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SEISMIC ANALYSIS
OF THE
MIRROR FUSION TEST FACILITY SHIELDING VAULT

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SEISMIC ANALYSIS OF THE MIRROR FUSION TEST FACILITY SHIELDING VAULT

This report presents a seismic analysis of the vault in Building 431 at Lawrence Livermore National Laboratory which houses the Mirror Fusion Test Facility. The shielding vault structure is approximately 120 ft long by 80 ft wide and is constructed of concrete blocks approximately 7 x 7 x 7 ft. The north and south walls are approximately 53 ft high and the east wall is approximately 29 ft high. These walls are supported on a monolithic concrete foundation that surrounds a 21-ft deep open pit. Since the 53-ft walls appeared to present the greatest seismic problem they were the first investigated.

Section I

PRELIMINARY ANALYSIS

The initial effort was very traditional and conservative using the Uniform Building Code coefficients and treating the wall as an element in which the base shear applied was $0.2W$. Assuming the vault is a structure or a building in a seismic zone the base shear is

$$V = Z I K CS W, \text{ and}$$

$$\text{letting } Z = 1 \text{ (Zone 4)}$$

$$I = 1$$

$$K = 1.33 \text{ (rigid box structure)}$$

$$CS = 0.14, \text{ which are all conservative, the base shear}$$

$$V = 0.19W$$

These loads ($0.2W$ or $0.19W$) are conservative, but useful for a scoping analysis. The first observation based on this analysis was that little benefit would be derived from improving the quality of the tie-down rods that were used to secure the blocks. These rods, which are A36 steel,

have a maximum diameter of 2- $\frac{1}{4}$ in. at the base, stepping down to 1- $\frac{3}{4}$ in. and to a minimum size of 1- $\frac{1}{2}$ in. at the top. Each block has four of these rods, two on each face to establish moment resistance to overturning. Analysis of the footing, however, indicates that the footing has equal or less resistance than that provided by the rods, and upgrading of these rods would merely transfer potential failure to the footing.

From this preliminary analysis it was apparent that more complete soils data would be required to complete the foundation and footing analysis. More important, it was evident that a more sophisticated model would be needed to rationalize the behavior and/or survival of this vault wall structure during a damaging seismic event. Therefore the project was subdivided into a three-pronged attack, as follows:

- A. A dynamic analysis (Section II) of rocking blocks was pursued to evaluate the block wall response to a seismic event given that the rods holding them were loosened and the blocks allowed to rock. The virtue of permitting the rocking is that, with blocks of mass as great as these, the effort of tipping a block upwards requires a large amount of energy and when they fall back, the impact is a very inefficient energy storage mechanism. A lot of energy is dissipated during the collision process of the concrete blocks falling upon the foundation and each other. In both the static and dynamic analyses it has been assumed that the outer 2 inches of the block are spalled off during the rocking process. This assumption is consistent with damage observed on the existing shielding block structures at Lawrence Livermore National Laboratory, and of the compression zone from the U.S.D. concrete analyses in Appendix I.

Beyond the virtues of energy dissipation, another major gain in the rocking approach comes from the fact that the period of the structure becomes very long and shifts the response of the wall out to the tip of the response spectra where the structure, in spite of a significant earthquake, will only feel small "g" loads. For example, if the wall is bolted up it will have a natural frequency of 4 cycles

per second, which is toward the peak of the response spectrum, and it would have to withstand perhaps $2\frac{1}{2}$ times the peak ground acceleration. However, if the wall is allowed to rock to a displacement of 8 in. at the 32 ft level (the limit is set to keep the wall from colliding with the third floor of the adjacent building) and approximately 17 in. at the top, the period of the structure is approximately 2 seconds and the g level has dropped off to approximately one half of the peak ground acceleration (PGA), see Section II.

A way of visualizing the effect of "rocking" on an earthquake input is to separate the motion into two parts. One part is a "rigid-body" response and the other a rocking response, see Sketch 1.

Sketch 1.

First visualize the walls of the vault responding as rigid bodies. For ground accelerations equal to or less than $0.125g$'s the "rigid-body" portion of the response will be the shaded area of the acceleration curve in Fig. 1.* The rocking response will be only to the spikes outside this envelope.

* Fig. 1 is extracted from "Design of Multistory Reinforced Concrete Building for Earthquake Motion," Bland, Newmark and Corning.

Fig. 1. Ground Acceleration, Velocity, and Displacement, El Centro, CA Earthquake of May 18, 1940, N-S Component.

Hence, in the rocking mode that portion of the model responds to the spikes outside the rigid-body band. It is also observed these are short duration-high frequency components.

In reality, of course, the walls do not respond as "rigid-bodies" and the N-S walls will respond at about 4 Hz, hence, rocking will occur above about a peak ground acceleration of 0.05g, which is illustrated in Fig. 2. Although not as optimistic as a rigid-body, a great deal of the seismic event is handled by gravity alone and the rocking portion of the model must cope with short duration spikes.

Fig. 2. Ground Acceleration, Velocity, and Displacement, El Centro, CA Earthquake of May 18, 1940, N-S Component.

- B. The second effort (Section III) was applied toward studying all the soils data supplied by the Mirror Fusion Test Facility personnel and consolidating it into information most applicable to this particular project.
- C. The third portion of the effort (Sections IV and V) was the continued static evaluation of the wall footing soil interaction in light of dynamic data from the rocking analysis. This model includes a rigid

box approach for looking at the foundation portion of the building, i.e., including the torsional and bending resistance of the foundation walls and a major change in the approach of analyzing the massive shielding precast block wall as a "rocking" wall.

Section II DYNAMIC ANALYSIS

INTRODUCTION

A simplified dynamic analysis of the block walls of the Mirror Fusion Test Facility (MFTF) was performed as part of the determination of the facility's seismic capacity. In this analysis the walls were modeled as free-standing structures composed of independent blocks each stacked on top of one another. The configuration shown in Fig. 3a represents the out-of-plane motion of either the north or the south wall of the MFTF. Similarly, Fig. 4a represents the out-of-plane motion of the east wall, which is significantly shorter than the north and south walls. Since there is no west wall above ground elevation, out-of-plane motion of the north and south walls at the west end of the MFTF will be the most critical for seismic loads.

This section has been organized by subheadings, which describe the elements of work performed in the dynamic analysis. "Wall Rocking" defines the basic rocking phenomenon, as well as the size and location of the gaps that are assumed to form during wall rocking. "Simplified Approach" describes the approach and associated assumptions used to quantify the level of earthquake that the wall can sustain. Both impulsive and vibratory type loads are considered. "Response to Impulsive Loads" gives the magnitude and duration of half-sine pulses that could cause the north or south wall to overturn and indicates that this type of failure is highly unlikely. The following four subheadings ("Capacity Curves", "Bilinear Idealization", "Damping", and "Influence Due to Vertical Acceleration") are all part of the determination of wall capacity to withstand vibratory motion as defined by the LLNL Design Basis Earthquake Spectra (Design Spectra). "North and South Wall Capacity" provides the PGA levels that, in conjunction with the Design

Fig. 3. North or South MFTF Block Wall.

Fig. 4. East MFTF Block Wall.

Spectra, define the seismic capacity of the north and south walls. Similarly, "East Wall Capacity" provides the PGA levels that define the capacity for the east wall. A Summary and Conclusion is also provided.

WALL ROCKING

Large vertical steel rods (as shown in Fig. 3b for the north and south walls and in Fig. 4b for the east wall) pass through each block. These rods were originally designed as tension elements to resist overturning moment due to out-of-plane loads on the wall. At some elevations sleeve nuts are threaded on the rods and are used to transmit loads from the block elements to the rods. For the purpose of this analysis the sleeve nuts on the rods are assumed to be loose, permitting the block elements to tilt or rock when the lateral force due to seismic load exceeds the restoring force of gravity. Rocking is a phenomenon that has been observed in tall, unanchored structures subjected to dynamic loads. The rocking response mode has been used to explain why some structures survive dynamic loads that have peak acceleration values exceeding the static overturning capacity.

Without the rods, rotational gaps due to rocking could conceivably form at the joint between any two block elements or between the bottom block element and the top of the foundation wall. In the analysis it was assumed that the rod nuts are positioned on the rods such that small rotational gaps due to rocking can form at one or more locations. The size of the gaps is limited by the position of the sleeve nut on the rod. The assumed gaps for the north and south walls are 1 in. at the base of the wall (at elevation 0 ft-0 in.), $\frac{1}{2}$ in. between block elements 1 and 2 (at elevation 6 ft-6 in.), $\frac{1}{2}$ in. between block elements 2 and 3 (at elevation 13 ft-0 in.), and 1 in. between block elements 5 and 6 (at elevation 29 ft-0 in.). These gap sizes were selected to permit maximum rocking motion without excessive lateral displacement. If all four gaps are simultaneously developed, the lateral displacement at a wall elevation of 32 ft (i.e., 3rd floor of adjacent building) would be approximately 8 in. and at a wall elevation of 53 ft (i.e., top of the wall) would be approximately 17 in. (as shown schematically

in Fig. 3c). The assumed gaps for the east wall are 2 in. at the base of the wall (at elevation 0 ft-0 in.), and 1 in. between block elements 2 and 3 (at elevation 12 ft-9 in.). These gap sizes were selected to permit maximum rocking motion without excessive lateral deflection. If both gaps are fully developed, the lateral deflection at the top of the east wall (i.e., elevation of 29 ft) would be approximately 11 in. (as shown schematically in Fig. 4c).

