Hence, any fault that is related to a growing mountain is reasonably subject to the suspicion of being an active fault in the sense that a slip may occur."

Cluff (1964): "An active fault is a fault that shows conclusive evidence of movement in Recent geologic time. To be classed as active, the fault must cut the Recent deposits such as alluvial fans or alluvium, have had ground rupture during historic times, or show seismologic evidence (epicentral plots along the fault) of activity. Many of the known active faults mapped today show all three types of evidence, i.e., geologic, historic, seismologic."

Bonilla (1967 and 1970): "An active fault can be defined as one that has moved in the recent past and may move in the near future. The 'recent past' as used here includes the current hour and extends back an indefinite time that many geologists would take to include at least the Holocene Epoch (about 10,000 years). The 'near future' as used above includes a length of time on the order of the useful life of engineering structures or the time span considered in long-range plans for the future. The determination of whether a fault is 'active' as defined above involves geology, geophysics, geodesy, and engineering. Some criteria currently in use are (1) the occurrence of earthquakes that can be related to the fault with reasonable assurance; (2) one or more episodes of surface rupture (including tectonic creep) or acute bending in the recent past as defined above; (3) instrumental evidence of elastic and inelastic strain; and (4) structural coupling to another fault (or other tectonic feature such as a monocline) that is active. At present, some active faults may not be identifiable, but the ability to identify them should improve with time."

Wentworth et al. (1969): "A fault is active if because of its present tectonic setting, it can undergo movement from time to time in the immediate geologic future. This active state exists independently of the geologist's ability to recognize it . . . .

". . . selection of the criteria used to identify active faults for a particular purpose must be influenced by

the consequences of fault movement on the engineering structures involved . . . .

"Positive identification of specific faults as active, or as sufficiently active to be of concern for a particular engineering problem, is not possible except for those few faults that have exhibited repeated activity in historic or very recent geologic time . . . . In the following discussion, all faults exhibiting evidence of late Quaternary movement with appropriate length to site-distance ratios are included as worthy of consideration . . . ."

Wentworth, Ziony, and Buchanan (1970): "A fault is active if, because of its present tectonic setting, it can undergo movement from time to time in the immediate geologic future. This active state exists independently of the geologist's ability to recognize it. Geologists have used a number of characteristics to identify active faults, such as historic seismicity or surface faulting, geologically recent displacement inferred from topography or stratigraphy, or physical connection with an active fault. However, not enough is known of the behavior of faults to assure identification of all active faults by such characteristics . . . .

"Selection of the criteria used to identify active faults for a particular purpose must be influenced by the consequences of fault movement on the engineering structures involved."

International Atomic Energy Agency (1972): The following summary is from Krinitzsky (1974) - "The International Atomic Energy Agency criteria are similar to the U.S. criteria, but add:

"(a) Evidence of creep movement along a fault. Creep is slow displacement not necessarily accompanied by macroearthquakes.

"(b) Topographic evidence of surface rupture, surface warping, or offset of geomorphic features.

"They would further classify active faults on a geomorphic basis as follows:

"Class A - High rate of movement, greater than 1 m per 1,000 year.

"Class B - Topography shows clear evidence of dislocation.

"Class C - Topography shows indistinct evidence of dislocation.

"Class D - No evidence of amount or rate of dislocation on which quantitative assessment can be based, but fault is considered capable of causing surface faulting.

"In general engineering practice a fault is considered to be active if there is displacement within Holocene deposits regardless of datable evidence. (Examples: fault displacements within surface gravels, alluvium, or glacial outwash.)"

Ziony, Wentworth, and Buchanan (1973): "Faults of a region can be ranked according to likelihood of future movement on the assumption that those with the most recent displacements probably have relatively short recurrence intervals. Selection of those faults considered to be active, and thus requiring more detailed site investigations, must be influenced by the consequences of possible displacement on the engineering works involved. For example, all faults in the region with proved or likely movement during late Quaternary time (past 500,000 years) may be considered active for purposes of siting nuclear power reactors and other structures that require large safety factors, whereas many such faults might not be considered active for less critical land uses."

Grant-Taylor et al. (1974): "A Class I Active Fault is one that has shown repeated movement over the last 5,000 years, but may also include those with a single movement in the last 5,000 years and repeated movement in the last 50,000 years (Officers Geological Survey, 1966). A Class I fault moves sufficiently often and the displacement that occurs is so large that it has definite planning relevance. Officers Geological Survey (1966) states: 'Class I Active Faults are liable to movement of up to 4.5 m in a period of time that could be the same as the life of a structure. As no structure can hope to withstand such a dislocation it is recommended that no structures be built across the trace of a Class I Active Fault, or across the presumed continuation.'"

California Div. of Mines and Geology (1976): Alquist-Priola Act. "Active faults are those faults which have had surface displacement within Holocene time (about the last 11,000 years). Such faults are considered as active and hence as constituting a potential hazard."

Lensen (1976): "An active fault can be defined as a fault that has moved in late geological time and will move again. The N. Z. Geological Survey (Report N.Z.G.S. 7, 1966) defines an active fault more precisely as a fault 'along which there is either evidence of movement since the

beginning of the last Glaciation (50,000 years ago) or evidence of repeated movement in the last 500,000 years.'

"The main advantage of classifying active faults lies in assessing their liability to future movement, based on the assumption that a fault that has moved frequently in the immediate geological past is likely to move with a similar frequency in the future.

"Classification of active faults must thus be designed to reflect the history of past fault movements and classes with high, medium, and low frequency of demonstrable past activity can be assigned.

"The classification of active faults adopted by the N. Z. Geological Survey is shown below:

"A <u>Class I Active Fault</u> is thus either a fault that has shown repeated movement over the last 5,000 years, or with a single movement over that period and repeated movement in the last 50,000 years.

"A <u>Class II Active Fault</u> is less active with either repeated movement over the last 50,000 years, or one single movement in the last 5,000 years and repeated movement in the period of 50,000 to 500,000 years.

"A <u>Class III Active Fault</u> is the least active with either a single movement over the last 50,000 years or repeated movement during the 50,000-500,000 year period."

<u>U.S. Bureau of Reclamation (1976)</u>: The U.S. Bureau of Reclamation is considering for some regions the use of new criteria for the basis of a definition in which "active faults" are defined as those "that have exhibited relative displacement within the past 100,000 years."

(In the MKE UMTRA DESIGN PROCEDUE, Chapter 8,)

• I proposed adoption of this definition for the UMTRA project. However, if NRC prefers adoption of their definition of Capable Fault/Active Fault, I do not find any strong reason to object to that.

• In those sites where the terrain is flat, and the surface features are disturbed by human activity, and no trace of surface faulting has been identified in the past, establishing any subsurface fault activity will be difficult, no matter which definition of active fault is selected. Identifying fault from (surface) geologic evidence will depend (more) on the interpretational skill of the investigator.

