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SEISMIC DESIGN OF CIRCULAR-SECTION CONCRETE-LINED[1] UNDERGROUND OPENINGS—PRECLOSURE PERFORMANCE CONSIDERATIONS FOR THE YUCCA MOUNTAIN SITE

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Abstract

Yucca Mountain, the potential site of a repository for high-level radioactive waste, is situated in a region of natural and man-made seismicity. Underground openings excavated at this site must be designed for worker safety in the seismic environment anticipated for the preclosure period. This includes accesses developed for site characterization regardless of the ultimate outcome of the repository siting process. Experience with both civil and mining structures has shown that underground openings are much more resistant to seismic effects than surface structures, and that even severe dynamic strains can usually be accommodated with proper design. This paper discusses the design and performance of lined openings in the seismic environment of the potential site. The types and ranges of possible ground motions (seismic loads) are briefly discussed. Relevant historical records of underground opening performance during seismic loading are reviewed. Simple analytical methods of predicting liner performance under combined in situ, thermal, and seismic loading are presented, and results of calculations are discussed in the context of realistic performance requirements for concrete-lined openings for the preclosure period. Design features that will enhance liner stability and mitigate the impact of the potential seismic load are reviewed. The paper is limited to preclosure performance concerns involving worker safety because present decommissioning plans specify maintaining the option for liner removal at seal locations, thus decoupling liner design from repository postclosure performance issues.

Introduction

The Yucca Mountain Site Characterization Project (YMP) is currently investigating Yucca Mountain, Nevada, as a potential site for an underground nuclear waste

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repository in volcanic tuff. In the present conceptual design, several circular concrete-lined openings (shafts and ramps) will provide access to the underground repository. These openings will be excavated through a sequence of welded and nonwelded Tertiary ash-flow tuffs with a variety of characteristics. The Yucca Mountain site is situated in a region of natural and man-made seismicity. Underground openings excavated at this site must be designed for worker safety in the seismic environment anticipated for the preclosure period. This includes accesses developed for site characterization, regardless of the ultimate outcome of the repository siting process.

The scope of this paper is limited to the design of circular-section, concrete-lined underground openings to resist seismic loads. The primary purposes of the concrete liners are to provide structural support of the ground and to secure the openings against rock fall hazards to personnel. Important secondary purposes of the liners include (1) protecting the wall rock from weathering; (2) providing a low-friction surface to increase efficiency of ventilation; and (3) in shafts, providing a regular, finished cross section and stable anchorage for installing and aligning shaft equipment. The liners must perform these functions in a complex load environment that includes in situ ground pressures and thermal loads (high temperatures from the waste and resulting increasing ground pressure) in addition to the seismic loads.

Although this paper emphasizes seismic design, seismic effects must be combined in an integrated design methodology. Hardy and Bauer (1991) discuss design of underground repository drifts for the Yucca Mountain site. Richardson et al. (1988, 1989, 1990) discuss a general outline for shaft liner design for nuclear waste repositories that is also suitable for design of all concrete-lined openings. Figure 1 is a flowchart of the overall design methodology for shafts. In these design methodologies, seismic loads are combined with other loads in analyses of opening performance.

Analytical Methods

General—Nonrepository liner-design practice has traditionally involved calculating only peak hoop stresses caused by ground and water loads acting on the exterior of the liner. For repository applications, a more complete mechanical treatment is required so that ground pressure and thermal and seismic loads can be considered in a consistent manner. The mechanical analysis must include identification of important deformation modes, selection of appropriate mathematical models to investigate each of these modes, and a combination of deformation modes.

Models for liner stress analysis can be divided into three progressively more complex categories:

(1) "conformable strain" models—those that assume the strains in the liner are exactly the same as the free-field strains in the rock;

(2) "structural member" models—those that analyze the liner as a free-standing structure subjected to loads imposed on it by the rock mass; and

(3) "interaction" models—those that analyze the liner and rock together as a system and consider the interaction between them.

The first type of model involves the least computation, but is very conservative if the liner is thick and significantly stiffer than the surrounding rock mass. The second type of model assumes that it is possible to calculate a set of loads exerted on the exterior of the liner by the rock mass. These loads can then be used to calculate the

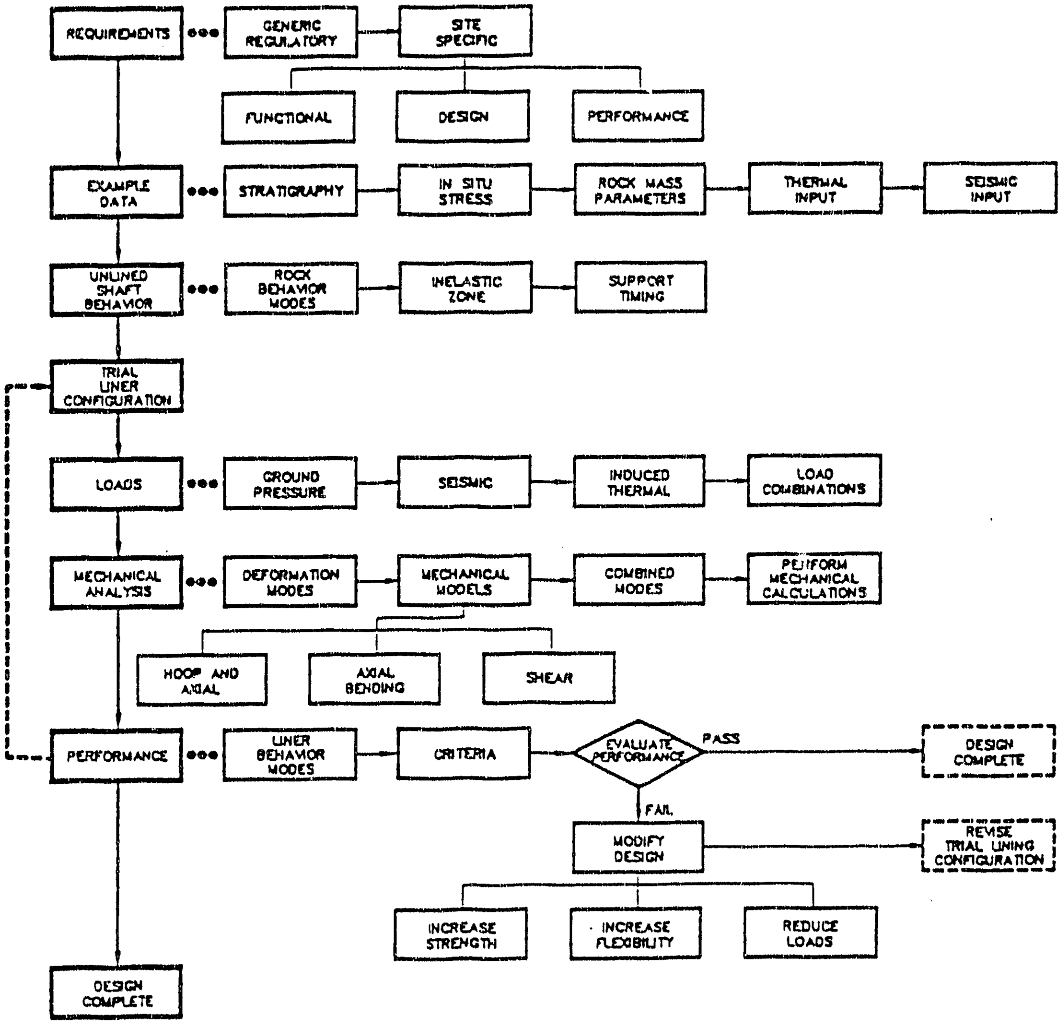

Figure 1. Flowchart of Liner Design Methodology (after Richardson, 1990)

movements and thrusts in the liner by assuming the liner behaves like a curved beam or similar structural element. In the third case, interaction is inherently considered.

