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Dalwyn R. Davidson
VICE PRESIDENT
SYSTEM ENGINEERING AND CONSTRUCTION

June 2, 1982

Mr. A. Schwencer, Chief
Licensing Branch No. 2
Division of Licensing
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Dear Mr. Schwencer:

Perry Nuclear Power Plant
Docket Nos. 50-440; 50-441
Structural Engineering Branch

The Structural Engineering Branch has identified several questions concerning the design of the Perry intake and discharge cooling water tunnels. These questions were based on a previous submittal dated March 11, 1982, on the same subject.

The answers to the Structural Engineering Branch's questions are attached.

Very truly yours,

Dalwyn R. Davidson

Dalwyn R. Davidson
Vice President
System Engineering and Construction

DRD:mlb

cc: Jay Silberg, Esq.
John Stefano
Max Gildner

B001

Responses to Additional Request for Tunnel Design

1. With respect to the table provided

   a. What is meant by the percent of reinforcing?
   b. More information on the Strawberry Tunnel is needed.

   With respect to the table of Tunnel Statistics, the column labeled "percent reinforcing" under the "final lining" category was calculated by dividing the length of tunnel which had a reinforced concrete lining by the total lined length.

   In the case of Strawberry Tunnel, 90 percent reinforcing was used for several reasons:

   1. The tunnel carries high pressure water (150 ft of water) and has relatively shallow cover. The steel controls the tensile forces caused by the internal pressure.

   2. The tunnel is relatively short. Much of the reinforcing was installed to control hillside movements at the portals. The portal areas are a high percentage of the total tunnel length.

2. How would the collapse of the lines affect the rate of flow in the tunnel. Discuss % reduction and ESW flow requirements.

General

The rate of cooling water flow available to the service water pumps is predicated upon:

- lake levels
- system requirements
- source of flow
- minimum allowable water level required for safe pump operation
- lining collapsing scenario

Significant Lake Levels

The significant lake water levels (the level of Lake Erie has been systematically measured since 1860) used for the design of the hydraulic structures serving the Perry Nuclear Power Plant are as follows:

| | Water Elevation | |
|---|---|---|
| | USGS | IGLD |
| Maximum monthly level of record | 575.4 | 573.5 |
| Average water level | 572.3 | 570.4 |
| Lower water datum | 570.5 | 568.6 |
| Minimum monthly level of record | 569.3 | 567.4 |

ESW System Requirements and Head Losses for Each Source of Flow

The intake flow requirements as well as the expected head losses from the regular intake structure and the alternate intake structure (discharge nozzle) to the on-shore ESW pumphouse are shown below for two units operating at minimum and maximum summer flows:

| Operating Condition | Emergency Shutdown flow (gpm) | Regular Intake Head Losses (ft) | Alternate Intake Head Losses (ft) |
|---|---|---|---|
| Maximum Summer | 48,000 | 0.48 | 1.86 |
| Minimum Winter | 16,000 | 0.05 | 0.21 |

Minimum Allowable ESW Pumphouse Level

The minimum operating water level in the ESW Pumphouse, at which the emergency service pumps can operate without cavitating, is 555.1 ft IGLD (557.0 ft USGS).

Collapsing Scenario

In the unlikely instance that the upper concrete liner of any of the tunnels together with a 2.5' thick layer of shale rock collapse into the tunnel's lower portion, the available flow cross-sectional area will be reduced from 78.5 sq. ft. to 61.7 sq. ft. The reduction of the cross-sectional area was calculated conservatively considering that:

a) The bulk volume of the concrete and shale rubble is 40 percent larger than the material in place.

b) The space that opens up at the tunnel crown after the collapse of the concrete line and adjacent shale rock is not available for water flow.

c) The collapse occurs along the entire horizontal length of the tunnels.

Steady-State Response ESW System

Since the roughness of the tunnel walls is increased due to the segment pieces lying in the invert and due to the lack of coherent lining in the crown, it will be conservatively assumed that the wall friction factor will, therefore, be subject to a four-fold increase and, consequently, the total ESW system head losses for the collapsed tunnels will become:

| Operating Condition | Emergency Shutdown flow (gpm) | Regular Intake Head Losses (ft) | Alternate Intake Head Losses (ft) |
|---|---|---|---|
| Maximum Summer | 48,000 | 3.09 | 11.98 |
| Minimum Winter | 16,000 | 0.32 | 1.35 |

The water levels at ESW Pumphouse after the postulated tunnel crown collapse will, therefore, be:

| Lake Condition | Water Elevation USGS | Regular Intake Summer | Regular Intake Winter | Alternate Intake Summer | Alternate Intake Winter |
|---|---|---|---|---|---|
| Maximum | 575.4 | 572.3 | 575.1 | 563.4 | 574.0 |
| Average | 572.3 | 572.2 | 575.0 | 563.3 | 573.9 |
| Lower | 570.5 | 567.4 | 570.2 | 558.5 | 569.1 |
| Minimum | 569.3 | 566.2 | 569.0 | 557.3 | 567.9 |

Conclusion

Since the minimum safe operating water level in the ESW Pumphouse is 557.0 ft. (USGS), it can be concluded that in the unlikely scenario of a log segment of concrete lining collapsing into the tunnel the resulting reduction of the water level and corresponding pump bell submergence at the ESW Pumphouse is adequate and, therefore, will not endanger the safe operation of the ESW pumps under any reactor emergency shutdown or lake level conditions.

