Estimates of the Scope and Resources needed to perform Seismic Analyses of Facilities at the Western New York State Nuclear Service Center

June 22, 1979

by

Wayne A. Coffman

4405 Bel Pre Road

Rockville, MD 20853

975 337

7909200432

I SUMMARY

A series of analyses pertaining to facilities at the Western New York State Nuclear Service Center have been considered. Specifically, the tasks are in the two major areas of high level neutralized liquid waste storage tanks (HLW) and the fuel receiving and storage facility (FRS).

Available reports concerning previous analyses by LLL and LASL were reviewed to determine the scope and methodology of concluded seismic analyses for related facilities at this site. Some seventy drawings were reviewed of the design of these facilities to gain familiarity with the physical characteristics of the structures and their surrounding soil.

A narrative is presented in the subsequent sections of this report for each task. This includes a discussion of the approach which is expected to be most appropriate and productive. Cost elements including structural engineering effort, computer time and travel are estimated on a task basis.

A summary of the estimated resources required to perform the various tasks follows. The estimates tend not to contain conservatism or contingency for inefficiency due to delays, etc.

Problem	Method	Analysis Time (days)	Comp Time ($)	Travel ($)
Neutralized waste tank-vault concrete spalling	Hand calculations tank rupture toughness vs. particle shape, size, compressive strength	10	0	0
Neutralized waste tank piping	Modal analysis	25	500	113
Utility room equipment	Modal analysis	20	350	225
FRS rack	Time history analysis	40	3,500	0
FRS wall rework	1) base fix on hand calculations and study of LLL results (detailed) 2) reanalyze by best available method	25--30*	500	112
Totals		120 - 125 days	$4,850	$450

*Depends on availability of LLL analyses

975 339

II HIGH LEVEL WASTE TANKS

1. Concrete Spalling

LLL has suggested it is possible concrete chunks, presumably without any rebar, could break loose from the top of the vault during a seismic event and freefall and impact the top of the tank. They expressed concern in regard to the potential for rupture of the tank top.

The first activity is to estimate the size range of the concrete chunks.

A second consideration is to determine the probability the concrete chunks, perhaps in terms of various size classes, would have of reaching the top of the tank without being significantly impeded by the network of girders which acts as a superstructure on the top of the tank and which in turn is supported by the six columns.

For the particles which can reach the tank and give up their kinetic energy about three size classes should be considered. The large size would have more mass and thus more total kinetic energy but may well have a less sharp impact surface than a smaller concrete mass.

The drop distance is only 4.0 feet and therefore the specific kinetic energy of a concrete chunk is 4.0 (ft-lb/lb). A method drawing upon the Charpy and Izod impact toughness tests methodology will be used to help assess the integrity of the tank top in terms of its toughness and similarity of the impact with other articles

for which impact toughness data is available. Effort should be made to account for the possibility of concrete crumbling on impact vise tank penetration.

The freefall time is about 1/2 second and the impact velocity will be about 16.1 fps which is in the range between that for the Charpy (17.5 fps) and the Izod (11.5 fps) tests.

Material property data for the concrete and steel will be needed. It may turn out that the energy associated with a possible direct impact is sufficiently low so that penetration and rupture of the tank is entirely impossible. Alternatively, if the impact is in the range near where fracture is a feasible consequence, more effort will be required to refine material representations.

Hand calculations should be made to investigate those cases expected to be representative of limiting conditions. The effort required to perform this analysis is estimated to be ten days of a structural engineer's time. The task is not expected to involve any charges for computer time or travel.

2. Piping

The neutralized high level waste tanks have a network of underground piping which connects the two tanks together and to process equipment. The quantity of pipe is perhaps as much as 600 feet and much of it is of about 3.0 inch diameter. There is also other smaller instrumentation piping. In regard to waste containment, part of this piping may not need to be included in the analysis. A detailed review of the functional character of the instrument tubes should be made and ones with insignificant failure consequences excluded from the detailed seismic analysis procedure.