Ground/Support Interaction Analysis—If the liner is very flexible (relatively thin and/or having a modulus near that of the medium), the strains in the liner will closely match the strains in the ground, and ground-support interaction can be ignored with little loss of accuracy. If the liner is stiff and resists the ground deformation, ground-support interaction must be considered in some fashion. Closed-form solutions are available for simple cases of ground-support interaction, such as lined circular openings. More complex cases may require numerical solutions, as described in St. John and Zahrah (1987).

Pseudostatic Versus Dynamic Analysis—Unlike surface structures, such as buildings, which tend to move and deform independently when excited by earthquake-induced ground motions, concrete liners generally move and distort compatibly with the medium in which they are embedded. This constraint minimizes or precludes certain types of dynamic amplification that occur in free-standing structures. However, dynamic amplification can occur in underground structures if a nonuniform strain gradient exists across the opening cross section at any given position in time.

Provided that the wavelength of the seismic pulse is relatively large with respect to the opening diameter (i.e., if the rise time of the seismic impulse is long relative to the transit time of the wave front across the opening), dynamic amplification will be small.

Hendron and Fernandez (1983) concluded that there will be little dynamic amplification if the wavelength is at least eight opening diameters. The entire opening cross section will be in a near-uniform strain field in this case. Because the wavelength associated with the ground motion peak from an earthquake is generally at least ten times the diameter of a typical shaft, "pseudostatic" analysis will yield sufficient accuracy for most liner analysis. Pseudostatic analysis involves replacing dynamic loads with approximately equivalent static loads.

If pseudostatic assumptions are not satisfied, such as might be the case very close to large underground explosions, dynamic analysis techniques must be employed. Because of their complexity, most dynamic analyses are performed using numerical codes. Finite difference, finite element, boundary element, distinct element, and other numerical methods have been used to perform dynamic analyses. Because the pseudostatic assumptions appear to be satisfied for lined openings at Yucca Mountain, further discussion of dynamic methods is outside the scope of this paper. However, the general methodology outlined in Figure 1 is the same regardless of the level of analysis.

Proposed Models for YMP—Three separate closed-form models are proposed for pseudostatic analysis of circular lined openings for the YMP. These analytical models can be used to calculate stress states resulting from deformation of the liner in the three modes depicted in Figure 2 (hoop deformation, bonding, and shear). Models appropriate for analyzing nonlinear liner behavior, or openings of arbitrary shape, are outside the scope of this discussion. If all the modes of loading are to be considered, a three-dimensional model permits the most complete analysis. However, if the design remains in the elastic regime, behavior modes can be analyzed independently, and stresses can be combined using superposition. Figure 2 illustrates three independent deformation modes: (1) hoop deformation combined with axial deformation, (2) bending, and (3) shear. These provide an estimate of the stresses and strains in the liner and account for the important interaction between the liner and the rock mass within which it is embedded. The equations are presented elsewhere, and only a brief discussion follows:

- Hoop Deformation and Axial Strain (Figure 2a)—Ground pressure, thermal loads, and seismic loads can all result in hoop and axial deformations, which are likely to be the most important modes of deformation in shaft liners. The relationship between the free-field conditions and the radial, hoop, and axial stresses induced in the liner is provided by a closed-form elastic interaction solution derived by St. John (1991). The model on which this solution is based considers a cross-section of a thick cylinder (liner) embedded in a matrix (rock) and subjected to biaxial loading of the cross sections and out-of-plane axial strain. Transfer of shear stresses at the liner-rock interface (fully bonded condition) is assumed. Alternately, standard numerical models (e.g., finite element, finite difference) can be used to perform the same analyses.

Proposed Model

Deformation Mode

a) Hoop deformation and axial strain

Closed-form interaction model for elastic analysis of embedded cylinder under biaxial loading of cross sections. Considers the effect of axial strain (St. John, 1991).

b) Bending

Standard closed-form models for elastic stresses in a cylindrical beam conforming to the free-field curvature (Timoshenko and Young, 1968).

c) Shear

Elastic interaction solution for shear (St. John, 1991).

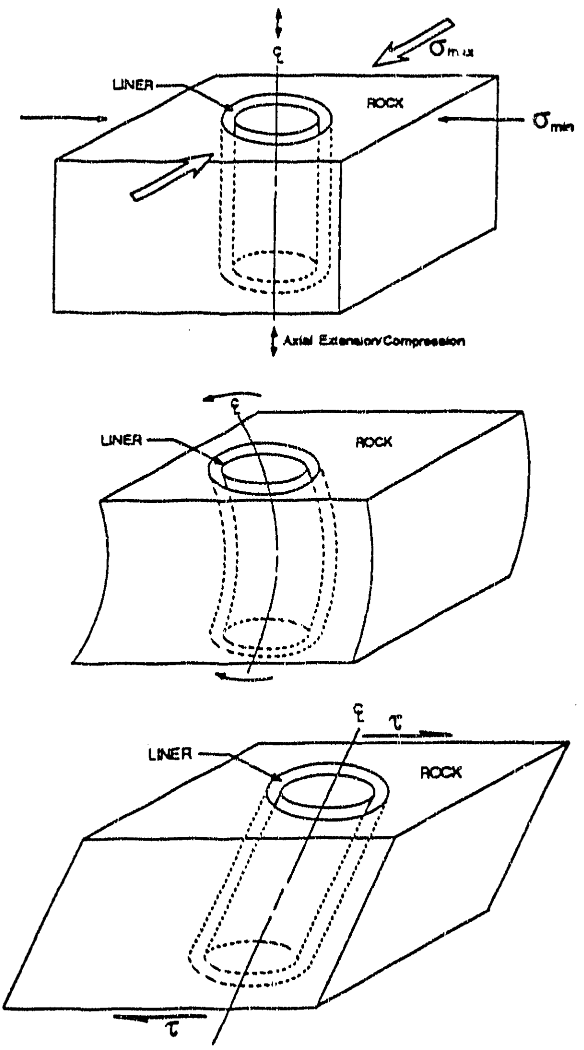

Figure 2. Schematic Diagram of Modes of Deformation Showing Appropriate Models if Modes are Analyzed Separately (after Richardson, 1990)

- Bending (Figure 2b)—Axial bending may result from curvatures associated with nonuniform, thermally induced displacement and with vertical or inclined seismic shear waves (S waves). Because the axial bending stiffness of the liner is low relative to the rock mass that surrounds it, it is reasonable to assume that the liner will deform conformably with the rock mass. Bending causes both axial and shear stresses, which can be evaluated using standard equations that represent the shaft as a vertical, hollow-cylinder beam.