Since additional lateral deflection due to elastic deformations of the steel rods will be small, the deflections given above are good estimates of the maximum deflections due to rocking for seismic loads that do not yield the steel rods. The above deflections are well below the lateral deflection limit of the wall at the point of instability, which is approximately one-half the width of the wall, or 42 in. Note, however, that the lateral deflections given above are based on the assumption that there is no translation or rotation of the foundation wall. This assumption may not be correct if the steel rods are highly stressed.

As rocking occurs, the wall will tilt up on one edge causing some local crushing of the block element at this point. The crushing was investigated and found to be of minor consequence (i.e., spalling of less than two inches of concrete). Further, the block elements have six-inch deep angled recesses at the block joint interface. These recesses insure that rocking cannot cause wall failure by a block's slipping laterally with respect to the block below.

Simplified Approach

The calculation of the dynamic response of a rocking wall generally requires a time history analysis since the problem is highly nonlinear. If a wall is composed of several block elements that can rock on one another, the analysis is even more complicated. An exact solution requires a step-by-step nonlinear dynamic solution of equilibrium equations for each block element of the wall excited by both horizontal and vertical time histories. This type of approach has been used by Lee (Ref. 1) in the analysis of a stacked fuel column. However, this approach was beyond the scope of the project, and the necessary time histories were not available.

As an alternative, a simplified approach based on a linear response spectrum technique was used to simulate the nonlinear wall rocking. The simplified approach considered rocking of the wall to initiate at only one gap location, although this gap could be at any one of the locations described above. Each of the four wall rocking configurations of the north and south walls was examined, and the one having the least resistance to lateral load was used to quantify the seismic capacity of the wall. Rocking at the base of the wall was found to be the most critical wall rocking configuration for the north or the south wall, although it was found that wall rocking at the other three elevations can also occur. Similarly, rocking at the base of the east wall was found to be the most critical configuration for that wall.

Wall rocking response was determined for two loading types, impulsive and vibratory. For impulsive loadings various half-sine wave acceleration pulses with different amplitudes and durations were applied to the north and south walls to determine both the force required to overturn the wall and the force required to create the assumed 1-inch rocking gap at the base of the wall. The vibratory analysis was performed by comparing the wall rocking capacity to the Design Spectra, which is shown in Fig. 5. The details of the solution for the two loading types are described in the following sections.

Response to Impulsive Loads

For slender blocks it can be shown (Ref. 2, 3 and 4) that the period, T_s , and amplitude, a , for a sinusoidal pulse required to overturn a block is related by the following equation:

$$\frac{a}{g_s} = \sqrt{1 + \frac{4R}{3g} \left(\frac{2\pi}{T_s} \right)^2} \quad (1)$$

Fig. 6 shows the block rocking geometry and defines the terms used in equation 1. This formula was solved using the north and south wall geometry, and various periods of the sinusoidal pulse. The corresponding overturning

Fig. 5. UCL Design Basis Earthquake Horizontal and Vertical Ground Response Spectra Peak Ground Acceleration Normalized to 1g.

$$C_y \sqrt{\frac{B}{H}} \cdot R$$

Fig. 6. Rocking Geometry.

Amplitudes are summarized in Table 1. The results indicate that very large acceleration levels are necessary at short periods to overturn the north or south wall. In the range of periods, 0.2 to 0.5 second, which corresponds to dominant accelerations of the Design Spectrum, a pulse amplitude of 1.43g to 2.73g is necessary for overturning to occur. These latter amplitudes indicate that, while the north or south wall may rock because of the acceleration pulses of an earthquake, overturning due to a single pulse is unlikely since the magnitude of pulse required for failure is far above the indicated pulse from an earthquake load. Since the east wall is shorter than the north and south walls, even greater acceleration pulses would be required for overturning failure. Hence, wall capacity appears to be limited by the wall's ability to resist vibratory motion.

WALL CAPACITY

In order to relate wall rocking to the Design Spectrum, lateral wall capacity was related to frequency of vibration. For each of the four assessed rocking configurations of the north and south walls a capacity curve was determined which is a plot of the restoring force on the block at a deflection as a function of rocking frequency. The restoring force is determined by the rocking geometry and, for small rocking deflections, is equivalent to the force required to overturn the block. The rocking frequency (Refs. 2, 3 and 4) was determined by:

$$\omega = \frac{4}{\pi} \cosh^{-1} \left(1 - \frac{1}{\gamma_1^2} \right) \quad (2)$$

Fig. 6 gives the definition of terms that are applicable to Equation 2. The capacity curves for the four north or south wall rocking geometries are shown in Fig. 7. It is important to note that for each configuration the rocking period becomes very long (i.e., .3 seconds) before the capacity begins to drop to zero, the overturning point. Likewise, capacity curves for the east wall indicate a similar lengthening of rocking period before a significant drop in wall capacity. Hence, as the wall rocks appreciably, the period of rocking becomes very long and the acceleration response defined by the Design Spectrum drops below the peak ground acceleration value.

Table 1. Half-Sine Pulse Period, T_s , and Amplitude, a

Overturning		1-Inch Gap at Wall Base	
T_s (sec)	a (g's)	T_s (sec)	a (g's)
0.2	3.73	0.2	3.56
0.4	1.86	0.4	2.78
0.6	1.49	0.6	2.22
1.0	0.75	1.0	1.11
2.0	0.37	2.0	0.56

Fig. 7. North and South Wall Capacity Curves.

BILINEAR IDEALIZATION

For relatively slender walls and moderate rocking deflections the restoring force due to gravity may be modeled as the maximum spring force of a bilinear system. That is, an equivalent force/displacement where the spring force is defined by the elastic capacity of the wall up to initiation of rocking, and where the spring force remains equal to the restoring force after rocking begins. Displacements represent lateral deflections of the center of gravity for the rocking wall. Fig. 8a shows such a bilinear curve for the wall. The dashed line in Fig. 8a represents an average stiffness corresponding to the lateral deflection. Rocking periods based on this stiffness and the mass of the wall were found to be very close to the rocking periods determined from equation 2, indicating that the bilinear representation of the rocking wall is accurate.

The bilinear idealization can be used to relate the linear Design Spectrum values to the nonlinear wall rocking accelerations. For relatively soft systems, such as a large rocking wall, the maximum relative displacement can be assumed to be the same as the displacement for a linear system. This assumption has been used in past work in the application of response spectra to nonlinear problems (Ref. 5). By equating the strain energy in the linear system shown in Fig. 8b with the bilinear system shown in Fig. 8a, it is seen that the bilinear system must have less force than its linear equivalent. Hence, it is conservative to use the linear Design Spectrum to predict maximum lateral forces on the rocking wall.

Maximum rocking response is assumed to occur when the rocking period (based on the lateral deflection of the wall) reaches the value at which the restoring force balances the Design Spectrum acceleration multiplied by the weight of the wall. This provided the basis for determining the wall capacity.

DAMPING

In applying the Design Spectrum a value of damping is assumed for the rocking system. Analysis of wall rocking does not use damping directly,

Fig. 8. Equivalent Linear and Bilinear Systems.

but instead, accounts for energy dissipation in the rocking equations by including a coefficient of restitution. Rocking equations given in Ref. 3, which are based on the coefficient of restitution, were reviewed and an equivalent viscous damping value was determined for different coefficient of restitution values. The equivalent viscous damping value is a nonlinear function of rocking amplitude, but appears to be stable after several cycles of free rocking vibration. Stable values of equivalent viscous damping are provided in Table 2 for different values of the coefficient of restitution.

The coefficient of restitution of concrete was not readily available. However, Ref. 6 provided values for glass (15/16), steel (5/9), wood (1/2) and clay (0). It is reasonable to assume that the coefficient of restitution for concrete will not be as high as glass and certainly can be as low as steel; thus, a value of 0.85 was conservatively selected for concrete. The equivalent viscous damping value corresponding to a 0.85 coefficient of restitution is approximately 10% of critical. Therefore, the Design Spectrum with 10% damping was used as a conservative basis for predicting acceleration response in the rocking wall.

INFLUENCE DUE TO VERTICAL ACCELERATION

The influence of vertical earthquake accelerations was considered by reducing the capacity curves by a factor, F:

$$F = 1 - \frac{2 S_a}{3g} \quad (3)$$

In equation 3, S_a is the greater of either the horizontal Design Spectrum acceleration value at the rocking period or the PGA value. For relatively slender walls, reducing the capacity by this factor is conservative since it implies that the vertical and horizontal components act in phase. The reduction factor, F, essentially decreases the restoring force by reducing the effective weight of the wall. It was noted that a reduction in weight increases the rocking period so that, at the same time the capacity is reduced owing to vertical load, the rocking period is increased. The

Table 2. Coefficient of Restitution and
Equivalent Viscous Damping

Coefficient of Restitution	Equivalent Viscous Damping
1.0	0%
0.95	3%
0.90	7%
0.85	10%
0.80	14%
0.50	40%

Design Spectra shown in Fig. 5 demonstrate that for long periods the spectral acceleration decreases significantly with an increase in period. As a result, the decreased capacity due to vertical acceleration was found to be approximately equal to the decreased spectral acceleration; and, therefore, at long periods and low spectral acceleration values the influence due to vertical acceleration was found to be of secondary importance based on the Design Spectrum approach. However, if a time history solution were to be used, the peak response would be, in general, sensitive to both the vertical and horizontal earthquake components since response would be based on the true phasing of the vertical and horizontal acceleration components.