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APPENDIX B

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APPENDIX C

Joyner, W.B. & Boore, D.M., 1981, Peak Horizontal Acceleration and Velocity From Strong Motion Records Including Records From the 1979 Imperial Valley, California, Earthquake, Bulletin of the Seismological Society of America, Vol. 71, No. 6, pp. 2011-2038, December.

Data base included 182 horizontal motion components from 23 earthquakes (20 in California, two in Alaska and one in Nicaragua). All events were shallow, with fault rupture being at a depth of 20 km or less, and had moment magnitudes (some calculated from seismic magnitude, and some assumed equal to local magnitude) of 5.0 to 7.7. Source distances of less than 1 km to 370 km were considered; for the larger distances, an accuracy of 5 km was required for a record to be considered. Acceleration is the larger peak value of the two horizontal components.

Derived the expression:

$$\log a = -1.02 + 0.249M - \log R - 0.00255\,R + 0.26\,P$$

where a = acceleration as fraction of gravity
M = magnitude
$R = (d^2+7.3^2)^{\frac{1}{2}}$
d = closest distance in km from the recording site to the surface projection of the fault rupture (fault distance)
P = 0.0 for 50 percent probability that the prediction will exceed the real value, and 1.0 for 84 percent probability

The standard deviation for log a is 0.26, and the standard error is 0.04. The authors also determined the sensitivity of the expression to individual earthquake events by deriving expressions with one or more events eliminated. Found that the expression is not sensitive to particular events but that the standard deviation is sensitive. Site conditions (rock versus soil) were considered but were determined to not have a significant effect on the expression for acceleration.

Campbell, K.W., 1981, Near-Source Attenuation of Peak Horizontal Acceleration, Bulletin of the Seismological Society of America, Vol. 71, No. 6, pp. 2039-2070, December.

Data base included 229 horizontal motion components from 27 earthquakes worldwide. Depth to fault rupture for events considered was 25 km or less. Moment magnitudes of 5.0 to 7.7 were considered, where magnitude (M) was defined as surface wave magnitude (M_s) when both M_s and local magnitude (Ml) were greater than 6.0, and defined as ML when M_s and ML are both less than 6.0. Source distances of less than 50 km were considered, but were coupled with magnitude. Thus, source distances were less than 20 km for M <4.75, less than 30 km for $4.75 \le M \le 6.25$, and less than 50 km for M 6.25. For larger distances, an accuracy of 5 km for source distance was required.

Acceleration is the mean of the peak values of the two horizontal components. The authors also considered the location of the recording instrument and the local geologic conditions underlying the recording site. To prevent skewing of the regression analysis by particular events (24 of the 116 records were for the 1971 San Fernando event and 31 were for the 1979 Imperial Valley event), the records were weighted as a function of distance and number of records.

Derived the unconstrained relationship:

$$a = 0.0159e^{0.868M} \quad R+0.0606e^{0.700M} \quad {}^{-1.09}$$

where a = acceleration as fraction of gravity
M = magnitude
R = closest distance in km from the recording site to the surface projection of the fault rupture (fault distance)

and a constrained expression:

$$a = 0.0185e^{1.28M} \quad R+0.147e^{0.732M} \quad {}^{-1.75}$$

with same terms as the unconstrained expression.

The expressions predict mean values of the average of the peak values of the two horizontal components. If an

estimate of the mean peak value independent of component direction is required, the author stipulates an additional factor of 1.13 be applied to the predicted acceleration. The 84 percentile value of the predicted acceleration is determined by multiplying the mean value from the unconstrained relationship by 1.45, and the mean value from the constrained relationship by 1.47. The standard error is 0.372 for ln a in the unconstrained relationship, and 0.384 for ln a for the constrained relationship.

For the constrained model, as compared f to the unconstrained model, the exponent of the (R + f(M)) term was set equal to -1.75 to account for far-field effects (greater than 30 or 50 km). For near-field events (fault distances closer than 3 to 5 km), the constrained model required a constant peak acceleration, independent of magnitude, at the fault rupture surface. This is in conformance with generally accepted physical interpretations that peak acceleration in the near-field is controlled by dynamic stress-drop.

Donovan, N.C., 1973, Earthquake Hazards for Buildings, in Building Practices for Disaster Mitigation, National Bureau of Standards, Building Science, Series 46, p. 82-111.

Date base included 515 strong motion records (no other definition provided). Site to source distances varied from 3 to 2,000 km, though most were in the range of 10 to 400 km. Acceleration is peak ground acceleration, but otherwise undefined, and the Richter magnitude scale is used.

Derived the expression:

$$a = 1320e^{0.58M} \quad R+25 \quad {}^{-1.52}$$

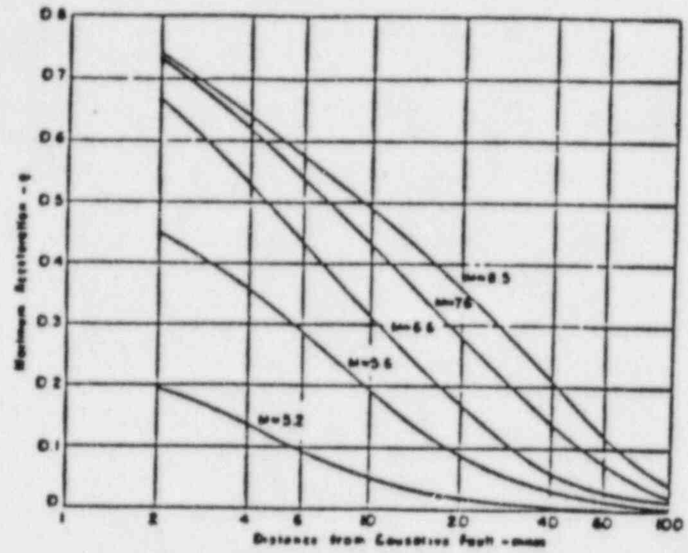
where a = acceleration in cm/sec^2
M = magnitude
R = hypocentral distance, distance to causitive fault or distance to center of energy release in km.

The expression predicts the mean value of the peak acceleration. The standard deviation for ln a is 0.84.

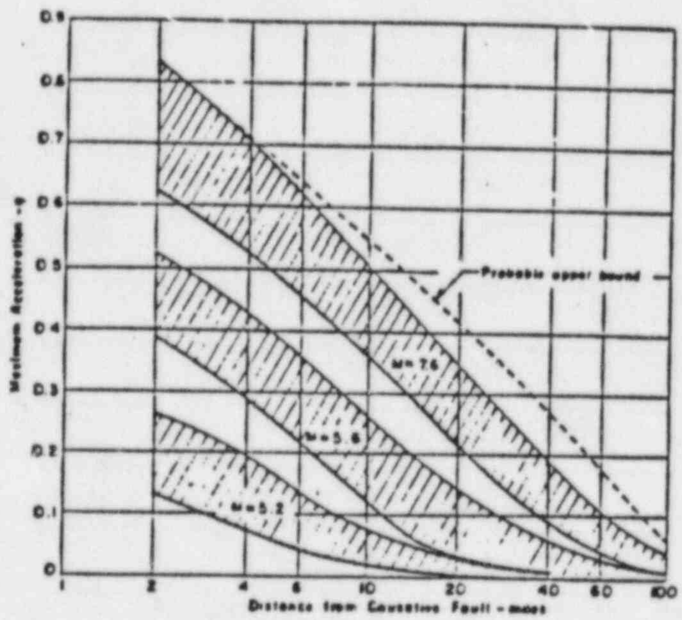
Schnabel, P.B. and Seed, H.B., 1973, Accelerations in Rock for Earthquakes in the Western United States, Bulletin of the Seismological Society of America, Vol. 63, No.2, pp. 501-516, April.