- Shear (Figure 2c) - Direct shear may be caused by inclined S waves or by thermal loading. A closed-form interaction solution for shear was also derived by St. John (1991) and can be used to calculate shear stresses on planes normal to the axis of the shaft. These equations relate the liner shear stresses to the geometrical and material properties, and to the free-field shear stresses on these planes.

Stress transformations and superposition allow hoop, axial, and shear stresses to be combined to calculate the total principal stresses throughout the liner. The values at the inner circumference of the liner are of greatest concern; it is here that failure would initiate in practical situations.

Load Environment

General—As mentioned previously, three sources of loading need to be considered to evaluate the liner stresses for the repository shafts and ramps: in situ ground pressure, seismic, and induced thermal. Two YMP documents (Richardson, 1990; Hardy and Bauer, 1991) discuss calculation of all three types of loads in some detail; in the following discussion, seismic loads will be emphasized. For purposes of this discussion, the free-field stresses, strains, and displacements at the shaft location are collectively termed "loads." "Free-field" refers to effects in the ground that would occur if the opening were not present. By defining loads in this manner, they become independent of opening and liner details. If the rock mass behaves elastically, they can also be directly combined in a single analysis, and the effect of combined loads evaluated locally at the opening.

Seismic Loads—A seismic event, whether associated with an earthquake or an underground explosion, generates elastic waves that propagate outward from the source. Body waves may be classified as P (pressure or dilatational) waves and S (shear or distortional) waves. P waves have an inherently higher velocity of propagation than S waves and will always arrive first at the shaft location.

Elastic waves generate ground motions that are defined at a control point, commonly the bedrock surface, and are often described by displacement, velocity, and acceleration components. Because seismographs are typically set to measure horizontal and vertical components of motion and as a design expediency, control motions are defined in terms of horizontal and vertical components. Distortional ground motions are typically resolved into S_h waves with horizontal particle motions and S_v waves with motions in a vertical plane. Both shear motions are orthogonal to the incident wave direction as illustrated in Figure 3. By definition, particle motions resulting from S_v waves are in a vertical plane, but have both horizontal and vertical components. Seismic ground motions tend to attenuate with depth away from the surface. If downhole measurements are not available, this attenuation can be determined approximately by dynamic computer simulation of the soil and rock strata, using one of several available numerical methods (e.g., Schnabel et al., 1972).

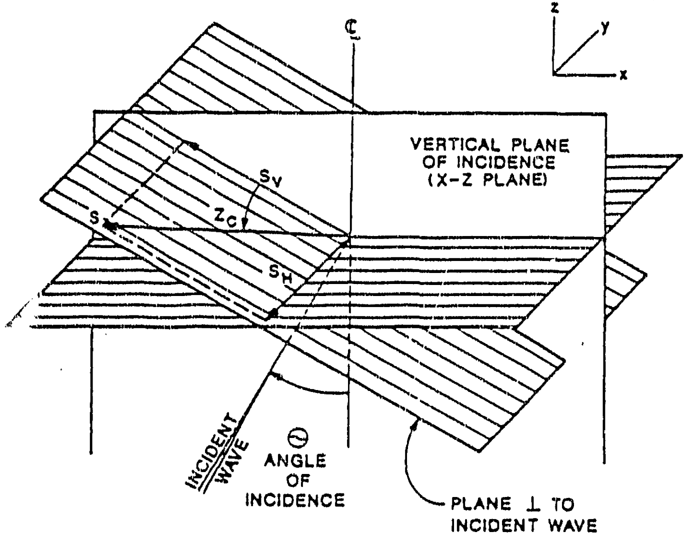

Figure 3. Resolution of Shear Wave Motion into S_v and S_h Components (after Richardson, 1990)

The elastic waves from a seismic event induce transient stresses and strains in a rock mass and, hence, in any embedded structure such as a shaft. The effects on the structure will depend on a number of parameters, including the physical properties of the rock and of the structure; and the direction, amplitude, and possibly frequency and duration of the ground motion. Seismic events also may impose direct shear displacements on the shaft liner if they cause movement along faults that transect the shaft axis.

If static methods are considered to be appropriate, a tensor of free-field strains can be directly calculated using equations for strain components in terms of peak particle velocities, propagation velocities, and angles of incidence. Although typically much effort is placed on estimating the ground motion levels for an important project, less attention may be given to calculating seismic propagation velocities, which contribute equally to strain calculations and which can be estimated from the properties of the rock. A summary of the equations for calculating free-field strains is shown in Table 1.

Table 1. Equations For Calculating Free-Field Strains (after SNL, 1990)

Wave	Free-Field Strains						Bending Strains
Type	ε_{xx}	ε_{yy}	ε_{zz}	τ_{xy}	τ_{yz}	τ_{xz}	ε_b
P	$\frac{V_p}{C_p}\sin^2\theta$	0	$\frac{V_p}{C_p}\cos^2\theta$	0	0	$\frac{V_p}{C_p}\sin 2\theta$	$\frac{Ra_p}{C_p^2}\sin\theta\cos^2\theta$ (in $x-z$ *plane*)
S_v	$\frac{V_{sv}}{C_s}\sin\theta\cos\theta$	0	$\frac{V_{sv}}{C_s}\sin\theta\cos\theta$	0	0	$\frac{V_{sv}}{C_s}\cos 2\theta$	$\frac{Ra_{sv}}{C_s^2}\cos^3\theta$ (in $x-z$ *plane*)
S_h	0	0	0	$\frac{V_{sh}}{C_s}\sin\theta$	$\frac{V_{sh}}{C_s}\cos\theta$	0	$\frac{Ra_{sh}}{C_s^2}\cos^2\theta$ (in $y-z$ *plane*)

where θ = angle of incidence for P, S_v, and S_h waves, defined from z (vertical) axis,
C_p = propagation velocity of the P wave,
C_s = propagation velocity of the S_v and S_h waves,
V_p = peak particle velocity of the P wave,
V_{sv} = peak particle velocity of the S_v wave,
V_{sh} = peak particle velocity of the S_h wave,
a_p = peak acceleration of the P wave,
a_{sv} = peak acceleration of the S_v wave,
a_{sh} = peak acceleration of the S_h wave, and
R = radius from the centerline of a circular opening.