North and South Wall Capacity

The seismic capacity of north and south walls is quantified in terms of the PGA, the value used to scale the Design Spectrum to match the north and south wall normalized capacity curve (i.e., capacity curves normalized by wall weight). Two levels of wall response were considered for comparing the Design Spectrum to the normalized capacity curves. The first level corresponds to rocking of the wall up to the gap size corresponding to initiation of stress in the steel rods. From geometric considerations, the lateral deflection at the center of gravity is 3.8 in. and the rocking period, including the vertical influence, is approximately 2.0 seconds. The normalized capacity at this period is approximately 0.1g horizontal acceleration. By equating the spectral value of the Design Spectrum at the 2.0-second period with this capacity, the corresponding PGA of the Design Spectrum is found to be approximately 0.2g horizontal acceleration. Note that the lateral deflection of the equivalent linear system at 0.1g and a 2.0-second period is 3.9 in., which is nearly the same as the lateral deflection of the center of gravity of the rocking wall.

After approximately 3.8 in. of lateral deflection at center of gravity of the north or south wall, the steel rods will be stressed and the wall capacity will increase as additional load is carried by the steel. By conservatively assuming that the period does not increase past 2.0 seconds, the lateral load carrying capacity of steel, approximately 0.075g at 33 ksi,

can be added to the 0.1g value to obtain the north and south wall rocking capacity at yield. Equating the spectral value of the 10% damped Design Spectrum with this capacity value, the corresponding PGA is found to be 0.35g. Fig. 9 graphically shows how the Design Spectrum and normalized capacity curves are matched.

The 0.35 PGA level is a conservative estimate of the earthquake that would cause the north or south wall to fail. Even after the steel reaches the yield point, the north and south walls may not fail since the restoring force due to gravity will still be acting on the walls. Further, as the steel stresses are developed, rotational gaps will form at all gap locations rather than just at the base of the walls and the increased lateral deflection will cause the period to significantly lengthen past the assumed 2.0-second value. At these longer periods the Design Spectrum has even less spectral acceleration than at the 2.0-second period, and a larger earthquake could be sustained.

East Wall Capacity

The seismic capacity of the east wall is quantified in terms of the PGA, the quantity used to scale the Design Spectrum to match the east wall normalized capacity curves. As in the preceding section, wall capacity was determined for two levels of response. At the first level, corresponding to rocking of the wall up to initiation of stress in the steel rods, the lateral deflection at the center of gravity is 4.1 in. and the rocking period, including the vertical influence, is approximately 1.6 seconds. The normalized capacity at this period is approximately 0.16g horizontal acceleration. By equating the spectral value of the Design Spectrum at the 1.6-second period with this capacity, the corresponding PGA of the Design Spectrum is found to be approximately 0.28g horizontal acceleration.

After approximately 4.1 in. of lateral deflection at center of gravity of the east wall, the steel rods will be stressed and the wall capacity will increase as additional load is carried by the steel. By conservatively assuming that the period does not increase past 1.6 seconds, the lateral load carrying capacity of steel, approximately 0.125g at 33 ksi, can be

Fig. 9. Matching Design Spectra and North and South Wall Normalized Capacity Curves.

added to the 0.16g value to obtain the wall rocking capacity at yield. Equating the spectral value of the 10% damped Design Spectrum with this capacity value, the corresponding PGA is found to be 0.46g. Fig. 10 graphically shows how the Design Spectrum and normalized capacity curves are matched.

The 0.46 PGA level is a conservative estimate of the earthquake that would cause the east wall to fail. Even after the steel reaches the yield point, the east wall may not fail since the restoring force due to gravity will still be acting on the wall. Further, as the steel stresses are developed, rotational gaps will form at the other gap locations, rather than just at the base of the wall, and the increased lateral deflection will cause the period to lengthen past the assumed 1.6-second value. At these longer periods the Design Spectrum has even less spectral acceleration than at the 1.6-second period, and a larger earthquake could be sustained.

SUMMARY AND CONCLUSION

The MFTF block walls were analyzed to determine their seismic capacity. The sleeve nuts were assumed to be positioned on the vertical steel rods to permit rocking of the wall before the blocks transmit load to the rods. A simplified approach based on the Design Spectrum shape and wall rocking capacity was used to quantify the level of earthquake that the block walls can survive. The results of the analysis indicate that the north and south walls can survive a 0.2g PGA and the east wall a 0.28g PGA LLNL Design Basis Earthquake before rocking is sufficient to produce stress in the steel rods. When the stress in the steel rods reaches yield (i.e., 33 ksi) the results indicate that the north and south walls can survive a 0.35g PGA and the east wall a 0.46g PGA LLNL Design Basis Earthquake.

Fig. 10. Matching Design Spectra and East Wall Normalized Capacity Curves.

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Section III SOIL PARAMETERS

GENERAL

No borings were drilled and no tests were performed for this study. Soil parameters used in our analyses were either taken or interpreted from information presented in the following reports.

1. "Preliminary Soil and Foundation Investigation for Magnetic Flux Testing Facilities, Building 431, Lawrence Livermore Laboratory, Livermore, California", by Bechtel Incorporated, December, 1977.
2. "Foundation Investigation, Proposed Accelerator Building, Project X-1, Naval Air Station, Livermore, California, for California Research and Development Company", by Dames & Moore, June 1, 1980.
3. "Soil Investigation for the Proposed Astron Tunnel, Building 157, Extension, Lawrence Radiation Laboratory, Livermore, California", by Woodward-Clyde-Sherard & Associates, March 13, 1964.
4. "Geotechnical Engineering Services for the Proposed Neutral Beam Test Stand Facility, Building 431, Lawrence Livermore Laboratory, Livermore, California, for Garretson, Elmendorf, Zinov, Reibin", by Woodward-Clyde Consultants, November 19, 1975.
5. "Soil Investigation for the Proposed A.E.C. Sherwood Laboratory Building 435, Livermore, California", by Woodward - Clyde - Sherard and Associates, May 20, 1958.
6. "Foundation Investigation for 230/13.8 KV Substation, Lawrence Livermore Laboratory, Livermore, California", by Peter Kaldvoer and Associates, January 3, 1979.

These reports were prepared by others during previous investigations in the general vicinity of the project site.

Shield Walls

The existing shield walls, as shown in Fig. 11, are about 27 feet in

Fig. 11. Field Wall.

width of 17 feet in width at the line of the footing. The adjacent floor slab is at elevation minus 21 feet.

Bearing Capacity

A net ultimate bearing capacity, q_{ult} , of 25 kips per square foot for a flat foundation at elevation minus 35 ft was quoted in referenced Figure 10.1 using the following equation:

$$q_{ult} = 1.3 C H_c$$

where C = undrained shear strength

H_c = bearing capacity factor

This value corresponds to an undrained shear strength of about 4 ksf as presented in Figure 5 of Ref. 1.

A value of 16 ksf for " q_{ult} " may be used for a footing at elevation minus 27 ft, based on a value of " C " of 3.2 ksf as indicated in the same referenced Figure 5 of Ref. 1.

No data were available for determining the yield strength of soils at elevation minus 27 ft. A reduction factor of 20% was applied to the peak

shear strength as mentioned above and a value of 13 ksf was calculated as the yield limiting value.

Passive Resistance and Bulgrade Reaction

Other soil parameters used in our structural analyses are calculated as follows:

Passive resistance = calculated based on 300 psf of equivalent fluid pressure

Bulgrade reaction coefficient k_{sg} for 1' x 1' plates = 300 P.C.F.

Section IV SUPPORT COMPUTATION

While Section II on Dynamic Rocking Analysis and Section III on the Soil Analysis were underway, supporting analyses to evaluate some of the structural capabilities were continued and are shown in Appendix II. These analyses were used to develop the bending capabilities and some of the torsional capabilities of the foundation structure in preparation for analysis that would be performed after the dynamics data and the soils data became available. From these computations it became evident that the foundation structure had torsional capabilities and flexural capabilities that could share the load induced by the wall above zero elevation. This would alleviate some of the apparent soil and flexural problems of the vault structure that were pointed out by the simplified assumptions of Section I. In fact, from this analysis it appears the entire foundation wall footing structure can be treated as a nearly rigid body and will remain elastic up until or near the ultimate strength of the block shielding wall above the zero elevation line.

Section V STATIC ANALYSIS WITH DYNAMIC IMPLICATIONS

As the dynamic analysis of Section II and the soil analysis of Section III become available, a static analysis of the wall system was conducted with the implications from the previous two sections included in the analysis.

As the rocking analysis progressed, displacements were studied, and it turned out that the wall structure at the 32-ft elevation line of the building was approximately 8 in. from the wall. Hence, gaps were established at various levels analytically to allow a maximum rocking excursion of 8 in. at the 32-ft level, or the third floor, and a maximum of 17 in. at the top of the wall at the 63-ft elevation. These gaps will be established by backing off the sleeve nuts at various elevations; A gap of 1 in. will be allowed at the zero elevation by increasing the sleeve nuts at the 6-ft, 4-in. elevation where the rods change from 2½-in. rods to 1-3/4 inches. At the 12-ft, 9-in. elevation, where the rods change from 1-3/4 in. to 1½ in., a ½-in. gap will be set, and an additional 1-in. gap at the 23-ft elevation where the rods are spliced, and a 1-in. gap at the top. This concept essentially allows the wall to rock with these excursions and not put any load into the rods. Hence, the load from the wall to the foundation and footing essentially remains vertical, and no moment is transmitted at zero elevation, just a shifting of the mass from edge to edge.

With a static philosophy one could achieve this rocking at the zero elevation at 0.125g. However, the dynamic or rocking analysis shows that this is actually equivalent to a peak ground acceleration of 0.2g, and at this time there is no stress in the steel rods whatsoever. In fact, the

entire wall including the foundation, will remain elastic during this rocking. The maximum foundation soil stress is 13.6 ksf and the yield is estimated at 13.2 ksf. Hence, from this analysis the dynamic capability of the wall is such that, for a peak ground acceleration of 0.2g, there will be no stress in the rods and the entire system remains elastic.