The data base is not well defined, and the reader is referred to the article. Acceleration is defined as the maximum value, magnitude is presumed to be the Richter value, and distance is the distance from the causitive fault.

Attenuation relationships are presented graphically, with mean values and a range in values plotted. These are shown on the next page. It is noted these relationships are for rock acceleration. The graphical presentation was later extended to other geologic site conditions by Seed and others (1975).



Average values of maximum accelerations in rock.



Ranges of maximum accelerations in rock.

Blume, J.A., 1977, The SAM Procedure for Site-Acceleration-Magnitude Relationships, Proceedings of the Sixth World Conference on Earthquake Engineering, Vol. 1, pp. 416-422.

Data base included all United States Earthquakes (U.S. Department of Commerce, 1933 through 1970) data. Magnitude is as given in the listing, but typically Richter magnitude, and acceleration is peak acceleration. Author assumed an average focal depth of 8 km.

Derived expression for M $\leq$6.5:

$$a = 0.318e^{1.03M}\ 29^{1.14b}(R+25)^{-1.14b}2.53^{y}$$

and for M >6.5:

$$a = 26.0e^{0.432M}29^{1.22b}(R+25)^{-1.22b}1.81^{y}$$

where a = peak ground acceleration in gallons
M = Richter magnitdue
R = hypocentral distance in km
b = 0.5 log (pV_s)
p = site specific density
V_s = site shear velocity in ft/sec
y = standard normal variable with zero mean and unit standard deviation (note: for mean value of a set y = 0, and for 84 percentile use y = 1.0)

The author also developed relationships for various portions of the data set, including distance and acceleration constraints or cutoffs. For general comparison with other data, he suggests the use of pV_s = 2,000 ft/sec, thus b = 1.65.

Bolt, B.A., 1982, Methods of Estimating Attenuation and Effective Peak Acceleration in the Near Field, Proceedings of the Third International Earthquake Microzonation Conference, Vol. 1, pp. 131-144.

Data base is described in Joyner and others (1981), to which the reader is referred, but is primarily records of California events.

Derived expressions of the form

$$a = A \ (R+D)^2+1 \ {}^{C}e^{-B(R+D)}$$

where a = peak acceleration as fraction of gravity
R = closest distance to the surface projection of the rupture.

and the parameters A, B, C and D are dependent on magnitude as follows:

M	A	B	C	D	Standard Error
5<M<6	1.2	0.066	0.033	23	0.06g
6.0<M<7.0	1.19	0.044	0.042	24.9	0.10g
7<M<7.7	0.24	0.022	0.10	15	0.05g

Donovan, N.C. and Bornstein, A.B., 1978, Uncertainties in Seismic Risk Procedures, Journal of the Geotechnical Engineering Division, Vol. 104, GT7, pp. 869-887, June.

Data base included 69 records from nine earthquake events, with Richter magnitude varying from 5.0 to 7.7. Other definition of the data base is not presented, except that over half the records are from the 1971 San Fernando earthquake. The authors indicate, however, this does not bias the results. For the California events considered, the focal depth was assumed to be 5 km. Distances varied from less than 1 to 321 km. Acceleration is peak horizontal ground acceleration.

Derived the expression:

$$a = b_1, e^{b_2 M}(R+25)^{-b_3}$$

where a = acceleration in cm/sec^2
M = Richter magnitude
R = distance to energy center of the causitive fault in km
$b_1 = 2,154,000\ R^{-2.10}$
$b_2 = 0.046+0.445 \log R$
$b_3 = 2.515 - 0.486 \log R$

The standard deviation was found to vary with the value of acceleration determined. Suggested values are as follows:

a, %g	Log Normal Standard Deviation
0.01	0.5
0.05	0.48
0.10	0.46
0.15	0.41
0.20	0.37
0.30	0.30
>0.30	0.30

The 84 percentile value for 0.10g is calculated as 0.1 exp (0.46), or 0.16g.

ATTACHMENT - A

(Part of MKE'S Review Comments Report of 14 June 1983)

Development of Maximum Credible Earthquake (MCE):

There is no universally accepted definition of MCE. Also, after reviewing various definition used by the engineering community we recognized that no single definitions of MCE was easy to adopt for the UMTRA Project. Therefore, the definition of MCE developed by MKE for UMTRA Project sites is presented in Ref. ~~12~~ 2, Chapter 8. Our definition of MCE could be adopted for sites where MCE is developed from capable/active faults identified in the vicinity of the UMTRA sites, or for sites where surface traces of active faults are lacking and the MCE has to be developed by the Probatilistic Method.

Design Earthquake and Related Parameters

Besides the selection of the MCE, we do not find any discussion in your report about the selection of the design earthquake and/or related parameters for analysis.

As is well recognized, the characteristics of the MCE event cannot be directly used for the design and analysis of UMTRA facilities. For the same MCE event at a particular site, the choice of the design earthquake and the related parameters are influenced by a number of factors, such as site conditions, material (tailings and foundation) properties and the method of analyses. Thus there

sometimes is a need for selecting more than one design earthquake or more than one set of design parameters suited to the type of analysis (Ref. 14). Or in other words, for the same MCE event, the design parameters will be different for liquefaction analysis, full scale dynamic slope stability analysis and/or pseudo-static slope stability analysis. This is also discussed in Ref. 2, Chapter 8. For compacted tailings piles, which will be subjected to little or no strength loss during an earthquake, a fraction of the peak acceleration (1/2 to 2/3 of a maximum acceleration ~~in~~ is selected by MKE) is considered adequate for the design horizontal seismic coefficient used in the pseudo-static slope stability analysis. A design horizontal seismic coefficient less than 1/3 of the maximum acceleration has been sometimes recommended for the pseudo-analysis of critical structures (Refs. 14, 15, 16 & 17). For the convenience of the reader, the maximum acceleration and the corresponding design seismic coefficient recommended for some critical structures by Newmark and Seed are presented in Tables 2 and 3. A note on the stability analysis of dams in Japan by the seismic coefficient method (Ref. 18) is also included as Appendix B for the convenience of the reader.

Some Suggestions

We present several suggestions which we believe will help develop a realistic program of seismic hazard evaluation, and also expedite review and approval of seismic design parameters and procedures.

