UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

Before the Atomic Safety and Licensing Board

In the Matter of	)	
Philadelphia Electric Company	)	Docket Nos. 50-352 50-353
(Limerick Generating Station, Units 1 and 2)	)	

TESTIMONY OF PHILADELPHIA ELECTRIC COMPANY REGARDING THE ABILITY OF SAFETY RELATED STRUCTURES TO WITHSTAND THE EFFECTS OF POSTULATED DETONATION RESULTING FROM THE ASSUMED RUPTURES OF THE ARCO AND COLUMBIA GAS TRANSMISSION PIPELINES

INTRODUCTION

1. On January 9, 1984, the Atomic Safety and Licensing Board ("Licensing Board") requested additional testimony from the parties regarding the ability of safety related structures at the Limerick Generating Station to withstand the effects of postulated detonations resulting from the assumed rupture of the ARCO and Columbia Gas Transmission pipelines. The Licensing Board expressed an interest both in the ability of the safety related structures to withstand such postulated blasts and the margins above such values inherent in building design. (TR 5934-44). This testimony is responsive to that request and includes the following:

   - A discussion of the various terms related to the analysis such that they can be understood and used consistently throughout.

- A description and results of the analysis regarding the ability of safety related structures to withstand pressures determined for the postulated accidents previously analyzed in testimony before the Licensing Board related to contentions V-3a and V-3b.

- A discussion of the TNT explosion on the Reading Railroad described in the Final Safety Analysis Report as indicative of the pressures to which certain safety related structures have been designed.

- A discussion of the margins above the calculated pressures for which the integrity of safety related structures can be assured.

- A discussion of the analysis used to demonstrate that a failure of the cooling towers resulting from a pipeline explosion would not affect safety related structures, components, or systems.

DEFINITIONS OF TERMS

2. Because there has been confusion regarding the terms associated with pressures resulting from detonations, the following are the definitions utilized in this testimony:

Incident Pressure is the sudden rise in pressure due to the violent release of energy from a detonation. The peak positive incident pressure (Pso) is the maximum incident pressure above the ambient pressure.

Reflected Pressure is the total pressure which results instantaneously at a surface when a shock wave travelling in one medium strikes another medium, e.g., the ground.

Peak Positive Reflected Pressure (Pr) is the maximum reflected pressure developed above the ambient pressure.

PHYSICAL DESCRIPTION OF EXPLOSIVE PHONEMENA

3. In the design of structures to resist the effects of accidental explosions, the effect to be considered is the resulting pressure. This pressure is in the form of a shock wave composed of a high-pressure shock front which expands outward from the center of the detonation with intensity of the pressure decaying with distance. As the wave front impinges on a structure, a portion of the structure or the structure, as a whole, will experience a structural loading as a result of the shock pressure. For the purpose of this report the terms explosion, blast and detonation are used interchangeably.

TYPES OF BLAST ENVIRONMENT

4. The types of detonation loading on the plant facilities can be identified as air-burst loads and surface-burst loads. The air-burst blast environment is produced by a detonation which occurs above the ground surface and at some distance away from the structure so that the initial shock wave, propagating away from the explosion, impinges on the ground surface prior to arrival at the structure. As the blast wave continues to propagate outward, a front known as the Mach Front (Figure 1) is formed by the interaction of the incident wave and the reflected wave which is the result of the reinforcement of the incident wave by the ground.

5. The surface-burst blast environment is produced by a detonation which occurs at or very near the ground surface. The initial wave of the explosion is reflected and reinforced by the ground surface to produce a reflected wave. The reflected wave merges with the incident wave at the point of detonation to form a single wave, which is essentially hemispherical in shape (Figure 2). For a given distance and TNT equivalent (Zg) a ground blast would give a greater pressure on the walls of a structure than an air burst blast. In this analysis of the ARCO and Columbia Gas pipeline

explosions, a ground blast was assumed uniform over the wall height when the walls of structures were being analyzed and an air blast was assumed for analysis of the roofs.