The dynamic analysis in Section II (Fig. 9) shows a lower bound curve with a steel shear of 33 ksi in the 1½-in. rods. This is a lower bound according to the analyses. In fact, from the static/dynamic analysis, the 1-3/4 in. rods will yield at about the same time as the 2½ in. rods at the base yield. This could be shown as the upper bound curve in Section II and represents yielding occurring in the 2½ in. rods at a dynamic peak ground acceleration of 0.56g, which is well above any requirement for the site. Section II, Fig. 9 has only the lower bound 0.36 peak ground acceleration curve plotted, which produces little distress in the foundation structure.

Appendix III analyses indicate that this yielding of the rods occurs at an equivalent static uniform acceleration applied to the wall of 0.270g. This level will be called the ultimate moment capacity of the wall and is more than twice as high as the totally elastic "rocking impending" 0.125g level. It must be observed that, although this is designated the ultimate capacity, this is static and, with the dynamic approach, further deformation could be permitted by going on into the yield domain of these rods. However, it does not appear necessary to further complicate the analysis, when the 0.270g static is exactly 1.4 greater than the maximum UBC of 0.19g. Hence, it appears to be quite conservative and consistent throughout.

Fig. 12, taken from Appendix III, shows the various static resistances of the rocking wall; i.e., gravity resistance alone at 0.125g and the yielding rod analysis of 0.270g. Fig. 12 also illustrates the cleverness of the original design of this structure as far as spreading out the resistances of the components in a rational manner is concerned. The analysis

Fig. 12.

of the footing structure in this section (the structure below the zero elevation including the retaining wall and footing) shows that the entire system will probably remain nearly elastic right up until the 0.27g static equivalent g loading, or the upper third 0.56 peak ground acceleration level. At that time some flexural cracking of the retaining wall on extensions toward the pit, and some torsional cracking of the retaining wall along the north and south walls can be expected, as the friction is being transmitted to the end walls. Additionally at the time these crackings occur, the deformations will be one larger and the soil will achieve its ultimate or at least will be well into the yield domain over the outer 5 or 6 ft of the toe closest to the pit. In other words, it looks as if the entire structural system at structure end and a peak ground acceleration of 0.56g, or the static equivalent of 0.27g with uniform acceleration, will reach some considerable distress. There will be spalling and yielding in the retaining wall and footing portion of the structure. There will be torsional cracks and yielding of the soil. Of course, all of these ultimate deformations will actually increase the period in the same direction of the seismic response of the structure as, while the deformation increases, the period lengthens and the response amplitudes are increased.

Appendix IV is the analysis of the east wall. The east wall had been somewhat neglected to this point in the study, in that it is only 24 ft high and thus, will induce much lower loads all through the east structural system. For example, the static equivalent required to induce cracking above the zero elevation is 0.230g and a dynamic spectral level of 0.28g from Fig. 10, with stressing the rods, and a 0.46 peak ground acceleration at rod yield. Hence, there is over twice the resistance to cracking on the eastern wall as on the north and south walls, and further, the mass of the wall is sufficiently less to lower the stresses all through the east wall.

The east wall is constructed somewhat differently from the north and south walls, in that the portion of the wall from minus 21 up to elevation zero is actually cast-in-place blocks. These blocks are similar in design,

but are keyed somewhat differently from the blocks above elevation zero. Further, they are resting against the soil on the east side so they cannot rock, hence the rods from the zero soil elevation to the footing on the east side will not be loosened. The block structure on the eastern wall between zero and minus 21 ft elevation will remain tight and be treated as poured-in-place reinforced concrete. The wall above the zero elevation will be allowed to rock. The east wall is more heavily reinforced below the zero elevation mark than the north and south walls. Hence, it has a somewhat greater capacity below grade. The east wall will essentially remain elastic up until approximately 0.23g. At 0.23g, some yielding of the soil may occur; little else will happen. The analysis of the east wall is not extended on into the ultimate domain below the zero ground elevation, for it will be well in excess of the 0.27g static equivalent on both the north and south walls, and it seemed little would be gained by crunching more numbers. These computations appear in Appendix IV.

Appendix V is an analysis performed on the behavior of the system as the wall(s) rocked toward the soil and away from the pit. In this direction, very large passive resistances are possible by the confined soil around the pit. The soil is being confined by the floor and building above it. In this analysis these confining stresses were fundamentally ignored (which is conservative) and an evaluation of the floor and surrounding structural system was performed to provide some feeling for the rocking and the ultimate effect on the floor. As concluded earlier in the toward-the-pit analysis, during the rocking mode the entire system will remain elastic and there will be little effect on the surrounding floor system or even for significant passive stresses in the soil. However, when the system approaches the ultimate capacity of the wall above the zero elevation level (full rocking plus yielding of the steel rods) some distress in the floor system can be expected.

At 0.27g static, or a peak ground acceleration equivalent of 0.56g, fairly large quantities of load will be transmitted into the floor system at the top (elevation zero) from the shear being transferred to the foundation by the rocking wall above. The shears are initially on the order of

15 kips per lineal foot, and could theoretically put loads into the drilled piers and the building of about 100 kips each. However, as soon as some deformation takes place, larger passive resistances of the soil develop. If the top 2½ ft of the soil yield or develop a passive resistance of 350 psf per foot of depth, and further if this 2½ ft of soil deforms plastically, no shears will be transmitted into the floor system. The entire building and direction of motion can be resisted by the strength of the wall, the footing, and the passive resistance of the soil. It appears that the racking toward the building or backfill side of the retaining wall structure will induce moderate loads initially into the footing system of the building, but as soon as a small amount of deformation takes place the loads will be carried by the soil-wall-footing combination and will allow the wall structure to achieve its full maximum potential seismic resistance.

SUMMARY/RECOMMENDATIONS

This report presents a seismic analysis of the Mirror Fusion Test Facility shielding vault housed in Building 431 at Lawrence Livermore National Laboratory. The shielding vault structure is approximately 120 ft long by 80 ft wide and is constructed of concrete blocks approximately 7 x 7 x 7 ft. The north and south walls are approximately 53 ft high and the east wall is approximately 29 ft high. These walls are supported on a monolithic concrete foundation that surrounds a 21 ft deep open pit. The analyses included:

- o Dynamic analysis of "Rocking Blocks" above the zero elevation level.
- o Evaluation of the soil parameters from six existing LLNL reports on the region in or near the MFTF vault, and
- o Static analyses of the rocking blocks, the monolithic foundation, the soil loadings, and dynamic implications of these analyses.

It is recommended that the rods in the north and south block walls be adjusted to permit gaps of 1 inch at zero elevation, $\frac{1}{2}$ inch at 6 ft 4 in. elevation, 1 inch at 12 ft 9 in. and 1 inch at 29 ft elevation. These gaps permit the block wall to rock freely and the rods act as snubbers or motion arrestors, which keep the rocking motion within the desired envelope (8 inches at the third floor level of 32 ft elevation and 17 inches at 53 ft).

The east wall recommendation is for no gap below the zero elevation, and a moderate preload (10 ksi) would be in order if possible. The rods above zero elevation should be set to provide a 2 inch gap at both the zero elevation and the 12 ft 9 in. elevation. Weak-frangible shim stock should be used to set gap at the respective sleeve-nut locations. These shims will control the gaps and not inhibit the rocking motion.

Analysis of the structural system with the foregoing modifications indicate that the north and south walls of the vault structure will survive a 0.28 PGA LLNL Design Basis earthquake and the east wall a 0.35 PGA without stressing the block snubbing rods, and the foundation structure will remain elastic. Yielding of the snubbing rods (herein conservatively referred to as the ultimate capacity) increases the survivability of the vault to a PGA between 0.35g and 0.56g for the north and south walls and approximately 0.46g PGA for the east wall. In the range of 0.35g PGA to 0.56g PGA the foundation and soil structure will show distress in the form of flexural and torsional cracking and permanent soil deformation. It must be kept in mind that a particular seismic event may excite an actual response that is not within the particular Design Spectra used in this report and could well change the predicted response and/or damage levels.

Appendix 1

Gabrielsen

2001

Sept. 29, 2001

Look at Free-Standing Wall (NAS Wall)

Use

U.B.C. 79

Sec 2312

ACI-77

$f'_c = 3000 \text{ psi}$

$E_c = 3.12 E_s$

$E_s = 29 E_s$

$\gamma_c = 150 \text{ pcf}$

$\gamma_s = 110 \text{ pcf}$

let: $K_a = 0.35$

$\mu_{s/c} = 0.35$

equiv fluid

$w = 40 \text{ pcf}$

SO.