1. The UMTRA Project could benefit from utilizing quality data available with various public agencies. For example, seismic studies for the BVPS Nuclear Power Station, Pennsylvania could have been used to advantage for the Canonsburg and Burrell sites in Pennsylvania. Similarly, a wealth of data is available from the U.S. Army Corps of Engineers, Portland, Oregon, who have performed seismicity studies for a number of dam projects not far from the Lakeview, Oregon site.

2. ERTS imagery/LANDSAT imagery and Radar imagery (SLAR) are readily available for all sites from the EROS Data Center and could be used for faults and lineaments studies and made an integral part of seismic hazard evaluation reports. Such exhibits will help regulatory agencies review the nature and extent of studies performed for each site, and thus have greater level of confidence in the completeness and quality of the studies.

3. The nature and extent of studies performed at each site should be clearly spelled out in the report and backed up by data and exhibits.

4. Exclusion from some of the stringent siting criteria spelled out in 10 CFR 50 and 100 (Ref. 3) will require a very well documented presentation to the DOE and the NRC.

5. An outside consultant like Professor David B. Slemmons of University of Nevada may be engaged to overview the study of active faults and detrmination of MCE by SHB. Utilizing Professor Slemmons' strong background in active fault studies will result in significantly reduced field work and greater level of confidence in the results of fault studies.

6. It is not enough to provide the Richter Magnitude and peak acceleration of the MCE event for each site. The reports should contain the following:

   a. The summary of findings.
   b. Useable design parameters for:

      - Liquefaction analysis
      - pseudo-static slope stability analysis of embankments.
      - full scale dynamic slope stability analysis (may not be required for any site)

o simplified dynamic slope stability analysis

o and the design of ductile structures (concrete and steel) by the UBC procedures.

Obviously the parameters in (b) above are dependant not only on the characteristics of the MCE event but also on the type of materials and methods of design. A suitable format for presenting these data is shown in Table 8-1 (Ref. 2, Chapter 8).

7. The report should also contain a section on landslides and on ground rupture potential of each UMTRA site and vicinity; the mechanism inducing these instabilities could be natural phenomena, like, earthquakes, rain, snow, flood and volcanic eruptions.

Earthquake Acceleration Attenuation with Distance:

The following earthquake acceleration attenuation relationship with distance has been proposed by Iwasaki, et. al. (1978, Refs. 10, p. 44) for average soil conditions based on the study 300 records.

$$a_{max} \text{ (gals)} = 18.4 \times 10^{0.302M} \times R^{-0.8} \quad .... \quad \text{(A-1)}$$

where a_{max} = maximum ground acceleration (gals)

R = epicentral distance (km)

and M = earthquake magnitude (Richter).

Like most empirical equations, there are some limitations on the ranges of applicability of this equation. Since this equation is developed based on the study of a large number of earthquake records it essentially incorporates the effects of wide variations in soil conditions, geological variations along the wave paths and within the fault zone. Therefore, it should be considered as a welcome addition to the list of selected attenuation relationships. Except at very close distances ($\leq$ 5km), the ground acceleration obtained from this equation (Table 1 inside text) is in close agreement with the acceleration obtained using the attenuation relationship after Joiner and Boore (1981, Ref. 13). It is also intersting to observe that the acceleration values obtained from this equation are in fairly good agreement with the mean acceleration values computed, based on eight attenuation relationships selected by SHP (Ref. 1). The wide variation in the peak acceleration values at close distances ($\leq$ 5km) as obtained by using various attenuation relationships is understandable but unavoidable; as the earthquake data population at such close distances being small, any data interpretation at this range is strongly biased by individual judgement of the researchers.

A Note on the Seismic Coefficient Method of Design of Dams in Japan

In the so-called seismic coefficient method of dam design, the weight of the dam body itself and a part of reservoir water determined by the formula of dynamic water pressure are multiplied by the seismic coefficient, and the value obtained is treated as the earthquake force. These forces of inertia are applied horizontally to the dam body to calculate stresses and stability. This method has been in use in Japan since the time high dams began to be constructed in that country.

This method has been improved based on studies and research on earthquake phenomena and behavior of dams during earthquakes. The seismic coefficients selected for dam design are at present determined by various factors such as the type of dam, geological conditions, and occurrence of historic earthquakes in the vicinity of the proposed dam. Table-B1 shows the design seismic coefficients established by the Japanese National Committee on Large Dams (Ref. 18, p. 20). The values are classified by the types of dams and the regions in which the proposed dams are to be located.

TABLE B-1 - DESIGN SEISMIC COEFFICIENTS FOR DAMS IN JAPAN*

(Ref. 18, p. 21)

Type of Dam	Part of Tohoku region; Kanto region; Chubu region; Kinki region; Southern Shikoku region	Hokkaido region; Hokuriku regions Other part of Tohoku region; Chugoku region; Northern Shikoku region: Kyushu region
Concrete Dams and Rock-Fill Dams	$k = 0.12 \sim 0.20$	$k = 0.10 \sim 0.15$
Earth Dams	$K = 0.15 \sim 0.25$	$k = 0.12 \sim 0.20$

One half of the values shown in Table B-1 could be adopted for design when the reservoir is empty because under this condition the damages caused by an earthquake would not be serious.

For embankment dams or concrete gravity dams, the seismic coefficient of the dam body is assumed to be equal to that of the foundation. However, in some cases involving very high dams, the seismic coefficient is increased in the dam body in consideration of amplification effects.

TABLE - 2

SELECTION OF DESIGN EARTHQUAKE PARAMETERS FOR CRITICAL STRUCTURES

Peak acceleration of Maximum Credible Earthquake (MCE)/Seismic Safety Evaluation Earthquake	Recommended Peak Acceleration of Seismic Engineering Design Earthquake	Comments & References
0.50g ~ 0.60 (Use for liquefaction analysis)	0.10g ~ 0.15g (Use for pseudo-static analysis)	According to Seed (Ref. 14, p.59): ". . . Both theory and experience show that this is perfectly reasonable procedure."
1.0g[a]	0.30g ~ 0.60g (For the design of earth structures)[b] 0.18 ~ 0.35g (For the design of steel structures)	[b]Specified by Newmark and Hall (1973) (Ref. 15)

a. Specified by Page R.A et. al., US&GS (1972, Ref. 17).