The LLL analysis (1) of the vault, tank and contents was done in a way which treated the dynamic effects of the soil surrounding the vault as though it were equivalent to a static pressure loading on the walls of the vault. The motivation for this was simplicity (and the authors stated that inclusion of soil dynamics was beyond the scope of their study). The justification was that the soil had a natural frequency (fundamental) which was much lower than that for the steel tank structure.

The static pressure loading method is likely to be much more applicable to the vault than to the pipe network because the stiffness of the pipe network is lower (because the lateral bending stiffness is relatively low for a pipe) than that for the vault. The consequence of this is that the pipe is expected to have several frequencies which are below the lowest vault frequency. Therefore, the pipe and soil are more likely to be dynamically interactive.

This suggests that the dynamics of the soil probably can't be treated as being equivalent to a static pressure load using the soil pressure theories of Mononobe and Okabe (2, 3).

For this analysis, it is suggested that the finite element model include:

1. pipe network in detail as 3D pipe elements
2. surroundingsoil as 3D solid elements or equivalent springs and mass elements
3. the vaults and their contents would be modeled as a small number of rigid masses, perhaps as few as three spring and mass pairs on each end of the pipe network for the liquid, the tank and the vault (for one particular degree of freedom).

Item 3 above would allow inclusion of the gross influences of the vaults/tanks upon this pipe integrity inquiry without attempting to analyze subregions of the vault/tank system. The soil surrounding the vaults would be treated either as having no dynamic effect (as LLL did) or as having a spring stiffness characteristic for its interaction with the surrounding soil. The model choice would depend on an independent estimate of frequency disparity (vault-soil).

The analysis should be made using the response spectra method and a linear model for all of the components (and materials).

If the modeling indicates the soil is highly nonlinear (uniaxial tension and compression) then a time history solution would be considered. It is not anticipated that this is likely to be needed.

In piping systems the mechanical loads can be very significant. For example, undesirable and high loads can be applied to force misaligned runs of pipe into alignment so connections or attachments to structural supports can be made. Also, settling of fill can cause a net downward load on the buried pipe. Other loads result from gravity, internal pressure and thermal cycles associated with material transfer through the pipes.

ANSI standard 31.7 applies to nuclear piping (for power plants) and can be used as a guide to suggest many elements of good practice in this analysis as to considerations such as connecting technique, inspection, load combinations and their stress limits.

It is estimated the seismic modal analysis can be accomplished in 25 days. A site visit is not definitely required. An initial effort should include sketching and isometric of the pipe network (with dimensions) and explicit definition of pipe supports (location, type) and whether the pipes are in an open channel or one back filled with soil.

Total estimated resources are:

1. structural analysis 25 days
2. computer time $500
3. travel $113
   (.5 probability of trip to West Valley)

975 344

3. Equipment

Auxiliary power generating plant equipment including diesel engine, generator controls and some power conditioning equipment as well as other support systems are located in the utility room. This equipment is intended to provide power in the case of commercial power outage for priority functions such as ventilation.

The analysis of this equipment in regard to a seismic event will be rather straightforward. The necessary activities include the development of a relatively simple but representative mass and stiffness model of the components and their method of attachment to the floor of the utility building. The massive concrete pedestals would also be included in the dynamic model. The engine and generator would typically have 'shock-mounted' links in their load paths to ground. These segments have low stiffness and serve to attenuate the high frequency vibrations of the stiff machine parts which result from imbalance of the high speed rotating machine.

The mass-stiffness model can likely be accomplished with the use of some 40 masses and 100 springs. Hand calculations would be used to determine 'equivalent' spring constants and mass values. Also, in various regions heavy walled pipe elements may be used to represent generator rotor, housing and perhaps the engine as well. These elements sometimes allow a more straightforward representation of the spatially distributed mass and stiffness of a component such as a generator and they can be easier to use. Control boxes and batteries can be modeled as point masses.

975 345

A critical aspect of this control room equipment modeling is the nature and strength of the attachments. Details in regard to the equipment is not shown on the utility room drawings. A visit to the plant will be necessary in conjunction with development of the model for this task. The equipment foundations appear to be independent of that for the building. The building analysis is not included herein.

Nonlinearities are not expected to be important in magnitude or number. The modal analysis method would be used to obtain the response to the seismic event per the Regulatory Guide 1.60 spectoral shape.