Seismically

$f_y = 33 \text{ ksi}$

V & U

Seismic estimate

$$V = Z I K C S W$$

$$Z = 1$$

$$I = 1$$

$$K = 1.33$$

$$CS = 0.14$$

$$\left. \begin{array}{l} Z = 1 \\ I = 1 \\ K = 1.33 \\ CS = 0.14 \end{array} \right\} V = 0.19 W$$

Alternately for Discussion
Sake we may consider this
as an element of a Bldg.

$$V = Z I C_p^* S W_r$$

$$\text{or } V = 0.20 W_p$$

Both of which are higher than
previously used, but are intentionally
higher until we learn more of
the soil at the site

$$V = 0.20 \left(\frac{7}{5} \right) 150 \text{ \# / ft}$$

$$V = 210 \text{ lb / ft from the}$$

Wall itself

* From U.B.C. '76 or 0.3 W - UBC '79

Wall induced Seismic Shear & M_s

$$V = 210 \text{ \#/ft}$$

V

M

Soil Effects Below Grade

(Neglect Surcharge)

Loading acts inward only

V

M

Look at Wall Base (elev -21)

let: $d = 80 \text{ in}$
 $b = 12 \text{ in}$
 $f_y = 35 \text{ ksi}$

$$W = 74 \text{ ft} \times 7 \text{ ft}^2 \times 150 \text{ pcf}$$

$$W = 77.70 \text{ kips}$$

Case I - Toward Soil

$$V = 15,540 \text{ lbs/ft}$$

$$M = 574,980 \text{ ft-lbs/ft}$$

$$U = 0.90 + 1.43E$$

$$\therefore V_u = 22.222 \text{ kips}$$

$$M_u = 822,221 \text{ kip-ft}$$

$$W_u = .9W = 69.93 \text{ k}$$

$$A_s = 0.79 \text{ in}^2/\text{ft}$$

UPT

$$a = \frac{A_s f_y}{0.85 f'_c b}$$

$$f_y = 33$$

$$a = \frac{0.79 \times 33}{0.85 (3.0) 12}$$

$$f'_c = 3$$

$$d = 80$$

$$a = 0.85 \text{ in}$$

$$M_n = A_s f_y \left(d - \frac{a}{2} \right) + \frac{4.0 \text{ in}}{12} W_u$$

$$M_n = 172.87 \text{ kft} + 233.1 \text{ kft}$$

$$M_u = \phi M_n$$

$$M_u = 155.59 \text{ kft} < 822 \text{ kft}$$

$$+ 209.79$$

$$365.38 \text{ kft}$$

∴ No Good

Case II - Toward Vault.

$$V = 15,540 + 11,025$$

$$= 26,565 \text{ #}$$

$$\therefore V_u = 37,988 \text{ lbs}$$

$$M = 574,980 + 77,175 \text{ ft-lbs}$$

$$= 652,155$$

$$\therefore M_u = 932,582 \text{ ft-lbs}$$

$$W_u = 69.93 \text{ kips}$$

$$A_s = 1.58 \text{ in}^2/\text{ft} \quad f_y = 33 \text{ ksi}$$

$$a = \frac{1.58(33)}{0.85(3)12} \quad d = 80$$

$$a = 1.70 \text{ in}$$

$$\begin{aligned} M_n &= A_s f_y \left(d - \frac{a}{2}\right) + \frac{40}{12} (W_n) \\ &= 343.90 \text{ kft} + 233.1 \text{ kft} \end{aligned}$$

$$M_u = \phi M_n$$

$$= 309.51 + 209.79$$

$$M_u = 519.30 \text{ kft} < 933 \text{ kft}$$

$\therefore$ No Good

$$V_u = 37.988 \text{ k}$$

$$v_u = \frac{37.988}{12 \times 80}, \quad \phi \geq 0.85$$

$$v_u = 39.57 \text{ psi} < \phi 2 \sqrt{f'_c}$$

$$39.57 \text{ psi} < 93.1 \therefore \text{OK}$$

673

$$\begin{array}{r} 54 \times 6 \\ 324 \end{array}$$

$$+ 3(2835)$$

636,940

$$\Sigma V = \underline{34,600 \text{ lbs}}$$

$$\approx 34.6 \text{ k}$$

close some passive available

Soil pressure

$$p = \frac{\sum W}{A} + \frac{\sum M}{S}$$

where $A = 17.5 \text{ ft}^2$ & $S = \frac{1}{6} (17)^2$
 $= 48.17$

$$p = \frac{98,810}{17} + \frac{636,890}{48.17}$$

$$p = 5,812 \pm 13,222$$

$$\left. \begin{array}{l} p = +19,033 \text{ psf} \\ a = 7410 \text{ psf Tension} \end{array} \right\} \text{No Good}$$

$$\begin{aligned} x &= \frac{636,890}{98,810} \\ &= 6.45 \text{ ft} \end{aligned}$$

$$\begin{aligned} p_{\text{elastic}} &= \frac{98,810}{3} \\ &= \underline{\underline{32,937 \text{ psf}}} \end{aligned}$$

No Good!

Note Look @ U.S.P. thoughts
 in future, Toe & Heel steel
 etc.

Reverse Loading

$$\Sigma V = 15,540 + 2,835$$

$$= 18,375 \text{ lbs} \quad \bullet \text{ Sliding O.K.}$$

$$\Sigma M_c = 574,980 + 6(15,540) + 3(2,835) + 2(77,700) + 7(6,930)$$

$$\Sigma M_c = 880,635$$

$$\Sigma W = 77,700 + 6,930 + 14,180$$

$$= \cancel{880,635} \text{ lbs} \quad 98,810$$

$$p = \frac{\Sigma W}{A} \pm \frac{\Sigma M}{S}$$

$$= \frac{98,810}{17} \pm \frac{880,635}{48.17}$$

$$p = \underline{5812 \pm 18,282} \quad \leftarrow \text{No Good}$$

At this point it is obvious that the building cannot ultimately behave this way, i.e. a cantilever wall. The roof system, of course, will make the problem even worse. Take a future look at pin-fixed, pin-pin etc. when modeling the entire Bldg.

Look at Base of Block - Elev O

$$A_s \text{ rod} = 3.98 \text{ in}^2 / 3.5 \text{ ft} \\ = 1.14 \text{ in}^2 / \text{ft} > \# 8 @ 12 \text{ in}$$

$$\therefore A_s = \# 8 @ 12 \text{ or } \underline{0.79 \text{ in}^2 / \text{ft}}$$

R = 5. pg 5

$$M_u = 155.59 \text{ k.ft.}$$

$$\phi M_u = 0.9 \left(\frac{40}{12} \right) \cdot \underbrace{.9(55,650)}_{W_u}$$

$$= 150.26 \text{ k.ft.}$$

$$\therefore M_u = 305.85 \text{ k.ft.}$$

Resisting Moment

Applied Moment

$$M_u = 1.43 (294,945 \text{ ft.lbs})$$

$$= 421.77 > 305.85$$

$\therefore$ No Good

M_u applied

M_u allowable = 1.38

Note

o.k.

for

$C_p = 0.14$

Look @ Hook

Ref ACI 318-63

ASTM A908 Smooth

$$u_n = G \sqrt{f_c'} / 2 =$$

$$u = 250 \text{ psi}$$

$$u_n = \underline{u = 164.32 \text{ psi}}$$

$$\lambda_2 = 63 - 4(2.25)$$

$$P_y = \left[\frac{u_n}{\lambda_2 = 59} \right] + \left[\right]$$

$$P_y = \pi D \lambda_2 u_n + H_{vol}$$

$$A_s f_y = 62720 + H_{vol}$$

$$131,210 = 62720 + H_{vol}$$

$$\therefore H_{vol} = 68,490 \text{ lbs}$$

probably in keeping with 63
philosophy H_{vol}

$$f_u = 17,225 \text{ psi}$$

$$63 \text{ ACI } A_v + 9194 \quad f_u = \underline{19,000}$$

$\therefore O.K.$

Next Look at Shortened Wall

at Elev 0. for $x = 38.5$

$$V = 210x$$

$$= 8,085 \text{ lbs}$$

Ref. pg 3

$$M = 105x^2$$

$$= 155,636 \text{ ft-lbs/ft}$$

$$W = 1050x$$

$$= 40,425 \text{ lbs/ft}$$

applied

$$M_u = 1.43M$$

$$= 222,559 \text{ ft-lbs/ft}$$

Resisting from pg 11

$$155,590 + \overbrace{40,425(.9)\phi\left(\frac{40}{12}\right)}^{w_u}$$

$$M_u = \cancel{305,850} \text{ ft-lbs/ft}$$

$$= 264,738 \therefore \text{Safe}$$

d.k.

ϕ
 $C_p = .2$
 $\phi = .29$

Look at Elev -21 $x = 59.5$

$$V = 210x$$

$$V = 12,495 \text{ lbs/ft}$$

$$M = 105x^2$$

$$M = 371,726 \text{ ft-lb/ft}$$

$$W = 62,475 \text{ lbs/ft}$$

0524

14

$$W_u = .9W$$

$$= 56,228 \text{ lb}$$

Toward Soil

$$V_u = 1.43V$$

$$= 17,868 \text{ lb}$$

$$M_u = 1.43M$$

$$M_u = 531,568 \text{ ft-lb} \text{ applied}$$

$$M_u = 155,540 \text{ py S}$$

$$+ 56,228 (.9) \frac{40}{12}$$

$$= 324,274 \text{ ft-lb} \text{ Resisting}$$

No Good!

Overstressed

$$1.69 \div 1.00$$

 $\phi = 1.2$
Toward P, T

$$V = 12,495 + 11,025$$

$$V = 23,520$$

$$V_u = 33,634 \text{ lbs/ft} \text{ ref.}$$

$$M = 371,726 + 77,175 \text{ py S}$$

$$M = 448,901$$

$$M_u = 641,928 \text{ ft-lb/ft}$$

$$W_u = 56,228 \text{ lb}$$

BS 81

15

Resisting.

$$M_u = 309,510 + 0.9(40)_{12} 56,228$$

$$309,510 + 168,684$$

$$M_u = \underline{478,194 \text{ ft lbs}}$$

Over Stressed 1.39:1.00

No Good!