TABLE - 3

DESIGN CRITERIA FOR THE PSEUDO-STATIC ANALYSIS OF EMBANKMENTS
[After Seed (1979), Ref. 16, p. 236]

Earthquake Magnitude	Design Criteria	Comments & References
6-1/2	FS = 1.15 for Seismic Coefficient = 0.1	a) Applicable for embankments constructed of soils which do not build up large pore pressures due to earthquake shaking nor show more than 15% strength loss (usually).
8-1/4	FS = 1.15 for Seismic Coefficient = 0.15	

[Attachment - E]

JUN 14 1985

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MEMORANDUM FOR: Leo B. Higginbotham, WMLU
Division of Waste Management

FROM: Malcolm R. Knapp, WMGT
Division of Waste Management

COMMENTS: DRAFT STANDARD REVIEW PLAN FOR GEOLOGIC-SEISMOLOGIC REVIEWS OF UMTRAP DOCUMENTS

Pursuant to Technical Assistance Request TAR-85023, we are pleased to submit to you a draft of the Standard Review Plan (SRP) for Geologic-Seismologic Reviews of UMTRA Documents (i.e., Remedial Action Plans). This SRP was prepared and coordinated by Jose Valdes (geology) with support from Michael Blackford (seismology).

The draft SRP contains separate sections on the statutory basis for the geologic and seismologic reviews of RAP's that we consider to be an integral part of the justification of the roles of these disciplines in the UMTRAP review process. If, as you have indicated, WMLU plans to merge this SRP with that of WMEG, retention of these separate sections might need to be discussed prior to the next iteration. In that case, we would want to incorporate these sections into the appropriate technical sections of the revised document.

WMGT would like to emphasize that the present version of this SRP is intended to be a working draft, as the document has not undergone peer review beyond the branch. Aspects of the seismic hazard analysis review plan have been recently developed by Mr. Blackford and we are seeking peer review from seismologists in NRC and in our contractor pool. We look forward to receiving your comments on this draft. If you have any questions about the SRP, please contact Philip Justus.

Original Signed By

Malcolm R. Knapp, Chief
Geotechnical Branch
Division of Waste Management

Attachment: 4005-GEN-R-09-00663-00

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NAME	:JValdés	:PJustus	:MBlackford	:MRKnapp	:	:	:
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DRAFT STANDARD REVIEW PLAN FOR GEOLOGIC AND SEISMOLOGIC REVIEWS OF UMTRAP DOCUMENTS

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1. UMTRCA DOCUMENTS (Remedial Action Plans)

1.1. Statutory Basis for Reviews

1.1.1 Uranium Mill Tailings Radiation Control Act of 1978 as Amended (UMTRCA), PL 95-604

Title I of the Uranium Mill Tailings Radiation Control Act of 1978 as amended (UMTRCA), provides the regulatory framework for the Uranium Mill Tailings Remedial Action Project (UMTRAP). Under UMTRCA, Congress authorized a program of assessment and remedial action at designated inactive uranium mill tailings sites to stabilize and control such tailings in a safe and environmentally sound manner and to minimize or eliminate radiation health hazards to the public. Remedial actions, which are proposed in the form of Remedial Action Plans (RAP's), are to be selected and performed by the Department of Energy (DOE) with the concurrence of the Nuclear Regulatory Commission (NRC) and in accordance with the standards prescribed by the Environmental Protection Agency (EPA) [Sec. 108]. The DOE's authority to perform remedial actions under UMTRCA terminates 7 years after promulgation of the EPA standards unless Congress grants an extension [Sec. 112]. As the EPA standards became effective on March 7, 1983, this means that the UMTRA Project is to be completed by 1990.

The Act requires that, upon completion of remedial actions at a given site, the DOE maintain institutional control of the property in accordance to a license issued by the NRC [Sec. 104(f)(2)]. The NRC may require in its license that the DOE undertake a program of monitoring, maintenance, and emergency measures as necessary to protect public health and safety.

UMTRCA also addresses the matter of post-remediation disruption of tailings piles for purposes of natural resources recovery during the period of institutional control. Section 104(h) states that:

> "No provision of any [DOE-State cooperative] agreement under section 103 shall prohibit the Secretary of the Interior, with the concurrence of the Secretary of Energy and the [Nuclear Regulatory] Commission, from disposing of any subsurface mineral rights by sale or lease... which are associated with land on which residual radioactive materials

> are disposed and which are transferred to the United States as required under this section if the Secretary of the Interior takes such action as the Commission deems necessary pursuant to a license issued by the Commission to assure that the residual materials will not be disturbed by reason of any activity carried on following such disposition. If any such materials are disturbed by any such activity, the Secretary of the Interior shall insure, prior to the disposition of the minerals, that such materials will be restored to a safe and environmentally sound condition as determined by the Commission...."

Other parts of UMTRCA deal with legal and financial considerations for the remedial action programs which are not directly relevant to this document.

1.1.2 40 CFR 192 Standards for Remedial Actions at Inactive Uranium Processing Sites

1.1.2.1 Subpart A--Standards for the Control of Residual Radioactive Materials from Inactive Uranium Processing Sites

1.1.2.1.1 Applicability

As mandated by Section 108 of UMTRCA, amending Section 275 of the Atomic Energy Act, EPA promulgated standards applicable to the control of residual radioactive material associated with designated Title I sites and to subsequent restoration of such sites following any use of subsurface minerals under Section 104(h) of the Act. Control, as used in the current context, is defined as "any remedial action intended to stabilize, inhibit future misuse of, or reduce emissions or effluents from residual radioactive materials."

1.1.2.1.2 Standards

"Control shall be designed to:

(a) Be effective for up to one thousand years, to the extent reasonably achievable, and, in any case, for at least 200 years, and,

(b) Provide reasonable assurance that releases of radon-222 from residual radioactive material to the atmosphere will not:

   (1) Exceed an average release rate of 20 picocuries per square meter per second [over the entire surface of the disposal site and over at least a one year period], <u>or</u>

(2) Increase the annual average concentration of radon-222 in air at or above any location outside the disposal site by more than one-half picocurie per liter.

1.1.2.2 Rationale and Implementation Guidelines

Though 40 CFR 192 does not provide any prescription of how the Subpart A standards should be implemented, generalized guidelines for their implementation are presented in Subpart C and, along with the bases and rationale for the standards, in the "Supplementary Information" sections. An derstanding of this collateral information is critical, as EPA states in section IV(A) that: "It is our objective that implementation of these standards be consistent with the assumptions we made in deriving them."

The excerpts presented below have relevance in determining the scope of geologic-seismologic reviews of UMTRCA documents.

1.1.2.2.1 Supplementary Information Sections

Section II(B)(1), Cleanup and Control of Tailings

"The objective of tailings control and stabilization efforts are to prevent their misuse by man, to reduce radon emissions (and gamma radiation exposures), and to avoid the contamination of land and water by preventing erosion by natural processes. The longevity (i.e., long-term integrity) of control is particularly important. This is affected by the potential for disruption by man; by the probability of occurrence of such natural phenomena as earthquakes, floods, windstorms, and glaciers; and by chemical and mechanical processes in the piles." [Emphasis added.]

"If necessary, erosion can be inhibited by...moving them away from a particularly flood-prone or otherwise geologically unstable site." [Emphasis added.]

Section III(A)(3), Cost-Benefit Analysis

"The major hazard, the extent of possible future misuse of tailings by man, is almost impossible to quantify." [Emphasis added.]