The combined effects of different waveforms in the liner must be assessed. Simplified design approaches commonly assume that all the seismic energy may be assigned to a single, critically oriented waveform, or alternately, that the effects of P and S waves may be assumed to occur separately if the epicentral distance from the opening is large. Although S_h and S_v waves may occur simultaneously because they have the same propagation velocity, the effects of these waves are likely to be out of phase. For example, the effects of acceleration (curvatures) will not peak at the same time as the effects of particle velocities (strains). Such assumptions may not always be conservative. Complete P- and S-wave separation cannot be assumed at Yucca Mountain because some potential seismic sources are close to the site, and it is possible that several waveforms may act on the shaft simultaneously.

Newmark and Hall (1977) have suggested that the three orthogonal components of earthquake input motion can be considered to be randomly phased; i.e., the three control motions have statistical independence. Thus, it is an oversimplification to treat them separately. However, because there is only a small probability of the maximum responses occurring simultaneously, they should be combined probabilistically in some fashion rather than by direct vectorial combination, which would be overly conservative. Rather than a full probabilistic treatment, Newmark and Hall (1977) recommend a conservative simplified approach commonly called the 100-40-40 combination rule. Extending this rule to structural effects from earthquakes, 100% of the largest peak effect (e.g., strain in a given direction) from any of the three seismic wave components plus 40% of the peak effects from each of the other two components are combined. In some cases, a vector sum is involved to combine effects acting in different directions. In all cases, the objective is to determine the combination that maximizes a particular structural effect (e.g., strain, hoop stress, maximum principal stress, etc.). The procedure involves trying all possible combinations to determine the one yielding the maximum effect.

Complete seismic loads for input to the liner analysis include strains and curvatures. These strains and curvatures result in stresses; Figures 4 and 5 associate these stresses with the deformation modes discussed earlier.

If static methods are not appropriate, as in the case of large openings very close to the source, then dynamic analysis will be required. This method requires input from complete time histories of ground motions. Further discussion is not in the scope of this paper.

Site-Specific Seismic Loads—At the potential site, both natural seismicity from earthquake and artificial seismicity from weapons testing must be considered. Geologically, Yucca Mountain is located in the basin-and-range region of North America, which is characterized by normal and strike-slip faulting with accompanying seismicity. In 1987 and 1988, a working group was convened to study the significance of the seismic environment to the performance of the Exploratory Studies Facility (ESF) (SNL, 1990). Studies of the average slip rates on faults near the site suggest the average recurrence interval for potentially damaging earthquakes to be quite long relative to the preclosure performance period. However, slip rates on the relatively active Bare Mountain fault could result in a magnitude 6.5 earthquake on a minimum occurrence interval of 6000 years. This event was used as a deterministic basis for establishing design ground motions for earthquakes at the site, and was substantiated by suitably conservative probabilistic hazard analyses. The working group recommended a control horizontal acceleration and velocity of 0.3 *g* and 30 cm/sec, respectively, to be used for ESF design. Vertical control motions of 0.3 *g* and 20 cm/sec were recommended.

The YMP repository site is also adjacent to nuclear weapons testing areas at the Nevada Test Site (NTS), and the working group considered potential ground motions from this source. Considering the largest weapons yield possible without risk of off-site damage at the closest site practical for fielding such a test (Buckboard Mesa), a design basis underground nuclear explosion (UNE) of 700 kt at 22.8 km was selected. This resulted in 0.2 *g*, 9 cm/sec for the vertical component and 0.1 *g*, 12 cm/sec for the horizontal component. Because these values resulted in smaller hoop stresses around vertical shaft openings than the earthquake ground motions, the latter were considered to control the design.

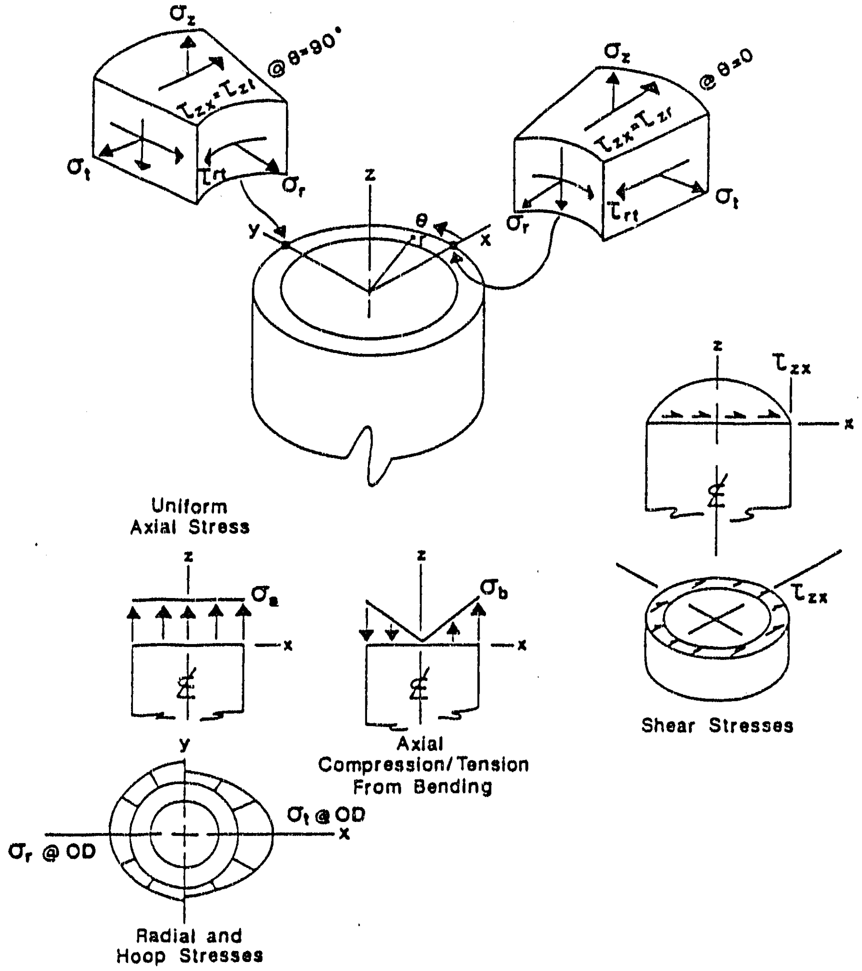

Figure 4. Typical Stresses Resulting from S_v Wave Incident in the X-Z Plane (after DOE, 1987)

Because of uncertainties in the depth attenuation properties of the site, the working group further suggested that no attenuation of ground motions with depth be considered until further study.

Other Loads—Although this report concerns seismic design, other types of loads exist and must be considered and combined with the dynamic loads as appropriate to obtain a complete design.