It is estimated that the resources needed for making this analysis are as follows:

1.	structural	20 days
2.	computer time	$350
3.	trip to West Valley	$225

075 346

III FUEL RECEIVING STATION

1. Fuel Rack

The fuel rack and canister system involves nonlinearities due to the sliding of the canister in the rack (slip-stick motion under the applied force system). Since the canisters are not absolutely contrained in the N-S direction against sliding, a significant departure from linear elastic behavior is possible. The modal analysis method is dependent upon the existence of a completely mathematically linear (physically elastic) structure. The force and motion system associated with slip-stick behavior under the predominance of friction (no slip condition) is linear. There is no motion until a threshold force is achieved and then motion tends to continue without the development of increased resistance.

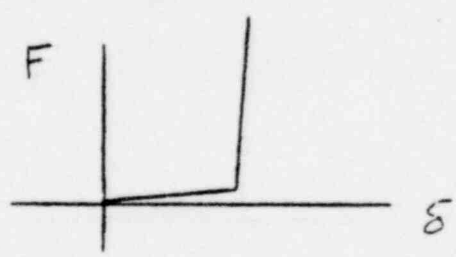
The modal method could be applied on a limit analysis or try-and-see basis. This would involve:

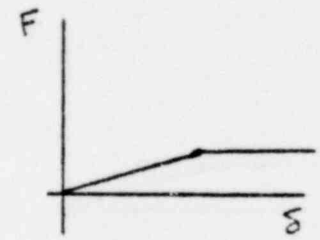
1. Assume the canisters are each bolted to the rack and do not move. The modal analysis would give the load on the bolt.
2. Assume the canisters have negligible resistance to sliding. The individual canisters can be assumed to be connected to the rack by weak springs in the N-S direction.

The submerged weight of the canister and its contents would be taken into account to assess the weight which when multiplied by a friction coefficient for wet surfaces would yield a sliding force. This would be done by a hand calculation. If the force in the bolt of case 1 above is greater than this sliding force then analysis

1 is an upper limit in regard to tie down load, canisters will slide but we haven't determined how many will slide out. The second analysis would be used to provide information on the maximum compression of canisters in the N-S direction. The first analysis would also give information on the low frequency interaction in the E-W direction in terms of maximum deflection and rack strength.

The straightforward method of handling the fuel rack problem is to do a transient analysis with a time history forcing function. It is anticipated that a single rack with 26 canisters would be modeled. Finite element models are generally not comprehensive of the physical constraint that two objects can't occupy the same space. Nonlinear interface (gap) elements are used to simulate the E-W clearance between racks.





These same general element types are used to simulate the canister's tendency to stick to the rack until a threshold sliding force is achieved. A very weak spring constant is used to establish a unique displacement at any given time vise constant force motion (sliding).

This transient analysis would be comprehensive of:

1. rack tie down stresses
2. canisters sliding out of rack (south end)
3. canister interaction in N-S direction

4. interaction of rack with artificial fixed walls in E-W direction to simulate interaction of adjacent racks
5. strength of rack due to E-W motion
6. overall rack strength

Deflections (E-W) would be checked to see if they tend to open up enough space to allow a canister to 'drop through'.

The transient analysis is more likely to produce meaningful results. The modal analysis would be both <u>incomplete</u> (the extent of fuel sliding out would not be known) and pose a risk in regard to providing sufficient information to the analyst so that conclusive results and effects can be stated.

The transient analysis would require 40 days of structural analysis, about $3,500 of computing time and no travel.

349

2. FRS Wall Damage

Dong and Ma (UCRL-52575 5/5/78) say the results of their detailed analysis indicate that walls F/N and C/E are most severely stressed (ref. 4). Also interior walls F/C, W/F and W/C were highly stressed. They discredit the applicability of their thermal stresses calculated for a 35° F. ΔT and elastic concrete on the basis of the ΔT not applying in the lower part of the pool, stress relaxation opportunity during life and cracks providing stress relief. The authors didn't report any stress magnitude data for the load cases such as thermal, seismic, water and soil pressures.