"Under this standard most of the 24 tailings pile will be stable against erosion and casual intrusion for misuse for much longer than 1000 years." [Emphasis added.]

Section III(B)(1), Longevity of the Control

"We consider the single most important goal of control to be effective isolation and stabilization of tailings for as long a period of time as is reasonably feasible because tailings will remain hazardous for hundreds of thousands of years. The longevity of tailings control is governed chiefly by the possibility of intrusion by man and erosion by natural forces. Reasonable assurance of avoiding casual intrusion [for misuse] by man can be provided through the use of relatively thick and/or difficult-to-penetrate covers (such as soil, rock, or soil-cement). No standard can guarantee absolute protection against the purposeful works of man and these standards do not require such protection. [Emphasis added.]

Section IV(A) Guidance for Implementation

"The standard for control and stabilization of tailings piles is primarily intended as a design standard. Implementation will require a judgement that the method chosen provides a reasonable expectation that the provisions of the standard will be met . . . . This judgement will necessarily be based on site-specific analyses of the properties of the sites, candidate control systems, and the potential effects of natural processes over time, and, therefore, the measures required to satisfy the standard will vary from site to site." [Emphasis added.]

"We have concluded that primary reliance on passive measures is preferable, since their long-term performance can be projected with more assurance than that of measures which rely on institutions and continued expenditures for active maintenance." [Emphasis added.]

"As long as the Federal Government exercises its ownership right and other authorities [under UMTRCA] regarding these sites, they should not be systematically exploited by people or severely degraded by natural forces. We believe that these institutional provisions are essential to support any project whose objectives is as long term as are these disposal operations, and for which we have little experience. This does not mean that we believe primary reliance should be placed on institutional controls; rather, that institutional oversight is an essential backup to passive controls." [Emphasis added.]

Section III(B)(2) The Radon Release Limit

"We believe that limiting radon emissions from tailings piles serves several necessary functions: reducing the risk to nearby individuals and individuals at greater distances; and furthering the goals of reliable long-term deterrence

of misuse of tailings by man and control of erosion of piles by natural processes. The degree of reduction of radon emissions achieved by a disposal system is more or less directly related to the degree of abatement of each of these hazards." [Emphasis added.]

"Congress did not intend that EPA set standards for one generation only, or that it set standards without consideration of the long-term reliability of whatever means are available for implementing them. [Emphasis added.]

1.2.2.2 Subpart C, Guidance for Implementation

"The implementing agencies [including the NRC] shall establish methods and procedures to provide 'reasonable assurance' that the provision of Subpart A . . . are satisfied. This should be done as appropriate through the use of analytic models and site-specific analyses . . . ." [Emphasis added.]

"The purpose of Subpart A is to provide for long-term stabilization and isolation in order to inhibit misuse and spreading of residual radioactive materials, control releases of radon to air, and protect water. Subpart A may be implemented through analysis of the physical properties of the site and the control system and projection of the effects of natural processes over time. Events and processes that could significantly affect the average radon release rate from the entire disposal site should be considered. . . . Computational models, theories, and prevalent expert judgement may be used to decide that a control system design will satisfy the standard. The numerical range provided in the standard for the longevity of the effectiveness of the control of residual radioactive materials allows for consideration of the various factors affecting the longevity of control and stabilization methods and their costs. These factors have different levels of predictability and may vary for the different sites." [Emphasis added.]

1.2. Implementation Objectives, Review Elements and Procedures

1.2.1 Objectives

It is the staff's position that the requirements and implementation guidelines of 40 CFR 192 necessitate that due consideration be given to geologic and seismologic processes that bear on engineering and hydrologic site-suitability considerations. Consequently, geologic-seismologic reviews of UMTRCA documents shall be directed toward the following objectives:

1. Determination of the site stratigraphy as input into engineering (ground failure) reviews as well as hydrologic (groundwater flow and contaminant migration) reviews and geomorphic evaluations;

2. Evaluation of the potential for geomorphic hazards, such as landslides, subsidence, and stream encroachment;

3. Estimation of earthquake-induced ground accelerations (vibratory ground motion) that could occur at the site;

4. Assessment of the potential for ground rupture (surface faulting) that could affect the tailings pile due to fault displacement;

5. Assessment of the potential for other types of tectonic hazards (e.g., volcanic activity) that could affe the site.

6. Evaluation of the natural resources exploitation history and/or potential of the site as input into geologic stability assessments./ The goal of this evaluation shall be to determine how resource exploitation in the surface or subsurface of the site area may indirectly impact on the geologic stability of the pile, rather than to exclude the possibility of direct human disruption of the pile for purposes of resource recovery. The latter would go beyond the requirements of 40 CFR 192 (refer to Sec. III(B)(1) in particular), which are only meant to provide protection against casual human intrusion for misuse.

1.2.2 Review Elements

The staff considers the geologic-seismologic information in the documents reviewed to be acceptable if it satisfies the requirements and scope specified in this section.

1.2.2.1 Stratigraphy

Information pertaining to the formation, composition (including internal variability) sequence and correlation of the lithologic strata under the site and the region surrounding the site should be presented. The scope of stratigraphic investigations should be defined in part by the requirements of sections 1.2.2.2 and 1.2.2.3 and hydrology investigations. The level of stratigraphic understanding to be achieved shall be commensurate with the influence stratigraphy has on the determination that there is reasonable assurance that the remedial action will comply with the EPA standards.

Regional stratigraphic information may be obtained from published reports, maps, private communications or other sources. The information should be discussed, adequately referenced, and illustrated by regional surface and subsurface geologic maps, stratigraphic columns and cross sections. Sufficient detail should be provided to give clear perspective and orientation to the site-specific stratigraphic information to be presented.

Detailed data on the stratigraphic characterics of the site should be obtained from site-specific studies incorporating combinations of boring, trenching, geophysical investigations, and surface mapping. Plot plans that graphically show the locations of all site exploration localities should be provided. The limits of the site should be superimposed on the plot plans. Descriptions of the exploration and surveying techniques used should be furnished, as well as all geologic and/or geophysical logs, supplemented by ground-based and aerial photographs where appropriate.

The origin, depth, thickness, physical characteristics (e.g., color, sorting, texture), mineralogy, and degree of consolidation of each lithologic unit should be adequately described, noting zones of alteration or weathering profiles. The relationship of the site stratigraphy to the regional stratigraphy should be discussed. Selective stratigraphic cross sections (and/or fence diagrams) should be provided that incorporate the location of the borings or other exploratory locations from which the information in the sections was derived (i.e., "idealized" cross sections not based on discrete site-specific data are not adequate for the purpose of site characterization).

1.2.2.2 Geomorphic Hazards

Geomorphic investigations should include systematic analysis of regional and local landforms to provide evidence of geomorphic processes that may influence the stability of the site. As appropriate, such analysis should take into account the information discussed in sections 1.2.2.1 and 1.2.2.3 and that derived from hydrological investigations. The level of understanding of geomorphic processes to be achieved should be commensurate with the influence the processes have on the determination that there is reasonable assurance that the remedial action will comply with the EPA standards.