BASIS OF ANALYSIS

6. This analysis of the structural integrity of safety related structures of the Limerick Generating Station utilizes as a basis the methodology discussed in the testimony of John Walsh. (Testimony of John D. Walsh relating to contentions V-3a and V-3b) (Tr. ff 5411). That testimony developed the maximum pressure that would be developed at any of the safety related structures for the Station. Such maximum pressure would result from a rupture at the closest approach of the of the Columbia Gas Transmission pipeline to such structures, i.e., approximately 3500 feet, leading to a postulated detonation approximately 1200 feet from the structure. However, to analyze the effect on all safety related structures, it must be recognized that the detonation could be assumed to occur at locations farther away than that assumed to give the maximum pressure, but which could produce more limiting pressure for particular structures, e.g., spray pond pump house. Therefore, utilizing the same methodology as used in the Walsh

Testimony, the Columbia Gas Transmission pipeline explosion was assumed to occur along a line parallel to and 700 meters (approximately 2300 feet) from the pipeline (see Figure 3). Utilizing the distances from this line to safety related structures, the peak positive reflected pressure for each of the particular structures was determined.

7. It was not necessary to calculate the pressures resulting from the assumed rupture and detonation of gasoline from the ARCO pipeline inasmuch as the resulting pressure, assuming an explosion centroid along the Possum Hollow streambed, as did Walsh, is always significantly less than that resulting from the assumed detonation of the vapor from the Columbia Gas transmission pipeline as developed above. As calculated by Walsh, the maximum peak positive reflected pressure from an ARCO pipeline explosion is 1.9 psi.

8. For purpose of analysis two calculations were made for each safety related structure. The first calculation, done for the purpose of calculating roof loads, varied the assumed elevation of the detonation from roof height to 500 feet above the ground in order to find the maximum roof load. A second calculation was done for the purpose of calculating wall loads. It assumed a surface-burst blast to determine the peak positive

reflected pressure to be utilized for the analysis. This pressure was then resolved and applied normally to the wall in order to assess the structural capability of the critical portion of the wall.

9. As discussed above, various points along the line of the possible explosion, as indicated in Figure 3, were examined to determine the pressure applied normally on safety related structures. The pressures have negligible effects on safety related buried pipes, manholes and ductbanks. The analysis of building wall response to the calculated peak positive reflected pressure was divided into two portions. Initially, local response of each structural element was examined. By examining the structural drawings of each wall face, the critical element of that structural face could be determined based upon the peak positive reflected pressure as determined for such wall. Once that determination was made, the critical element was examined as if it were a beam element with appropriate end conditions representative of those for such element in the structure. Physical properties of the structures determined from design values such as location and amount of reinforcing steel and the minimum specified 28-day design concrete strength were used. Using the methodology of Reference 1, pages 6-1 through 6-13 and

6-21 through 6.23, shear and bending capacities were calculated for the critical locations and compared to the acceptance criteria presented in Reference 1 at page 6-48.

10. For the reactor enclosure and diesel generator building, inasmuch as the Reading Railroad accident analysis discussed below had already been performed, the normal pressures on critical locations from this event were compared to the assumed maximum value of 10 psi from the Columbia Gas Transmission pipeline explosion as calculated by Walsh. For these structures, the Reading Railroad explosion was found to bound the Columbia Gas Transmission pipeline explosion for the structure walls. Inasmuch as the maximum pressure calculated by Walsh of 10 psi was not normal to the structure walls, this is conservative.

11. The second part of the analysis involved the global response of each structure. The loadings on the entire structure, i.e., story shear and overturning moment, were calculated and compared to the loadings resulting from the Safe Shutdown Earthquake (SSE). For each structure, the loading resulting from the SSE was found to be controlling.

RESPONSES OF STRUCTURES OF THE READING RAILROAD BLAST

12. One of the events which had previously been analyzed with regard to design of the Limerick Generating Station was the pressures resulting from the hypothetical explosion of TNT assumed to be carried on the Reading Railroad. The analysis considered a surface detonation and examined the effects on safety related structures of the facility. The structural analysis utilized the same methodology as described in the previous section relating to the analysis of the Columbia Gas Transmission pipeline explosion. The relationship of the Reading Railroad to the facility resulted in the maximum loads being applied to the walls of the structures and not to the roofs.