8 Oct 80

10524

East Wall (Dwg. XB-3R-329)
missing

$f_y = 33 \text{ ksi}$

Vault
side

Soil
side

$$A_s = 2 \times \frac{\pi (0.75)^2}{7} \frac{\text{in}^2}{\text{ft}}, \quad A_s = 2 \times \frac{\pi (1.125)^2}{7} \frac{\text{in}^2}{\text{ft}}$$

$$A_s = \underline{0.50 \text{ in}^2/\text{ft}}$$

$$A_s = 1.14 \text{ in}^2/\text{ft}$$

$$d = 80 \text{ in}$$

$$d = 80$$

$$M_u = A_s f_y d$$

$$M_u = A_s f_y d$$

$$M_u = 0.5 (33) \frac{40}{12}$$

$$= 1.14 (33) \frac{40}{12}$$

$$M_u = 55.5 \text{ k ft/ft}$$

$$M_u = 124.46$$

$$M_u = 49.98 \text{ k ft/ft}$$

$$M_u = 112.47 \text{ k ft/ft}$$

Vault Direction

Soil Direction

Gravity Component $\perp$ 30.5 ft Wall

$$M_u = 0.9 (1.05 \frac{\text{k}}{\text{ft}}) (59.5) \left(\frac{40}{12} \right) = 187.43 \text{ k ft}$$

$$M_u = 168.68 \text{ k ft}$$

USFS

1/

Summary at Base of East Wall

$$M_u(\text{Applied}) = 531.57 \text{ k-ft}$$

$$M_u(\text{Applied}) = 671.93$$

$$\phi W_u = 56.23 \text{ k-ft}$$

$$M_u(\text{Resisting}) = 218.66 \text{ k-ft}$$

$$M_u(\text{Resisting}) = 281.15 \text{ k-ft}$$

Look at Alt. Model

A Cast-In-Place "Rigid Box"
on an Elastic Foundation!

North

U-5-1

Torsional Resistance Permed Walls

$$\frac{x^2 y}{3} = 313 \text{ ft}^3$$

$$6' = x_2 \quad \frac{x^2 y}{3} = 180 \text{ ft}^3$$

$$T_n = n_f \sum \frac{x^2 y}{3} \quad \text{Ref. Ferry p. 153}$$

Approx formula

$$n_{fc} = 1.5 \sqrt{f_c'} \quad \text{Without special provision}$$

$$T_{nc} = 1.5 \sqrt{f_c'} \sum \frac{x^2 y}{3}$$

Appendix II

2002

U b d

Bending Resistance N, W, & S Walls

Below Elev.

$$f_c' = 3000 \text{ psi}$$

$$f_y = 33000$$

$$E_c = 3.12 \text{ eV}$$

$$k_s = 291.6'$$

A. sumer. Positive Moment is provided by steel in footing for First estimate, i.e. top in compression

$$\begin{aligned}
 a &= A_s f_y / 0.85 f_c' b \\
 &= 14.07 \times 33 / 0.85 (3) 84
 \end{aligned}$$

$$a = 2.17 \text{ in}$$

Very little concrete need
to balance Steel N.A. ~~area~~
e.g.s. based on Strain
compatibility.

$$\epsilon_y = \frac{33}{2953} = 0.0114$$

assume 10%
max tension strain

$$\begin{aligned}
 A_s (\text{reqd}) &= (20 - 7.76) 0.88 \\
 &= 10.77 \text{ in}^2
 \end{aligned}$$

$$A_s (F_y) = 3.13 \text{ in}^2$$

$$A_s (F_y) = 10.94 \text{ in}^2$$

Neglect Comp. Steel

$$a = \frac{33(10.77 + 3.13 + 10.99)}{89(3)0.85}$$

$$a = 1 \text{ in.} \quad \begin{array}{l} \text{let } \text{respective} \\ \text{2 dis. be} \\ \text{e-l-u.} = 6'' \end{array}$$

$$M_n = 7A_s S_y z_c$$

$$\begin{aligned} &= 10.77 \text{ in}^2 (33 \text{ ksi}) (13.88 - .5) \text{ ft} \\ &+ 3.13 \text{ in}^2 (33) (24.5 - .5) \text{ ft} \\ &+ 10.99 (33) (27 - .5) \end{aligned}$$

$$M_n = 16,801 \text{ K ft.}$$

$$\underline{\underline{M_u = 15,121 \text{ K ft.}}} \quad \text{Positive } M_u$$

Next = Compute Negative Capacity

Use Strain Capacity approach

Neglect Comp. Steel
e-l-u.

(1)

Design Torsional Capacity of NSC

$$\left. \begin{array}{l} T_{nc} = 6188 \text{ k-ft} \\ T_{nc} = 5239 \text{ k-ft} \end{array} \right\} \text{RHS by 20 sec 1}$$

Only the Steel provides
additional Torsional capacity
at 44 inch

in the absence of beam shear
the concrete $v_c = 5\sqrt{6450}$

$$\text{i.e. } V_{hc} = 20,627 \text{ to } 29,152 \text{ ksf}$$

$$\text{and } V_{hu} = 15,777 \text{ to } 21,039 \text{ ksf}$$

These are the same order of
Magnitude as M_u & M_u . If
no beam type shear is present,
which ~~is~~ seem reasonable as
the beam type of shear is
considered in the soil footings.
 M_u is induced fundamentally
by the seismic forces.

" T_{us} " by the steel would
increase this Torsional
capability.

$$A_s = 0.44(19) = 8.36 \text{ in}^2$$

each side

$$T_{us} = A_s f_y$$

$$T_{us} = A_s f_y (d - d')$$

$$= 8.36(33)(84 - 6)$$

$$T_{us} = \frac{1793}{12} \text{ ksf additional}$$

- Look at
- 1) North Wall
 - 2) Assume North Quay
 - 3) ✓ E & W end have the Same Moment Resistance & Soil is flexible compared to Concrete, i.e. Ultimate Moment can be developed before Soil load acts.

$$M_{\text{cor}} = 621.62 \text{ (Live)} - 28.4 \text{ (Dead)}$$

$$\sum W_i = 91^9 (120) \quad \square$$

$$\Sigma M = (621.61 - 289.0) / 20$$

$$\sum V = 15.54 (120)^L$$

$$\sum W_{cu} = 0.9 \sum W_c = 7,817.2 \text{ k}$$

$$\sum M_u = 1.49L + 0.9D = 101,365 \text{ k ft}$$

$$\sum V_{cu} = 1.4V = 1865 \text{ k}$$

Equilibrium of Section

2-M axial shear center

$$7,817e = 101,365 \text{ k ft} - 29,328 \text{ k ft}$$

$$e = 7.39 \text{ ft}$$

Not good - we have an $e = 6.5$
 max available - must use
 passive soil in this direction -

< Look at from a strain point of view >

Assume 100 lb/ft^2 passive

i.e. uniform passive Resistance

$p = 100$ - Note this is

backfill and we

could ignore part

of the 27 ft

$$p_u = 140 \text{ lb/ft}^2$$

$$H_u = 27(140) = 3780 \text{ lb} \quad \Delta$$

$$M_u = 27(140)\frac{27}{2} = 51.03 \text{ k ft / ft}$$

$$\text{or } M_u = 6,124 \text{ k ft} \quad \left\{ \begin{array}{l} \text{for the 120 ft} \\ \text{of wall} \end{array} \right.$$

Need a larger passive
to do much - i.e. 350 p.s.f. or
more - See pg 8, $\Sigma M_u = 720.37$

$$\text{@ } p = 350, M_u = 21,439$$

which makes $e = 5.15$ (see pg 2)

$$p_s = 5.15$$

$$p_s = \frac{9817 \text{ k}}{2.69(120) \text{ ft}^2}$$

$$p_s = 30 \text{ ksf} \quad \text{too much}$$

Loaded Against Vault

Ref. pg 8

~~9,817e = 101,565 k s + 29,320 k s~~

neglecting $\left\{ \begin{array}{l} \text{ref. p4} \\ \text{passive} \end{array} \right. \Rightarrow + \frac{120 \times 35}{1000} \frac{y^3}{3}$

Passive ($y = 27$ ft

$$9,817 \text{ e} = 101,365 - 29,328$$

S.C. $e = 9.34 ft + 19,683$

$$e_{ps} = 35.35 \text{ ksf}$$

High

looks like some chance of making the facility function with an elasto-plastic $0.15 W = V$

Appendix III

look at free-standing wall (N.S. wall)

Use

U.B.C. 19

Sec 2312

ACI-11

$f_c' = 3000 \text{ psi}$

$E_c = 3.12 \times 10^6$

$E_s = 29 \times 10^6$

$f_c = 150 \text{ pcf}$

$f_s = 110 \text{ pcf}$

let: $K_a = 0.35$

$M_{3/4} = 0.35$

equiv fluid

$w = 40 \text{ pcf}$

5.0 Seismically

$f_y = 33 \text{ ksi}$

$W_1 = \frac{55.65}{7.70} \text{ k}$

$W_2 = 49.00$

$W_3 = 42.27$

$W_4 = 25.20$

Basic Moment Capacity

1) Wall - Below Elev. 0

$$\left. \begin{array}{l} M_u = 155 \text{ K ft / ft} \\ M_n = 173 \text{ K ft / ft} \end{array} \right\} \begin{array}{l} \text{pg 5 part I} \\ \text{assumed} \end{array}$$