Chapter 5 of NRC NUREG/CR-3276 (Schumm and Chorley, 1983) provides a useful generic outline of standard procedures and methods for geomorphic site evaluations. In general, the physiographic (geomorphic) province(s) in which the site is located should be identified and described. This description should expound on the areal extent, distinguishing characteristics (e.g., elevation, relief) and major active processes modifying the present-day topography of the province(s) and should be supplemented by means of pertinent large and small scale topographic maps (e.g., USGS 7.5-minute and 2-degree USGS quadrangle maps).

Site-specific characterization studies should include aerial photography and detailed topographic mapping of the site and its vicinities. Topographic mapping of the disposal site area should be at a scale on the order of 1:2400, with a contour interval on the order of 1 foot. Such maps should be utilized

to generate geomorphic-hazards maps that delineate areas where landscape changes associated with drainage networks, slopes, rivers and piedmonts (as discussed in NUREG/CR-3276) may adversely affect site stability. Areas that may be subjected to subsidence due to natural or man-made subsurface conditions should also be identified (subsidence related to tectonic processes is addressed under section 1.2.2.3.3). Delineation of such areas should take into account the various factors influencing geomorphic processes such as relief, landform morphology, near-surface geology-pedology, and resident biota. Each relevant geomorphic process identified should be described, including (1) rate of activity, (2) frequency of occurrence, and (3) specific controlling mechanisms or factors.

1.2.2.3 Seismic and Tectonic Hazards

The level of understanding of seismic and tectonic processes to be achieved should be commensurate with the influence these processes have on the determination that there is reasonable assurance that the remedial action will comply with 40 CFR 192.

1.2.2.3.1 Vibratory Ground Motion

The staff considers the derivation of the maximum credible earthquake (MCE) and the resulting ground motion at the site to be acceptable if the processes and procedures in this section are followed. This does not mean, however, that NRC will exclude from consideration other methods and approaches to seismic hazard analysis that can be demonstrated by DOE to adequately address the requirements of 40 CFR 192.

1.2.2.3.1.1 Basic Information and Investigations Required

The required information and investigations provide an adequate basis for selection of the maximum credible earthquake (MCE), as defined in 10 CFR 40, App. A, Criterion 4(e), and determination of the resulting vibratory ground motion at the site. The size of the region to be investigated and the type of data pertinent to the investigations should be guided by the requirements of section 1.2.2.3.1.2. Data should be obtained by standard photogeologic analysis and field reconnaissance of the study area and from review of the pertinent literature. Investigative activities and technical information relating to the site should include the following:

(1) Determination of the structural geologic conditions at the site and the region surrounding the site, including its tectonic history;

(2) Identification and description of tectonic structures, particularly faults, underlying the site and the region surrounding the site, whether buried or exposed at the surface. (As used in this document, the terms "tectonic structure" and "fault" have the same meanings as defined in 10 CFR 100, App. A III (i) and (e), respectively.)

(3) Listing of all recorded earthquakes that have occurred in the tectonic province (as defined in 10 CFR 100, App. A III(h)) or provinces that would be expected to influence the local seismicity. This listing should include the date of occurrence of the earthquake, its magnitude, and the location of the epicenter. Since earthquakes have been reported in terms of various parameters such as intensity at a given location, and effect on ground, structures and people at a specific location, some of these data may have to be estimated by use of appropriate empirical relationships;

(4) Identification of epicenters or locations of highest intensity of historically reported earthquakes, where possible, with tectonic structures. Epicenters or locations of highest intensity which cannot be reasonably identified with tectonic structures shall be identified with tectonic provinces.

1.2.2.3.1.2 Determination of Seismic Hazard Parameters

Selection of the maximum credible earthquake (MCE) and of the resulting ground motion at a site should incorporate the following steps:

Step (1), Determination of the Maximum Peak Horizontal Acceleration for Earthquakes Unassociated with Known Structures. For those earthquakes not associated with a known tectonic structure, the largest event that has occurred in each of the tectonic provinces that would be expected to influence the seismicity of the site should be identified. For each of these earthquakes, the peak horizontal acceleration at the site should be determined by using an accepted state-of-the-art attenuation relationship between earthquake magnitude and distance. Joyner and Boore, 1981, Campbell, 1982, and Nuttli, 1983, in Bernreuter et al., 1984, are examples of acceptable relationships. In applying these relationships, site-to-epicenter distances for each earthquake should be set equal to the larger of two values: (a) 10 km, or, (b) the closest actual distance of a given province from the site. The acceleration value adopted should be the mean-value plus one-standard-deviation (i.e., 84th percentile value).

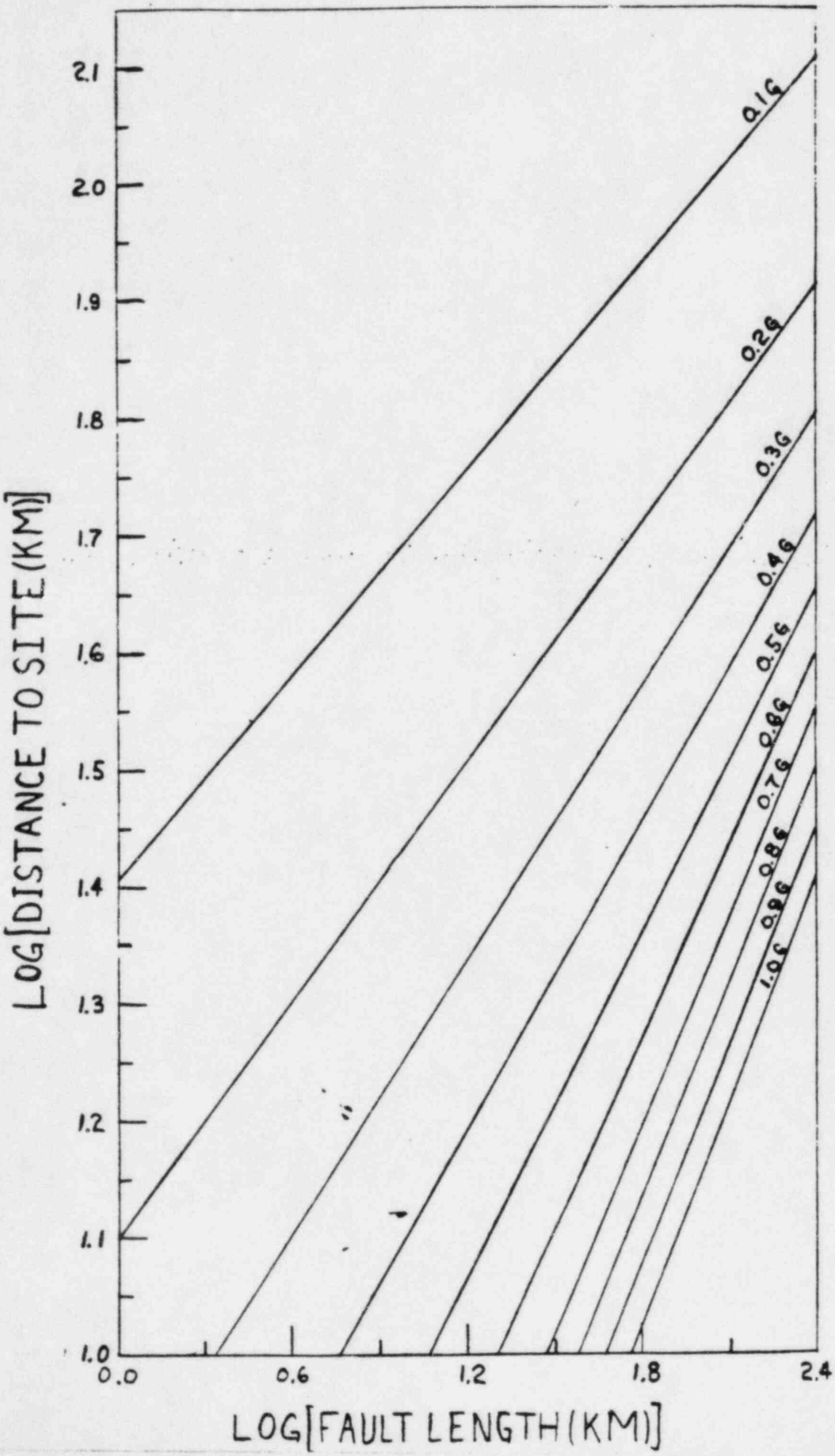
The maximum peak-horizontal bedrock-acceleration value derived through this exercise should be compared with the projected maximum-practical design-acceleration for the tailings embankment. If the latter is greater than the