SUMMARY OF RESULTS OF ANALYSIS

13. Table 1, Summary of Accidental Explosion Pressures, presents the controlling peak positive reflected pressures for each safety related structure. Because of the postulated location and magnitude of the various explosions, i.e., the track of the railroad versus the locus of the centroid of the assumed Columbia Gas Transmission pipeline explosion, the controlling accident is dependent upon the magnitude of the blast, the distance to the structure and their orientation. Inasmuch as the detonation resulting from the rupture of

the Columbia Gas pipeline was assumed, in the alternate scenario, to be elevated, the roof pressure was determined to be critical for this accident. One item should be noted. The peak calculated pressure resulting from the railroad car explosion is listed in Table 1 as 16.1 psi for the reactor building. This is the pressure experienced by the critical element of the wall rather than the average wall pressure which is approximately 12 psi.

MARGINS OF STRUCTURAL CAPABILITY

14. In order to respond to the Licensing Board's questions with regard to margin of structural capabilities of the safety related buildings, the maximum pressure that each structure could experience without exceeding the acceptance criteria in Reference 1 page 6-48 was calculated. For this analysis, in order to provide a realistic assessment of margin, the actual strength of the concrete as determined from field measurements at 28 days was utilized, rather than the minimum specified 28 day design value. This analysis did not utilize the additional strength of concrete which is a result of strength gain resulting from the years of additional aging since the concrete was 28 days old. This unaccounted for increase in strength is at least 20 percent above the value utilized in the evaluation of

margin and thus represents an additional conservatism. The technique used to analyze the structural elements was also conservative in that side support of the critical elements considered was neglected and redistribution of the imposed load was not assumed.

15. Even at the values contained in Table 1 for which the acceptance criteria of Reference 1 were just met, incipient failure of the structure is not implied. There is additional margin to failure as a result of additional plastic deformation which would take place without failure. With regard to shear, the acceptance values utilized also have certain inherent margins.

16. The margins of the global building response to the assumed detonation were also examined against the loadings resulting from the Safe Shutdown Earthquake in order to quantify the margin inherent in the global response of the structure. It should be noted that there is additional margin in the safety related structures with respect to their ability to withstand the Safe Shutdown Earthquake above the values for which they may have been analyzed. The overturning moment and story shear due to the assumed detonation were developed for each structure. The total force against each critical wall, as determined by the various pressures applied to it, was utilized in this evalution. The

worst case for each structure e.g., the Reading Railroad or Columbia Gas Transmission pipeline accident, as appropriate, was utilized in determining the margin which was present. As may be seen from Table 1, margin exists at each location with regard to global building response.

COOLING TOWER ANALYSIS

17. Since the cooling towers are not in and of themselves safety-related structures, they are treated differently in that they are conservatively assumed to fail given the occurance of a pipeline explosion resulting from a postulated rupture of the Columbia Gas Transmission line as discussed in the Walsh testimony. Thus, the discussion of the effect of the failure is limited to the impact of the hypothetical failure upon safety related structures, systems and components. Figures 4, 5, 6 and 7 show the dimensions of the towers and their locations relative to other structures and components at the Limerick Generating Station. Based upon observations of previous cooling tower failures, model tests and a comparison of the design of the Limerick cooling tower cooling towers to those which have experienced failures, the failure mode of the tower is expected to be by buckling. This failure mode results

in the debris falling predominantly within the tower base area (372' tower base diameter), with a small amount falling on outside areas away from the tower. As a limit, all such pieces of concrete would be expected to fall within a target area with a radius equal to one tower base diameter measured from the center of the tower. This is based upon failures evident in Ferrybridge, Britian (Reference 2); Ardeer, Scotland (Reference 3) and the Grand Gulf plant at Gibson, Mississippi. Model tests by Der and Fidler (Reference 4) also substantiate the inward bending and buckling of the shell.

18. For analysis purposes it was conservatively postulated that the cooling tower failure would produce a piece of concrete about 5' x 5' x 1' thick which would fall within a target area with a radius equal to one tower base diameter from the center of the tower (Reference 4, 6 and 7). The striking velocity of the piece of concrete at the ground is conservatively assumed to be 200 feet per second. This compares conservatively with the velocity of 188 feet per second for a free fall of approximately 550 feet from the top of the tower to grade at El. 217 feet. The worst orientation, i.e., a corner of the piece hitting the ground, was assumed.