3. Choose an unreinforced tunnel that may have undergone a seismic event. Compare the stress calculations of that tunnel with the Perry tunnel. Use a simplistic approach.

It is not practical to produce a one-to-one comparison design between the Perry Power Plant tunnels and another similar tunnel since a full finite element model analysis would be required of the other tunnel. The only way to directly compare the stresses is to use the same analysis procedure.

It is possible to compare the response of a similar tunnel as measured by damage observations. Dowding and Rozen (1978) prepared a catalog of damage observations of 71 rock tunnels that had been subjected to seismic ground motions. The 71 cases involve 13 different earthquakes whose Richter magnitude varied from 5.8 to 8.3. Six of the earthquakes occurred in California, six in Japan, and one in Alaska.

The reports of damage were separated into three main groups: shaking, fault movement, and portal failure. The last group contains cases predominantly related to landslides and the special boundary conditions at the portals. Neither the fault movement nor the portal failure categories are appropriate to consider for the Perry tunnel since there are no portals in the design or seismic faults intersecting the tunnel.

Three levels of response were delineated by Dowding and Rozen:

a. No damage implies post-shaking inspection revealed no apparent new cracking or falling of stones;

b. Minor damage due to shaking includes fall of stones and formation of new cracks; and

c. Damage includes major rock falls, severe cracking, and closure.

Of the 71 tunnels studied, two were as small as 6 ft in diameter. The majority were 10 to 20 ft in diameter, the size range of the Perry tunnels. Detailed geologic information was available for only 23 tunnels; 12 tunnels were in relatively competent rock, 11 tunnels were in sheared, weathered or broken rock, and 3 tunnels were located in soil-like material. For the 27 tunnels where information was available regarding the lining, 7 were lined with brick or masonry, 13 were concrete lined, 2 were timbered, and 2 were unlined.

The three levels of response were correlated with the calculated peak surface ground motion. There were no reports of even falling stones in unlined tunnels or cracking in lined tunnels up to 0.19 g. Up to 0.25 g there were only a few incidences of minor cracking in concrete lined tunnels. Between 0.25 g and 0.52 g there was only one partial collapse. It was associated with landsliding and was lined with masonry. The Safe Shutdown Earthquake ground motion at Perry power plant was specified to have a peak horizontal ground acceleration of 0.15 g.

The conclusions drawn by Dowding and Rozen from this historical data are:

a. Collapse of tunnels from shaking occurs only under extreme conditions. It was found that there was no damage in both lined and unlined tunnels at surface accelerations up to 19 g. In addition, very few cases of minor damage due to shaking were observed at surface acclerations up to 0.25 g. There were a few cases of minor damage, such as falling of loose stones, and cracking of brick or concrete linings for surface accelerations above 0.25 g and below 0.4 g. Most of the cases of similar damage appeared above 0.4 g. Up to surface acceleration levels of 0.5 g, no collapse (damage) was observed due to shaking alone.

b. Tunnels are much safer than above-ground structures for given intensity of shaking. While only minor damage to tunnels was observed in MM-VIII to IX levels, the damage in above-ground structures at the same intensities is considerable. Furthermore, it should be noted that the effect of the damage is a function of the use of the tunnel and building.

c. More severe but localized damage may be expected when the tunnel is crossed by a fault that displaces during an earthquake. The degree of damage is dependent on the fault displacement and on the conditions of both the lining and the rock.

d. Tunnels in poor soil or rock, which suffer from stability problems during excavation, are more susceptible to damage during earthquakes, especially where wooden lagging is not grouted after construction of the final liner.

e. Lined and fully grouted tunnels will only crack when subjected to peak ground motions associated with falling stones in unlined tunnels.

f. Tunnels deep in rock are safer than shallow tunnels.

g. Total collapse of a tunnel was found associated only with movement of an intersecting fault.

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Dowding, Charles H. and Rozen, Arnon, "Damage to Rock Tunnels From Earthquake Shaking", ASCE, Journal of the Geotechnical Engineering Division, GT2, Feb. 1978, pp. 175-191.