2) Elev. 0 to 6' 4"

$$A_s = 2 \left(2 \frac{1}{4} \phi \text{ Rods} \right) / 7 \text{ ft}$$

$$A_s = (0.57) 2$$

$$A_s = 1.14 \text{ in}^2 / \text{ft}$$

$$\text{let } d = 78 \text{ in (No. check ff)}$$

$$a = \frac{A_s f_y}{0.85 f_c' b}$$

$$a = \frac{1.14 (36)}{0.85 (3) 12} \rightarrow 33$$

$$a = 1.39$$

~~2.4~~

$$d - \frac{a}{2} = 77.33$$

$$M_n = A_s f_y \left(d - \frac{a}{2} \right)$$

$$M_n = \frac{245}{265} \text{ K ft / ft}$$

$$M_n = \frac{219}{230} \text{ K ft / ft}$$

3) Elev 6' 4" to 12' 9"

$$A_s = 2(1\frac{3}{4}\phi) / 7 \text{ ft}$$

$$A_s = 0.687 \text{ in}^2 / \text{ft}$$

$$M_u = A_s f_y \left(\frac{77.3 \text{ in}}{12 \text{ in}} \right) / \text{ft}$$

$$M_u = \frac{146}{159} \text{ K ft}$$

$$M_u = \frac{131}{143} \text{ K ft} / \text{ft}$$

4) Elev 12' 9" to 29

$$A_s = 2(1\frac{1}{2}\phi) / 7 \text{ ft}$$

$$A_s = 0.505 \text{ in}^2 / \text{ft}$$

$$M_u = A_s f_y \left(\frac{77.3}{12} \right)$$

$$= \frac{107.3}{117.1} \text{ K ft} / \text{ft}$$

$$M_u = \frac{96.6}{105.9} \text{ K ft} / \text{ft}$$

5) 29 to 5' 1"

$$M_u = \frac{107.3}{117.1} \text{ K ft} / \text{ft}$$

$$M_u = \frac{96.6}{105.9} \text{ K ft} / \text{ft}$$

Gravity Moments

$$M_u = \phi W_c \frac{d}{2} \quad \& M_u = \frac{M_u}{\phi}$$
$$= 0.9 W_c \frac{40}{12} \text{ k-ft.}$$

$$M_u = 3.0 W_c \text{ in kip feet}$$

Note d is like a Vent only

Below elev. 0

Very lightly Reinforced

Cracking Moment

$$M_u = \phi 7.5 \sqrt{f_c'} \frac{b d^3}{6}$$
$$= (0.7) \frac{7.5 (3000)^{1/2} 12 (40)^3}{(12,000) 6}$$

$$M_u = 306.7 \text{ k-ft / ft}$$

therefore assume zone above
joint controls - not below

$$\underline{\underline{307 > 245}}$$

* Non-Reinforced Conc.

$$\& M_u = \underline{\underline{438 \text{ k-ft}}}$$

Fig III - 1

hook at E-qui. Static Ult. Load

$$W_g = xg \cos x$$

$$V_g = xg \cos x$$

$$M_g = xg \cos x^2$$

~~$$M_g = 0.113 \times 10^4$$

$$xg = 0.113 \times 10^4$$

$$xg = 0.113 \times 10^4$$~~

~~$$\text{let } M_g = 1671.6$$~~

~~$$xg = 0.113 \times 10^4$$~~

~~$$\text{let } M_g = 0.059 \times 2 \quad \text{--- gap forms}$$~~

~~$$\text{let } M_g = 117$$~~

~~$$xg = 0.279g \text{ or } M_g = 0.147 \times 2$$~~

Gripping Moments & Resistance

Elev. 0

$$M = 185 \text{ k ft} / \text{ft}, \quad x = 16 \text{ ft}, \quad x = 33$$

$$\therefore 185 = \alpha g \frac{1.05 x^2}{2} \quad 1.05 \text{ k/ft} = w$$

$$\therefore \alpha g = \frac{0.125}{0.134}$$

Elev. 6' 4"

$$M = 163 \text{ k ft} / \text{ft}, \quad x = 46.67$$

$$163 = \alpha g \frac{1.05 x^2}{2}$$

$$\alpha g = 0.143$$

Elev. 12' 9"

$$M = 141 \text{ k ft}, \quad x = 40.25$$

$$141 = \alpha g \frac{1.05 x^2}{2}$$

$$\alpha g = 0.166$$

Elev. 29' 0"

$$M = 84, \quad x = 24.0$$

$$84 = \alpha g \frac{1.05 (x^2)}{2}$$

$$\alpha g = 0.278$$

For Gap at elev 0.

$$M = 0.125 \left(\frac{1.05}{2} \right) x^2$$

$$M = 0.066 x^2$$

Resistance for Gap + Rods @ ϕ_y

l.e.v. G

$$M_y = 450 \text{ K ft/Ct} \quad x = 53$$

$$450 = \alpha_g \frac{1.05}{2} x^2$$

$$\alpha_g = 0.292$$

$$\text{or } M = 0.153 x^2$$

check @ ~~l.e.v.~~ e(v. G. 33

$$M = 0.153 (53 - 0.33)^2$$

$$= 333 \text{ K ft} > 309$$

Resisting Moment.

$\therefore$ Yield may occur @ the
6' to 12' a' zone in the $1\frac{3}{4} \phi$ rods

$$309 = \alpha_g \frac{1.05}{2} (x^2)$$

$$\text{for } x = 46.67 \text{ ft}$$

$$\alpha_g = 0.270$$

$$\text{or } M_y = 0.142 x^2$$

} Static
Ult
Capac.

Look at Soils - footing etc

For 0.125g
at Gapping
toward pit

$$A_b = 17 \text{ ft}$$

$$S_b = \frac{17^3}{6} = 48.17$$

Note: at elev. 0
the wall is tied
to the slab with
#4 @ 12" - we
will neglect to be
conservative.

Soil Stress do to Rocking @ 0.125g

$$M_d = 6.95(27) + 55.65(3 + 3\frac{1}{2} + 3\frac{1}{3} - 8\frac{1}{2})$$

$$M_d = 261.66 \text{ kft} @ 0.125g$$

$$M_d = 15.03(27) + 55.65(1.33) + \frac{245 \text{ kft}}{2.78} = 278.6 \text{ kft} @ 0.270g$$

$$p = \frac{P}{A} + \frac{M}{S}$$

$$= \frac{55.65}{17} + \frac{261.66}{19.17}$$

$$= 3.27 + 5.43$$

$$= 8.71 \text{ k/sg ft cum}$$

$$\pm 2.16 \text{ k/sg ft cum}$$

Hall for Saline Superpoints

$$\left. \begin{array}{l} W = 55.65 \text{ k} \\ M = 261.66 \text{ ksf} \\ V = 6.95 \text{ k} \end{array} \right\} = 0.125 \text{ g}$$

@ 0.270 g

$$W = 56.55 \text{ k}$$

$$M = 878.6 \text{ ksf}$$

$$V = 15.03 \text{ k/ft}$$

$$M = 727.8 \text{ k/ft}$$

Develop Footing Loads / Stresses

Nol. k_{avg} = 30 pcf

We will increase by 14.9

Note Neglect lateral loads from -27 to -21 the large Footings etc inside pit & passive

$$M = \frac{13}{1000} (1 + \alpha g) \frac{30 h^2}{2} + 1.05 \alpha g h \left(\frac{h}{2} + 6 \right)$$

$$-2(2.205) - 7(6.43)$$

$$M = 86(1 + 2g) + 363.8(2g) - 92.6 \times 10^3$$

$$M = 49.62 \text{ kg} \text{ at } 0.125 \text{ g} \text{ \& } V = 10.20 \text{ K}$$

$$M = 114.05 \text{ kg} \text{ @ } 0.2709 \text{ dV} = 14.35 \text{ kg}$$

@ 0.125 g

$17 - 17, S = 48.17$

$W = 55.65$
 $M = 261.66$
 $V = 6.95$

+

$M = 49.62$
 $W = 43.16$
 $V = 10.20$

$p = \frac{\sum W}{17} \pm \frac{\sum M}{S}$

$p = \frac{55.65 + 43.16}{17} \pm \frac{261.66 + 49.62}{48.17}$

$p = 5.81 \pm 6.46$ - slight tension

$e = \frac{\sum M}{\sum W}$

$e = 3.15$

$X = 4.85$

$p_{max} = \frac{2 \sum W}{3X}$

$p_{max} = \frac{2(98.81)}{3(3.15)} = 13.58 \text{ at}$

Soil yield

$p_{s(yield)} = 13.2 \text{ ksf}$

Remains Elastic when Rocking

$$0.0270 \text{ g}$$

$$\left. \begin{array}{l} W = 56.55^* \\ M = 724.8 \\ V = 15.03 \end{array} \right\} 13^{10}$$

$$\left. \begin{array}{l} W = 43.16 \\ M = 114.85 \\ V = 19.35 \end{array} \right\} \text{pg 11}$$

$$e = \frac{\sum M}{\sum W}$$

$$e = 8.5$$

Unstable

Maximum Resistive Moment

$$p_u = 16.5 \text{ ksf.}$$

$$\therefore 2x = \frac{98.71}{16.5}$$

$$x = 2.99$$

$$e = 8.5 - x$$

$$e = 5.51$$

$$M_u = e \cdot 98.71$$

$$M_u = 544 \text{ K ft} \rightarrow \text{UH Resistive}$$

Moment-Soil

Neglecting Foundation Torsion

Foundation Tensional Strength.

$$\text{Min } T_n = \text{line item} \quad @ \text{ 1.9 pg 20 part 1}$$

@ Pg 5 part 2

Note $T_{nc} = 6188 \text{ k ft}$ may be neglected!

$$@ 1.5 \text{ ft}^2$$

$$6 \text{ ft}^2 \quad T_{nc} = 24,752 \text{ k ft}$$

$$+ \quad T_{ns} = 1793 \text{ k ft}$$

If we assume that the wall uniformly picks up tension as the soil enters the yield domain

$$\text{then } T_{nc} = 24,752 \text{ k ft / ft}$$

(60)
(53)

$$= 412.5 \text{ k ft / ft} \quad (467 \text{ k ft / ft})$$

$$\text{We need } 7248 + 114.85 - 544$$

p p Soil

Block 17 ft Soil Res.