- In situ stresses exist at depth due to gravitational and tectonic forces and residual stresses. The in situ stresses are constant over the preclosure period of repository performance, but are locally redistributed due to excavation. The in situ stresses vary with depth and may vary spatially due to inhomogeneities in the host rock and local geologic structures. In a supported opening, loads are induced in the support system as a result of deformation of the newly unconfined rock as the excavation is advanced. The magnitude of this load will be a function of the ground and support stiffness and the distance from the face to the point of support installation, and may be expressed as a percentage of the free-field in situ stresses. Several studies of lined tunnels and shafts in elastic media (e.g., Ranken and Ghaboussi, 1975 and Pariseau, 1977) have indicated that little load will develop in the support system if the support is installed further than one diameter from

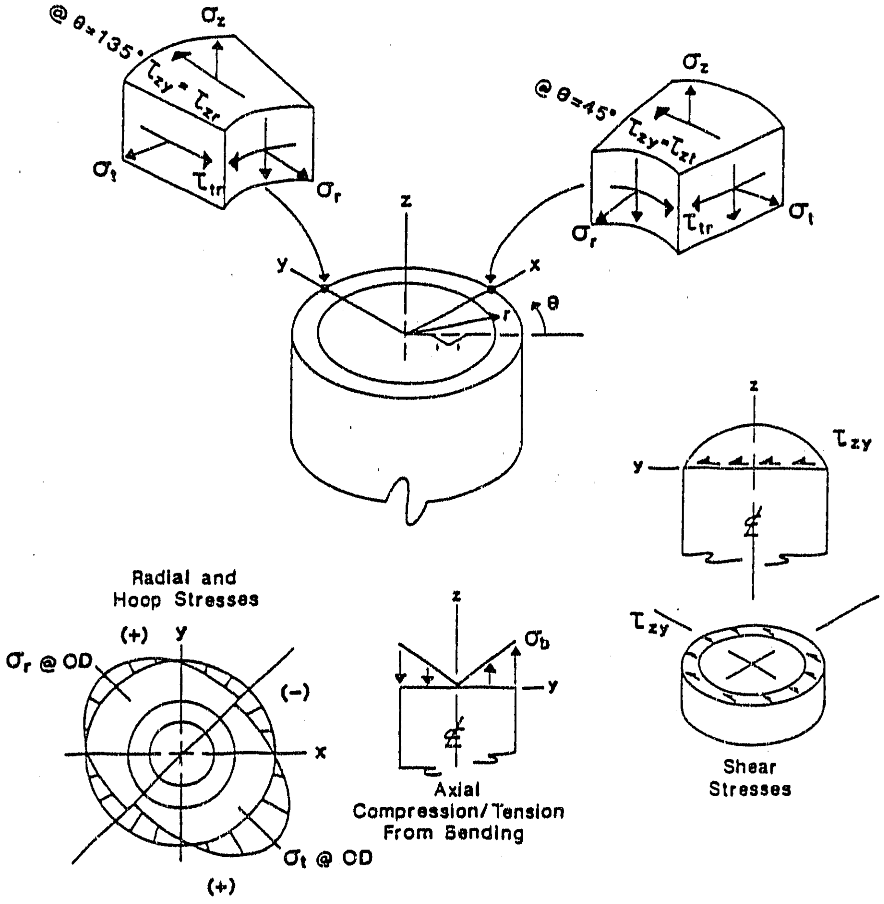

Figure 5. Typical Stresses Resulting from S_h Wave Incident in the X-Z Plane (after DOE, 1987)

the advancing face. The percentage of the deformation resulting from relaxation of the in situ stress at the excavation and interaction with the liner may be calculated by numerical simulation of the excavation process. In ground assumed to be linear-elastic, the value for the ground pressure is generally less than 25% of the free-field in situ stress if the linings are installed one diameter or more behind the face. In ground that behaves in a nonlinear manner, numerical methods may be required to estimate ground pressure for support interactions.

- Thermal loads, resulting from expansion of the rock near emplacement areas as it is heated by the waste, must be considered in designing those openings that must remain functional following the period of waste emplacement. The load acts both horizontally as a free-field stress increase, and to a lesser extent, vertically where uplifting of the ground surface limits the development of vertical stresses. Due to standoff distance from the emplacement areas, repository shafts and ramps will be largely unaffected by temperature changes during their functional period, but regional stress changes can still be significant for lining design. Emplacement drifts and other repository openings may encounter significant thermal loading.

Liner Performance

Historical Perspective—Reports of the performance of lined underground openings during earthquakes are relatively few. More information is available on performance of dynamically loaded underground openings with varying degrees of ground support, some exposed to shock waves from underground explosions. The classic paper by Dowding and Rozen (1978) documented a number of cases; Owen and Scholl (1981) added to this data base. Schmidt and Richardson (1989) reported additional cases specific to shafts.

Most recently, Sharma and Judd (1991) revisited this topic and compiled a data base of 192 reports from 85 earthquakes throughout the world. They were able to document 24 openings lined with plain concrete and 9 reinforced concrete liners, for a total of 33 concrete-lined openings. Of these, 7 were undamaged, 12 were slightly damaged, 3 were moderately damaged, and 11 were heavily damaged during dynamic events. However, the majority (67%) of the 192 case histories regarded unlined openings. The proportion of the unlined openings in the study that were damaged (38%) was lower than the proportion of concrete-lined openings damaged (79%). Sharma and Judd attributed this to (1) relatively poor ground conditions that originally required the openings to be lined, and (2) soil/structure interaction effects caused by presumed stiff liners. Not mentioned were that (3) damage in the form of cracking or spalling is easier to identify in lined openings, and (4) lined openings are typically more expensive and important than unlined openings, and perhaps more likely to be classified as damaged. Note that these case histories represent only a small fraction of the total underground openings exposed to dynamic loads; the likely inference is that little damage occurred in most cases.

General conclusions from this work are that (1) underground openings are usually more resistant to earthquake shaking than are surface structures; (2) the effects of earthquakes on underground openings diminish with depth; and (3) damage consists of cracking, fallout of blocks, and spalling but only occasional collapse. However, (4) underground openings are vulnerable to damage from direct fault offset, and (5) tunnel portals may be damaged from shaking.

Liner as a Structural Member—In the United States, most design of concrete structures is performed using the methods and criteria developed by the American Concrete Institute (ACI) and prescribed in its concrete codes (ACI, 1983a; 1983b). The design and analysis provisions of the concrete codes were primarily developed for surface structures, and there are important differences between concrete liners for underground openings and apparently similar surface structures such as chimneys that may lead to overconservatism if standard structural design methods are applied to shaft liners. In particular, a cast-in-place concrete liner is not a free-standing structure that must support an externally applied load; rather, it reinforces the surrounding rock mass, which is itself largely self-supporting. Furthermore, except for water pressure, loads acting on the liner will generally lessen with liner deflection caused by minor cracking, creep, and shrinkage in the concrete, yet no consideration is typically made for these effects in liner designs. A liner is inherently stable under external loading conditions because of its cylindrical geometry and its bonding to the rock, and catastrophic failure in a cracked liner cannot develop until considerable post-yield distortion has occurred. Performance criteria used for repository shaft design must realistically reflect this inherent stability.