In view of the conclusions drawn by the Dowding and Rozen study, the Perry power plant tunnels should function well under seismic loads.

a. The Perry tunnels are located in competent massive rock.

b. The Perry tunnels are deep into the rock (80 to 90 ft of rock over the top of tunnel).

c. The Perry tunnels are not intersected by an active seismic fault.

d. The Perry tunnels are concrete lined and fully grouted.

e. The Perry tunnels have no portal entrances.

4. Reference question 4.c. What are the cumulative participation factors of the first six modes.

   Of the six modes analyzed for the seismic load cases, the responses were combined when the frequencies of the modes were within 10%. The closely spaced modes were summed and the square root of the sum was taken to reflect the total response of the system. The participation factor was therefore 1.0 on each of the individual mode responses prior to sumation.

5. Reference question 5. Compare SRP 3.7.3 for the Perry design. Does Perry comply or can it meet the SRP. (Seismic analysis in the longitudinal direction).

   With regard to SRP 3.7.3, the Perry power plant tunnel design complies with the document criteria.

6. Reference page 24 of previous response. What is the basis for the numbers used--especially 9000 feet.

The numbers used to study the response of the tunnel in the longitudinal direction fall out of relationships developed by Kuesel (1969) and the properties of the Chagrin shale. Figure 19, attached herein, shows the relationships between wavelength and amplitude for 1) loose sand and soft clay; and 2) dense sand and stiff clay. I have drawn in the assumed curve for Chagrin shale. The curve was assumed to have the same shape as the curves developed by Kuesel but the magnitude of the wave amplitude is decreased to reflect the stiffness of the shale.

Figure 20 the curve for Chagrin shale only. Basic arithmetic was required to calculate and plot the A/L curve for Chagrin shale. The maximum point on this A/L curve corresponds to 9000 ft wavelength and $19 \times 10^{-6}$ in./ft amplitude to wavelength ratio. These values were used in the analysis procedure developed by Kuesel.

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Kuesel, Thomas R., "Earthquake Design Criteria For Subways", ASCE, Journal of the Structural Division, ST6, June 1969, pp. 1213-1231.