Wall

$$T_n(\text{req'd}) = 295.7 \text{ k ft} < 412.5$$

We Must Conclude that the
Wall + Soil O.K. @ 0.270

Must Check Capacity at Wall Base

$$T_{nc} = \left(\frac{393}{180 + 3.13} \right) (295.7)$$

$$T_{nc} = 177 \text{ k ft} / 11 @ 4.3 \text{ ft}$$

$$270.5 @ 6 \text{ ft}$$

$$V = 55.65 \text{ kg}$$

$$@ 0.770 - 2 \text{ g}$$

$$V = 15.03 \text{ k}$$

$$M = 245 \text{ k ft max}$$

$$a = \frac{129,840 \text{ lb}}{12(0.85) \text{ ft}}$$

$$f_c' = 3000$$

$$a = 429 \text{ in}$$

Small

$$\sum M_o = 0$$

$$15.03(21) + 245 + 59.5(10.5) + 7(8.40)$$

$$- 52.14 \left(\frac{80}{12} \right) - 22.05 \left(\frac{40}{12} \right) = T_{nc} (\text{Req'd})$$

$$260.8 = T_{nc} (\text{Req'd})$$

@ O.K. @ 6 ft with steel

Check Footing Steel

From pg 13

$$M_u = 544 \text{ k ft / ft} - \text{Ult}$$

Soil Resisting Moment.

$$f'_c = 3000$$

$$f_y = 33,000$$

$$\rho_s = 2.52\% / \text{ft}$$

$$d = 68 \text{ in}$$

$$b = 12 \text{ in}$$

$$\alpha = \frac{\rho_s f_y}{0.85 f'_c b}$$

$$= \frac{2.52(33)}{0.85(3)(12)}$$

$$\alpha = 2.157$$

$\rho = 0.0025$
Very light
Reinforcement

$$M_u = \frac{1}{12} \rho_s f_y (d - \frac{\alpha}{2})$$

$$= \frac{2.52(33)}{12} (68 - \frac{2.157}{2})$$

$$M_u = 368 \text{ k ft / ft} > M_{u \text{ req'd}}$$

$$@ 0.125 g$$

$$\phi = 311 \text{ k ft / ft}$$

but less than Ult. Capability
of Soil of 544 k ft
which means only, part of
the Soil will get to Ult.

Hence we must use the
excess ultimate torsional capacity
from pg 14

$$\tau_{nc} = 467 \text{ ksi}$$

$$\text{Used by wall} = 276 \text{ ksi}$$

$$\text{Available } \tau_{nc} = 171 \text{ ksi}$$

$$\text{Available } M_n = 368 \text{ k-ft}$$

$$M_n (\text{Available}) = 539 \text{ k-ft}$$

$$\text{Req'd } M_n = 574 \text{ k-ft}$$

for 0.270 g equiv

like the One-Horse-Shay

When Rod @ zero yield ~~1 1/2~~ (2 1/2 ft)
a the (1 3/4) @ 6'4" the wall
and footing are at ultimate
in flexure, torsion & soil.

Fig. IV

Sept 21 1964

Calvin L. ...

Studying East Wall at Ft. ...

$$S_c = 500 \text{ psi}$$

$$f_y = 53,000$$

$$\gamma_c = 150$$

$$W_3 = 17.06 \text{ k/c}$$

$$V_3 = 2.18 \text{ k/c}$$

$$W_1 = 34.50 \text{ k/c}$$

Rocking can only occur above zero elevation as the blocks below grade are poured in place and keyed to prevent tipping.

The rods above elev. 0 will be loosened and at elev. 0 not loosened perhaps torqued.

Wall Capacity (1.51)

East Wall Capacity - above elev. 0

East Wall Capacity - Below elev. 0 ↓ -21
Inward Vault

Look at Soils - fig etc.

1st @ 0.230g at Gapping toward the pit.

$$V = 0.23 (W_2) = 7.00^k$$

$$A_b = 16 \text{ ft}^2$$

$$S_b = \frac{I_b}{6} = 42.67 \text{ ft}^3$$

$$V = 7.00^k$$

$$W = 30.45$$

$$\sum M_{L_c} = 7.0(27) + 30.45(3 + 3\frac{1}{2} + 3.33 \cdot 8.0)$$

$$M_L = \underline{244.72 \text{ kft/ft} @ 0.230g}$$

Soil pressure

$$p = \frac{P}{A} \pm \frac{M}{S}$$

$$= \frac{30.45}{16} \pm \frac{244.7}{42.67}$$

$$p = 1.90 \pm 5.74 \text{ ksf}$$

$$p = +7.64 \text{ Comp.} \left. \begin{array}{l} \\ \end{array} \right\} \text{Hold}$$

$$-3.84 \text{ ten}$$

Superposition.

From pg 11 sec. III

Loads do to footing below zero elev.

$$W = 43 \text{ k}$$

$$M = 111 \text{ k}^{\text{ft}} / \text{ft} \rightarrow @ 0.230 \text{ g}$$

$$V = 11.69 \text{ k}$$

above
-21 ft

$$\left(\begin{array}{l} \text{Note} \\ -1.5(22.05) \\ -6.5(6.73) \end{array} \right)$$

Soil Stress total

$$p = \frac{30.45 + 43.0}{16} \pm \frac{111.0 \text{ k}^{\text{ft}} + 244.7}{42.67}$$

$$p = 4.59 \pm 8.32$$

$$p = +12.91 \text{ comp}$$

$$-3.73 \text{ ten \& cant}$$

han

$$e = \frac{\Sigma M}{\Sigma W}$$

$$= \frac{355}{73.45}$$

$$= 4.83$$

$$x = 3.17$$

$$p_{max} = \frac{2 \Sigma W}{3x}$$

$$= \frac{2(73.45)}{3(3.17)}$$

$$p = 15.47 > p_{yield} = 13.2$$

∴ Some yield @
0.230 g.

Note, some soil deformation at rocking at 0.230 g. However the east wall will have an ultimate far in excess of the 0.27 g of the N & S walls. therefore little reason to compute at this time.

Appendix Y

200K

Behavior toward Soil

$V = 6.95 \text{ k/ft}$
 @ 0.125g
 @ Gapping
 statically

* $V = 15.03 \text{ k/ft}$
 @ 0.270g
Static Ult.

passive Soil
 use $p_s = 350 \text{ pcf}$

Reaction at Slab

$$f_c = \frac{15,030 \text{ \#}}{6 \times 12 \text{ in}^2}$$

$$\underline{f_c = 209 \text{ psi}} \quad \text{very low} \\ \therefore \text{O.K.}$$

Slab Sliding

$$V \geq 75(35) \cdot .35$$

$$V \geq 919 \text{ lbs/ft}$$

there for
would drag loads into
Footings of Building and
mod, lize passive soil
resistance

$$6.95 \text{ (Rocking)} \approx (0.125) \text{ (uH)}$$

$$15.03 \text{ (uH)}$$

$$0.279 = 0.289 \text{ ft}$$

Passive Resistance
vs
Depth

Displacement
vs

$$p_p = 0.35x \text{ in ksf. Depth}$$

Spring Constant for soil
on the order of 125 - 150 (Tcf)
or 20 to 25,000 lbs/ft²/in

In other words very little displacement
is required to develop significant
passive Resistance and nearly
uniform in nature

To avoid cracking find β_{pr} req'd, note the "g" load of wall is already assume & earthquake loads by passive resistance.

$$\sum M_{\phi} = 0$$

$$0 = 55.65(3.33) + 245 + 21(14) - 307 - \beta_{pr} \left(\frac{21^2}{2} \right)$$

$$\beta_{pr} = 1.89 \text{ k/ft} \leftarrow \text{very high}$$

Hence require considerable

Look at Rocking Force

$$\sum M_E = 0$$

$$0 = 55.65(3.33) + 5.95(21) + \sqrt{M_{cr}} + \frac{0.126(21)^2}{2} - \beta_{pr} \frac{(21)^2}{2} - 307$$

$$\underline{\underline{\beta_{pr} = 0.267}}$$

Hence no cracking will occur at Rocking and only 267 psf passive will be required $\therefore$ Elastic @ Rocking

Reconsider Ult. (.27 g)

As motions get larger the slab will transmit load to the sta footing and piers, i.e. $\frac{15 \text{ K}}{\text{ft}} \times 20 \text{ ft} \times \frac{1}{3} = 100 \text{ K/ft}$ at $D=2.0 \text{ ft} \rightarrow \frac{100,000}{\pi (14)^2} = f_v = 221 \text{ psi}$ this may shear of piers.

Assume edge foot + passive at wall could let $\beta_{ff} = 0.70 \text{ K/ft}$

find V for piers required

$$\Sigma M_e = 0 \quad , \quad \text{let } \beta_{pp} = 0.7$$

$$0 = 55.15(3.33) + 245 + (15-v)21 \\ - \frac{\beta_{pp}}{2}(21)^2 - 307^k + 0.239(21)^2$$

$$0 = 185 + 245 + (15-v)21 \\ - 165 - 307^k + 62.6$$

$$(15-v)21 = 20.56$$

$$v = 14^k / ft$$

~~$$\text{Q. } \beta_{pp} = 1.05^k / ft$$~~

for $v = 0$ $\beta_{pp} = ?$ & No Cracking

$$0 = 185 + 245 - 307^k + 63 - \frac{\beta_{pp}}{2}(21)^2$$

$\beta_{pp} = 849 \text{ psf}$ which means
if passive yield was at
849 psf. about 2.4 feet of
confined soil would yield
and little load would be trans-
mitted to piers. At
ultimate (yield of 2 1/4 red)
we will expect distortion in
the slab at the piers to
permit the distortions need
for passive pressure to
develop.

* Note: $M_{ucv} = \phi M_{cr} \text{ used} - \text{Conserv.}$