Overloading a beam or column may ultimately result in sudden, catastrophic collapse and complete loss of its load-bearing ability. This behavior was considered when the building codes were developed, and an appropriate amount of conservatism was built into codes for designing these numbers. Unlike a beam or column, the liner and rock reach a stable equilibrium through load redistribution. Hence, less conservatism should be necessary when designing liners for circular openings than for beams and columns. In the following sections, ACI criteria will be reviewed for applicability to repository shaft liner design.

Liner Behavior Under Overstress Conditions—Before discussing criteria, the levels of progressive failure of a concrete liner should be discussed. There are several modes of inelastic behavior (cracking, spalling, crushing) that could develop if the liner is overstressed. Not all of these are equally likely, nor will all of them affect the maintainable performance of the liner.

Three overall levels of liner damage have been defined for the purpose of this discussion. Level I involves minor tensile cracking. Level II involves compressive crushing and spalling, possibly combined with more severe tensile cracking. Partial collapse and failure of the liner occur in Level III. The damage levels (I, II, III) described above were selected to be roughly equivalent to the damage levels used for assessing tunnel response to UNEs at the NTS (Kipp et al., 1986). The NTS work also considers a fourth category (Level IV), representing complete collapse. For repository design, the distinction between Levels III and IV is unnecessary. The potential impact of each of these levels of damage on the liner functional requirements will largely determine appropriate acceptance criteria for analysis.

- Level I Damage (minor tensile cracking)—Tension cracks perpendicular to the shaft axis may arise from induced axial tension. These tension cracks are likely to be quite small, evenly distributed along the length of the shaft, and similar to cracks resulting from shrinkage of the concrete. Hairline cracking is common in concrete and can be observed in perfectly stable structures. More continuous axial tension (circumferential) cracks, should they occur, will be similar to axial construction joints that naturally occur between successive liner pours during construction. Neither distributed hairline cracks nor the more continuous axial tensile cracking are expected to affect any of the liner functions.

  Radial tensile cracks may occur from nonuniform hoop distortions. If the cracks fully penetrate the liner, it could be postulated that further loading of a radially cracked liner could push blocks formed by radial cracking into the opening and cause the liner to fail (Level III damage). However, this type of failure is highly unlikely because (1) normal stresses are generated across the failure surfaces as an individual block is pushed in—this is caused by wedging of the inward tapering blocks; (2) the loads typically are not "following" loads; and (3) there may be some interlocking or bonding between the liner and the rough rock surface.

  Taken to its unlikely extreme condition, a combination of radial and circumferential tension cracks could cause a condition similar to a liner made of bricks or blocks. In fact, ungrouted brick or block liners have a long history of successful application in mining and tunneling. Although tension cracks will certainly occur before Level III damage, major distortion and crushing of the liner must accompany the tensile cracking before the liner can collapse. Thus, tensile cracking may be a symptom of liner problems but does not in itself constitute a failure mechanism.

- Level II Damage (compressive crushing and spalling, major tensile cracking)—The primary concern in concrete shaft liner design is crushing or spalling of the concrete which results from excess compressive stress at the interior face of the liner. Severe combinations of all three types of loading (in situ, thermal, and seismic) could cause this type of inelastic behavior in the liner. Along with spalling, severe tensile cracking can also be considered Level II damage. The onset of spalling does not represent loss of the ground support or ventilation functions of the liner. However, concrete spalls could become detached from the liner, possibly resulting in hazards associated with falling objects, especially a concern in shafts. In hoisting shafts, Level II damage raises questions about the integrity of the conveyance anchorages. For these reasons, it is appropriate to limit the allowable compressive stresses or strains in the concrete to prevent Level II damage.

- Level III Damage (collapse)—Collapse of a properly constructed, suitably thick-walled concrete liner would not occur unless the liner experienced severe distortions, as discussed above. Collapse of the liner would result in a loss of all liner functions. However, a structure designed against Level II damage will have a large factor of safety against collapse.

The concrete codes allow two design methods: (1) the "ultimate strength" or "strength design," which is the primary method, and (2) the working stress method, which is an alternate. In the working stress (alternate) method, linear analysis is used to calculate working stresses in the concrete, which are compared to allowable stresses. For this reason, the working stress method is sometimes called the "elastic" method. Linear elastic analysis is used in the design example with a working stress approach. The concrete code does not provide recommended ultimate strength equations for embedded cylinders.

Yucca Mountain Perspective—It is important to note that several features of the potential Yucca Mountain repository distinguish it from past conceptual designs for proposed repositories at other sites: (1) none of the shafts (and hence no equipment attached to the shaft liners) will be used to transport waste, (2) the openings are located in the unsaturated zone and the liners need not be watertight, and (3) there are currently no credible accident scenarios involving shaft or ramp liners (Hartman and Miller, 1991). Furthermore, present decommissioning plans specify liner removal at seal locations (Fernandez et al., 1989), decoupling concrete liner design from seal performance issues. Thus, the primary functional requirement of the concrete liners is solely to maintain worker safety and operational efficiency; they are not important to public safety or postclosure containment and isolation.

Proposed Criteria—The discussion above establishes that Level I damage involves minor tensile cracking, which is probably unavoidable in liners subjected to significant seismic loading and which should not interfere with any opening functions. It is appropriate to develop criteria to guard against the onset of Level II, as well as to prohibit any progression to Level III.

ACI 318-89, Appendix A (Alternate Design Method) (ACI, 1983a) specifies extreme fiber stresses in flexural compression of 0.45 f_c' for reinforced concrete (A.3.1) where f_c' is the 28-day cylinder strength of the concrete. Load factors are taken as unity in this method (A.2.1). Members may be proportioned for 75% of capacities required by other parts of Appendix A when considering wind or earthquake forces combined with other loads, provided the resulting section is not less than that required for the combination of static loads (A.2.2). This is equivalent to an allowable stress of 0.60 f_c' for dynamic load combinations.

The Building Code Commentary (ACI-318R, 9.3.1) states one of the purposes for the strength reduction factor is to reflect the degree of ductility and required reliability of the member under the load effects being considered. Given the inherent stability and ductility of the circular concrete liner, it is not unreasonable to suggest a somewhat higher allowable stress than that proposed for other structural elements, with much less inherent stability.

Concrete liners are typically made of unreinforced plain concrete, for which the Plain Concrete code ACI 318.1-83 (ACI, 1983b) recommends a strength reduction factor of 0.65 on factored loads. For most load combinations (average load factor of 1.4), this is equivalent to working stress reduction factor of 0.45.