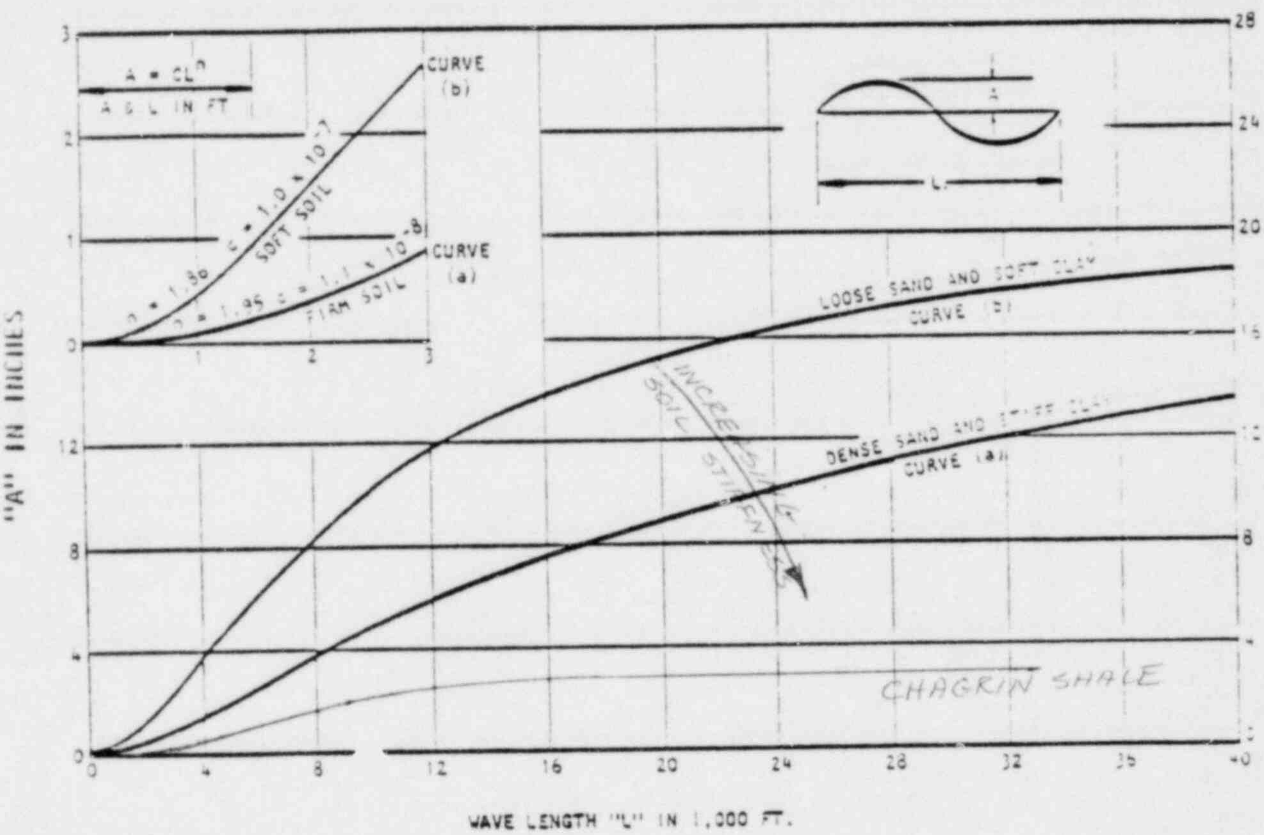


FIGURE 19. TRANSVERSE GROUND DISPLACEMENT SPECTRUM (FROM REF. 1)

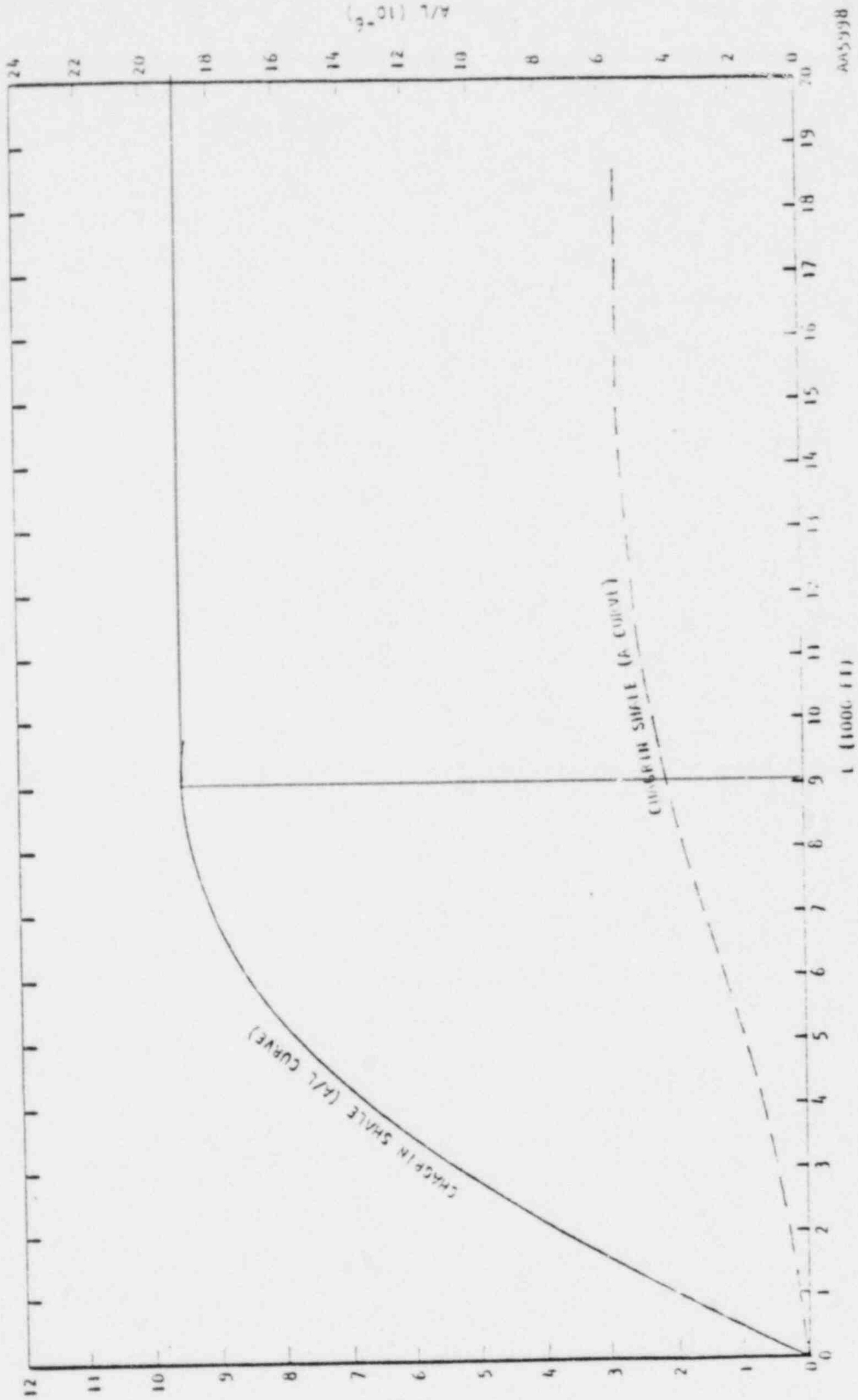


FIGURE 20. TRANSVERSE GROUND DISPLACEMENT SPECTRUM FOR CHAGRIN SHALE