Without the thermal stress the wall F/N, which is the north wall of the pool, exceeds the allowable criteria in terms of wall moment at 0.16g seismic (zero period) spectrum excitation. The most severe location is at the top near the intersection with the W/F wall which partitions the pool and the water treatment cell.

Appreciation of the analysis is impeded by the unavailability of responses to the independent load cases and by not reporting any stress values at all.

If the results were in terms of stress, results could readily be thought of in terms of material strength. This is lost when only 12 values of moment are reported.

If it is accepted that the task is to bring the M_{ult}/M to a minimum value of unity in wall F/N for every element then:

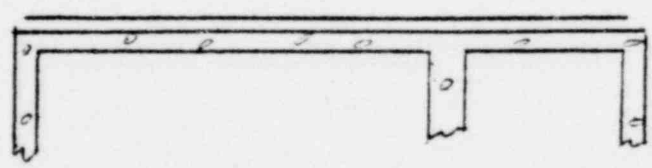
1. structural change need be made
2. evaluation of the stress must be made to determine whether the improvement is adequate.

975 350

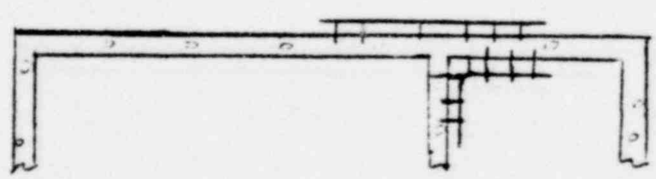
For a prismatic and isentropic beam carrying a given moment the bending stress is inversely proportional to the square of the thickness. Also, the bending moment capability increases as the square of the thickness. Thus an increase of thickness of 10% would decrease the stress to 82.6%. Similarly, if the concrete/steel wall had its bending rigidity increased by 13 or more percent the M_{ult}/M should approach unity.

It is not clear from looking at the drawings what method of reinforcement would be most appropriate. It will be assumed that one of the following is possible:

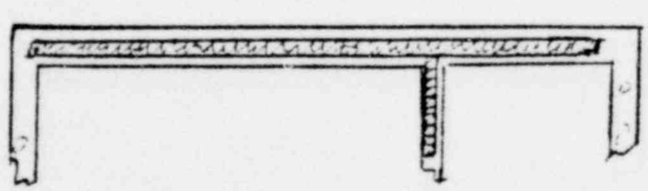
1. wire mesh reinforcement across the entire N face (outside)



2. steel plates epoxy bonded with dowels into the wall



3. reinforcement of the top of the walls with steel plate caps (epoxy dowel bonded to concrete)



Constraints may be identified during the course of the rework which would eliminate any of the above. This list is only suggestive and not complete.

975 351

In order to proceed with a solution to this task it is necessary to:

1. obtain LLL results in regard to load case, location of stresses, forces or
2. develop an independent model and repeat their task.

Making a new load path sometimes constrains the existing structure so that the reactions are significantly redistributed and the original structure may still be overstressed. Therefore some analysis should be done to evaluate load path readjustment.

If 1 is possible the fix should be structurally analyzed in [illegible] days provided decks and documentation is forthcoming.

The LLL SAP 4 model decks should be reviewed and LLL should be asked for their detailed work in regard to determining the resulting response (figure 24, p. 36, ref. 4).

If 2 is necessary the cost will increase by about 60 to 70 percent of LLL pool analysis cost (excluding the cask drop portion). There is at least a 0.5 probability a site visit will be necessary with a travel cost of $112. The computing time cost is estimated to be $500.

References

1. "Seismic Analysis of the Acid Liquid Waste Tanks at the Western New York State Nuclear Service Center," UCRL-52600, LLL, March 1979.
2. N. Mononobe and H. Matsuo, "On the Determination of Earth Pressures During Earthquakes," Proceedings of World Engineering Conference 9, p. 176, 1929.
3. S. Okabe, "General Theory of Earth Pressure," Journal of Japanese Society of Civil Engineering, vol. 12, no. 1, 1926.
4. R. G. Dong, S. M. Ma, "Structural Analysis of the Fuel Receiving Station Pool at the Nuclear Fuel Services Reprocessing Plant, West Valley New York," UCRL-52575, LLL, May 1978.