For concrete liner design for the YMP, it is recommended that an allowable stress in compression of 0.45 f_c' be used for static loads per the ACI codes. A more liberal allowable stress of 0.65 f_c' is proposed for combinations involving seismic loads, providing the static loads in these combinations do not exceed 0.45 f_c'. The proposed allowable stress for seismic combinations is consistent with the philosophy of the concrete codes. If analysis shows that principal compressive stresses in the inner circumference of a standard concrete liner (8 to 18 in. of concrete) can meet these limits, then a conservative design has been achieved and no special features are required.

Shear stresses do not require separate criteria since they are incorporated in calculating principal stresses in the liner.

Design Alternatives—If the allowable compressive stress criteria are exceeded in a linear analysis or if the principal tensile strains in the rock wall are large, the designer may consider one of the following alternatives to preclude the possibility of Level I damage progressing to Level III damage or otherwise impairing the liner's function.

- Increase concrete strength. Although some advantage may be gained using high-strength concrete, this option involves tradeoffs because the stiffness (and thus the liner stresses) increases with strength.
- Apply welded wire mesh to the concrete surface.
- Add fiber reinforcement to the concrete.
- Increase liner thickness. This also involves tradeoffs between liner stiffness and strength.
- Add embedded steel, either structural ring members or reinforcing.
- Use frangible backpacking to absorb rock displacement without loading the liner.
- Investigate alternative means for increasing liner flexibility.
- Install an inner steel liner to confine the concrete.
- Increase standoff from waste emplacement areas, if possible, to reduce thermal stresses.
- Investigate less conservative design methodologies which permit nonlinear deformation of the concrete past its electric limit.

Site-Specific Example

Description—An example problem specific to the Yucca Mountain site was analyzed. The example illustrates the process of pseudostatic ground-support interaction analysis of a 12-ft-diameter circular concrete-lined shaft exposed to ground pressure and thermal and seismic loads representative of those expected at a specific location in the repository. The location selected was the former location of the exploratory shafts. Figure 6 shows the example shaft and the site stratigraphy.

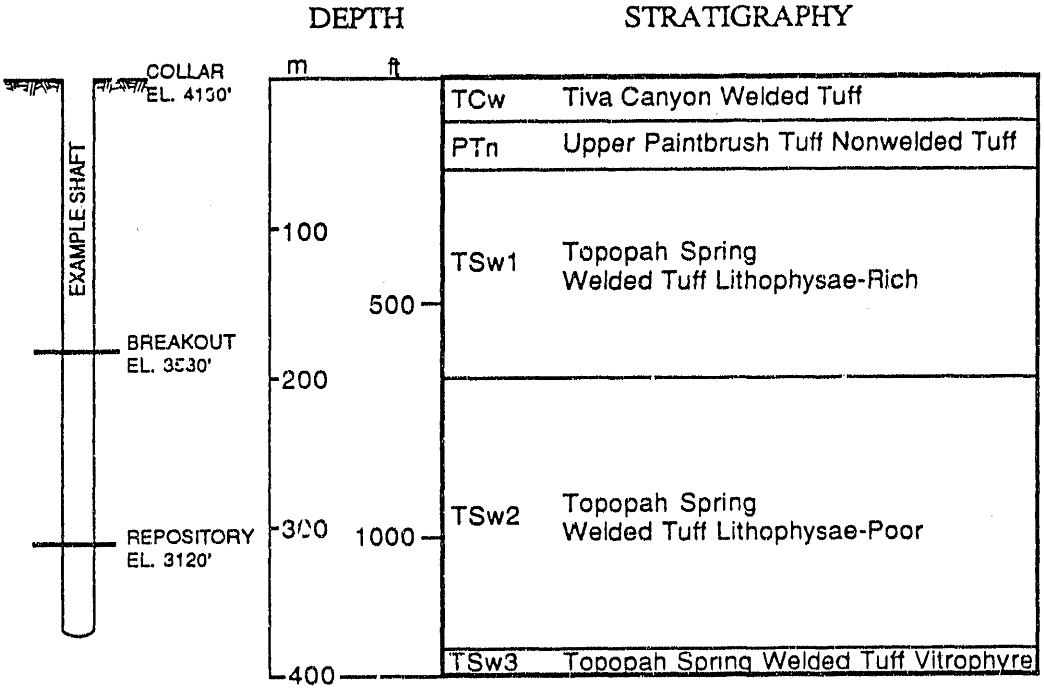

Figure 6. Section of Example Shaft Showing Stratigraphy

Loads—Tables 2, 3 and 4 show representative loads for the example shafts. Ground pressure loads are tabulated in Table 2 in units of pressure. They were calculated from in situ stress values, assuming that 85% of the in situ stress will be dissipated by elastic convergence before liner installation. The 85% figure was confirmed by finite element simulation of the excavation process. The direction of the maximum ground pressure coincides with the direction of the major principal in situ stress axis, which for the purpose of this example is N30°E.

Thermal loads are tabulated in Table 3 in dimensionless units of strain. Thermal loads at the shaft location are those developed for the Site Characterization Plan, Conceptual Design Report (SCP-CDR) (SNL, 1987) designs and were determined based on numerical analysis of the reference repository configuration with a thermal loading of 57 kW/acre. Induced thermal loads are unique because they build very slowly as the heat is transferred from the waste to the surrounding rock. Because they reach their maximum value late in the functional life of the shaft liners, thermal loads at 100 years were used.

Table 4 shows the representative base case seismic loads for the Yucca Mountain site, expressed as strains. The loads are based on design basis ground motions and

Table 2. Ground Pressure Loads

Unit	Elev.	Bottom Depth		Min. Ground Pressure	Max. Ground Pressure	Avg. Ground Pressure
	(ft)	(ft)	(m)	(MPa)	(MPa)	(MPa)
Collar	4130	0	0	0	0	0
TCw	3990	140	42.7	0.22	0.44	0.33
PTn	3868	262	79.9	0.16	0.32	0.24
TSw1	3451	679	207.0	0.15	0.30	0.22
TSw2	2783	1347	410.7	0.45	0.89	0.67
TSw3	2749	1381	421.0	0.45	0.90	0.68

Table 3. Thermal Loads For Base Case*

Unit	Strains ($\times 10^{-6}$)					
	ε_{xx}	ε_{yy}	ε_{zz}	γ_{xy}	γ_{yz}	γ_{xz}
TCw	-5.0	12.3	-6.3	27.3	1.2	1.3
PTn	-5.3	12.3	-14.5	25.2	3.0	2.7
TSw1	12.8	47.3	-81.8	49.7	15.2	8.6
TSw2	20.6	49.2	-95.2	42.1	28.1	0.3
TSw3	19.0	45.0	-89.5	39.6	25.8	3.2
* *100 years; compressive normal strains are positive; x-direction is north*						

Table 4. Seismic Loads For Base Case

Unit	Strains ($\times 10^{-6}$)							
	ε_{xx}	ε_{yy}	ε_{zz}	γ_{xy}	γ_{yz}	$(0.4) \times \gamma_{yz}$	γ_{xz}	Bending
TCw	148	0	175	54	93	37	202	4.39
PTn	158	0	185	58	100	40	214	5.04
TSw1	89	0	104	33	57	23	120	1.56
TSw2	80	0	94	29	51	20	108	1.30
TSw3	54	0	63	20	34	14	72	0.58

typical site data using the procedures presented in the Seismic Working Group Report (SNL, 1990). The seismic loads were based on inclined P, S_v and S_h waves. It is assumed that these waves follow the same path, being incident in the x-z plane at 30° to the vertical. This angle of incidence was determined to provide maximum strains over the likely range of 0° to 30°. For the purpose of illustration, the seismic strains at the repository horizon represent transient stresses of approximately ±4 MPa.