former, the viability of stabilizing the pile at the given site should be appraised before proceeding to steps (2), (3) and (4).

Step (2), Determination of Area for Fault Investigation and Identification of Faults to be Investigated. By reference to Figure 1 (derived from relationships of Joyner and Boore, 1981, and Bonilla et al., 1984.), which presents a family of curves relating fault lengths to closest fault-to-site distances for different accelerations, a determination should be made of where the curve for the maximum peak horizontal acceleration value obtained in Step 1 intersects the log fault length value of 2.4. For all faults within this area, the fault length and distance to the site should be noted. With th's information, it should be determined whether a point for a fault, plotted on Figure 1, lies above or below the acceleration curve corresponding to the maximum peak horizontal bedrock acceleration value obtained in Step 1. For points that fall above the curve, no further investigation is necessary. For points that lie below the curve, the maximum earthquake magnitude that can be associated with a particular fault should be determined using a relationship between the length of the fault and the size of the earthquake it could generate (assuming that the fault is seismogenic). Acceptable state-of-the-art relationships, such as those of Bonilla et al., 1984, or Slemmons et al., 1982 should be used. Ground motion should be attenuated to the site using the attenuation relationships described in Step (1).

Step (3), Identification of Capable Faults. For faults whose ground motion exceeds, as determined in Step (2), exceeds the maximum peak-horizontal acceleration determined in Step (1), a determination should be made of whether they are to be considered as capable faults. As used in this document, the term "capable fault" has the same meaning as defined in 10 CFR 100, Appendix A III(g). The DOE should evaluate whether or not a fault is a capable fault with respect to the characteristics outlined in paragraphs III(g)(1), (2), and (3) by conducting a reasonable investigation using suitable geologic and geophysical techniques (such as those outlined by Slemmons, 1977). The viability of stabilizing the pile at the given site, and the need to proceed with Steps (3) and (4), should be appraised when at any point in this process a capable fault is identified that would produce ground motion at the site in exceedance of the projected maximum practical design acceleration for the embankment.

Step (4), Designation of the Maximum Credible Earthquake. From among all the earthquakes associated with capable faults, as derived in Steps (2) and (3), and the earthquakes identified in Step (1), the event that yields the maximum



LOG[DISTANCE TO SITE(KM)]
2.1
2.0
1.9
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1.0
0.0
0.6
1.2
1.8
2.4
LOG[FAULT LENGTH(KM)]
0.1G
0.2G
0.3G
0.4G
0.5G
0.6G
0.7G
0.8G
0.9G
1.0G

FIG. 1

peak horizontal bedrock acceleration at the site should be designated as the maximum Credible earthquake (MCE).

1.2.2.3.2 Surface Faulting (Ground Rupture)

As used in this document, "surface faulting" refers to differential ground displacement at or near the surface caused by tectonism. It is distinct from non-tectonic types of ground disruptions such as landslides, fissures and craters. features. Fault investigations related to this issue should be directed at providing reasonable assurance that no capable faults are present within 3,000 feet of the embankment. Such investigations can be logically integrated with those undertaken to assess the hazards to the site from vibratory ground motion (section 1.2.2.3.1).

1.2.2.3.3 Other Tectonic Hazards

Investigations of other tectonic phenomena that could affect site stability should be an integral part of the overall tectonic hazard assessment. Two main items fall under this category:

(1) Volcanic activity; and

(2) Actual or potential surface or subsurface subsidence, uplift or collapse associated with regional or local tectonic deformational zones.

1.2.3 Review Procedures

The scope of UMTRCA document reviews by NRC is partly dependent on the stage of development of the report being reviewed.

1.2.3.1 Early Identification of Issues

Review of initial draft versions of Remedial Action Plans are primarily intended to evaluate the documents' completeness in relation to the review elements previously described, and to identify technical issues that should be addressed in later versions. The review will assess whether all geologic-seismologic representations and interpretations are based on adequate information.

1.2.3.2 Review of Remedial Action Plans

The Remedial Action Plans should contain sufficient information to allow the reviewer to make an independent assesment of the document's conclusions, i.e., the reviewer should be led in a logical manner from the data and premises given

to the conclusions that are reached without having to make an extensive independent literature search. Controversial information should not be ignored so as to enhance a particular position. The geologic terminology used should conform to standard reference works. Questions and comments transmitted to the DOE as a result of the staff review will identify issues that have not been addressed, or adequately documented by DOE, areas where staff interpretations differ from those of DOE, and issues that have not been sufficiently documented to permit the staff to concur with the conclusions reached by DOE.

Later reviews will concentrate increasingly more on evaluating DOE's responses to the initial round of questions and comments presented by the staff. Additional questions and comments for submittal to DOE are developed in regard to data or issues that have become apparent since the initial review or those that develop from the additional information provided in responses. Questions may arise from the reviewers discovery of references not cited in the documents which indicate conclusions or information that supports alternatives to those presented by DOE When insufficient information is provided by DOE to support its interpretations and conclusions, and other reasonable or adequate alternative interpretations are indicated, the staff will request additional investigations or sensitivity studies. All through this process, the staff provides its review input to the Technical Evaluation Memorandum.

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