Allowable Stress—The allowable compressive stress for a 5000 psi liner is calculated as follows:

$$0.45 \times f_c' = (0.45)(34.5\ \text{MPa}) = 15.5\ \text{MPa (combined static), and}$$

$$0.65 \times f_c' = (0.65)(34.5\ \text{MPa}) = 22.5\ \text{MPa (static and seismic).}$$

The allowable tensile stress is

$$3.5 \times \sqrt{f_c'} = (3.5)(70.7)/145 = 1.71\ \text{MPa.}$$

Analysis—Figure 7 shows the results of the base case analysis in the different layers. The peak stress case occurs in TSw2 because thermal stresses are higher, although results in all layers are comparable.

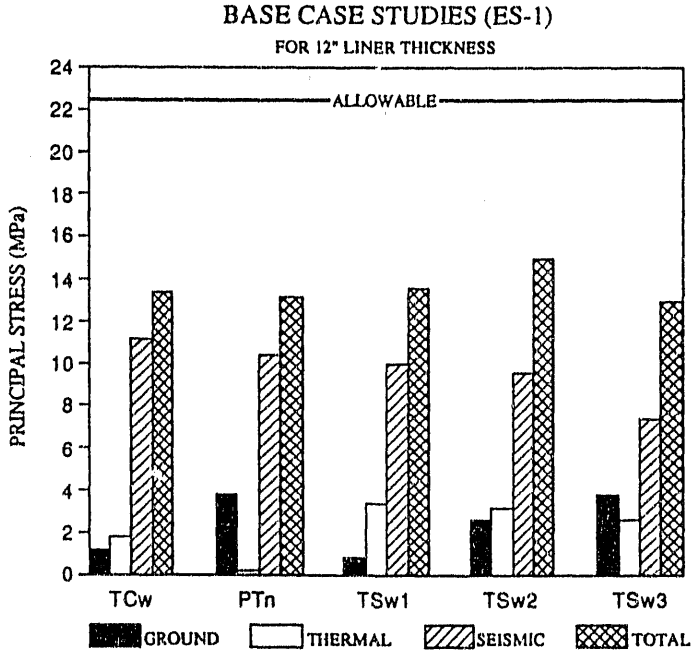

Figure 7. Histogram of Maximum Compressive Principal Stress in ES-1 Liner for the Base Case, Compared to Allowable Stress for 5000 psi Concrete

In addition to base case calculations, a parametric study was performed to investigate the effect of varying levels of seismic peak particle velocities from 10 cm/sec up to 100 cm/sec. Note that this range is not necessarily representative of likely ground motion levels at the site, but was selected to investigate the sensitivity of the shaft liner stresses to seismic loading. Figure 8 shows the effect on the liner of varying the ground motion levels. The allowable concrete stress of 3250 psi (22.4 MPa) is exceeded at a peak particle velocity of 50 cm/sec.

Although not presented here, load combinations involving the negative excursion of the seismic wave without static loads can easily exceed the allowable tensile stress of 1.71 MPa, and tensile cracking is likely in a seismic event. Level I damage will occur, but more severe damage is unlikely. However, light reinforcement, such as wire mesh, is recommended.

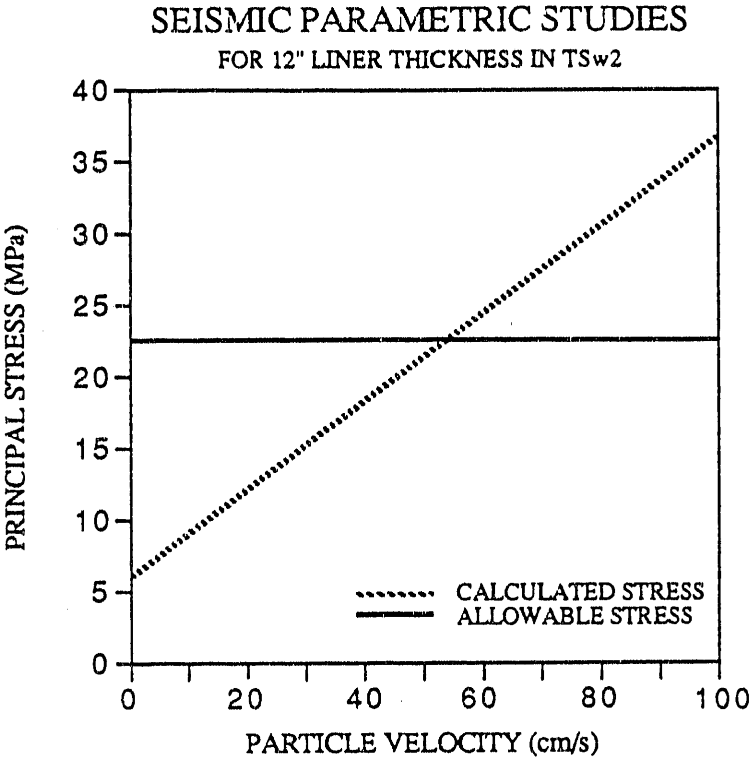

Figure 8. Histogram of Maximum Compressive Principal Stress in ES-1 Liner for the Seismic Parametric Study. Base Case Static Loads Also Included in Calculations.

Conclusions and Recommendations

The example problem presented above suggests that a one-foot-thick plain (unreinforced) concrete liner (5000 psi) installed in a typical repository shaft located in the ESF has sufficient compressive strength to sustain all load combinations involving the design basis seismic event for the preclosure life of the repository. Some minor tensile cracking may occur with seismic loading (Level I damage) which is not considered structurally significant but that requires consideration in the design. Low-cost measures such as embedded wire mesh or fiber reinforcing may be prudent.

For concrete-lined openings near the waste emplacement areas, thermal loads will predominate. However, design basis seismic loads are shown to contribute significantly to the overall load environment for the example problem. When peak particle velocities reach 50 cm/sec, compressive allowables are exceeded and special design measures should be considered, especially for shafts where a falling object hazard exists. The recommended option is light wire mesh in the face of the concrete, or bolted externally, to secure spalls.

The allowable stress for concrete of 0.65 f_c' for combinations involving seismic loads is proposed by the authors, but may require further review prior to final repository design. It is recommended that the ASCE committee sponsoring this symposium review these criteria in formulating their recommendations.

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