19. The size of the piece of concrete was selected because it is conservatively larger than pieces which might be generated as a result of consideration of the design of the structure including the size of the shell and its reinforcement. The analysis also considers the estimated buckling shape and wave length of the tower shell (Reference 4).

20. The assumed concrete piece is calculated to penetrate the soil approximately 2.8 feet using the same methodology as for penetration of tornado missiles. As shown on Figures 8 and 9, the minimum soil cover or equivalent protection for the seismic category I buried pipes and duct banks is 4 feet. Hence, the assumed cooling tower concrete piece is known not to affect these buried structures. The analysis further shows that the impact of the piece of concrete would not overstress the buried pipe or the concrete duct banks due to soil compression. Other category I items requiring protection from the assumed tower piece of concrete were examined. These include manholes for the duct banks. The top of these manholes are adequately protected from such missiles by steel and concrete covers. Other indirect failure modes, as a result of the failure of the cooling tower basin, have also been examined.

21. The cooling tower cold water basin walls have been designed as non-seismic Category I structures. They may fail under a safe shutdown earthquake tornado or blast. The water from the tower basin could flow through a possible breach in the damaged basin walls and flood the surrounding area.

22. The runoff pattern of the water would be similar to that established for the intense storm precipitation (Figures 10 & 11). Most of the flood water from the cooling tower basin would run away from the power plant complex. The worst-case flood conditions for the power plant complex would be created by a failure of the south side of the Unit 1 cooling tower basin wall. For this case, a portion of the cooling tower basin water would flow towards the turbine enclosure. Although some limited turbine enclosure flooding may occur, there would be no impact on safety related components. This scenerio was discussed in response to NRC Question 410.5 which is attached hereto and incorporated by reference.

23. While the differences in elevations between the cooling tower basins and the grade outside the power block buildings is approximately 41 ft, (Figure 6) the hydrostatic head on the seismic Category I manholes and duct banks would be relatively small compared to this difference in elevations based on the runoff pattern.

The access openings at the top of the manholes are protected from runoffs with tight-filling steel covers bolted to the adjacent concrete slabs. Water penetration would be minimal.

24. All electrical cables in the duct banks (Figure 12) have been designed to function under water. In addition, all electrical conduits that travel to electrical manholes outside the structures are sealed watertight to prevent water from entering the structures through the electrical duct banks. This has been addressed previously in Section 3.4.1 of FSAR and responses to NRC Questions 410.2 and 410.6 which are attached hereto and incorporated by reference herein.

25. Most of the seismic Category I piping is supported on rock where erosion from short time water flooding would be insignificant. To the northwest of the Unit 1 cooling tower portions of the seismic Category I buried pipes are supported on Type I granular fill. However, most of the soil cover over this location is more than 10 ft., with a small portion having about 5 ft. of cover. Since the water would run off rapidly on the ground surface, it would take the least resistant flow path with very little penetration into the ground to cause erosion of the pipe bedding (Figure 13). Some soil cover would be washed away, but it would take time

to expose the pipes completely. The water flow for a large breach in the basin wall would last approximately 30 minutes. Furthermore, the adjacent seismic Category I piping could span more than 30 feet with no supporting material underneath and still carry the weight of pipe and contents without loss of function. A considerable time (much longer than 30 minutes) would be required to cause a large erosion of this size to undermine the supporting capability of the pipe bedding. The result of this phenomenon is similar to, but less severe than, the failure of non-seismic Category I buried pipes as addressed in response to NRC Question 410.47 which is attached hereto and incorporated by reference herein. Hence, it can be concluded that undermining of seismic Category I buried piping would not be a concern.

26. Based on the above discussions it is concluded that the seismic Category I buildings, buried pipes, duct banks and manholes are suitably located and adequately protected against a conservatively postulated cooling tower failure resulting in missiles and water flooding. They will perform their design functions safely without adverse consequences due to such an incident.

CONCLUSION

27. The foregoing presentation demonstrates both quantitatively and qualitatively that margin exists for loading of the safety related structures due to the blast resulting from the controlling event. Furthermore, the failure of the cooling tower would not prevent the safety related structures systems and components from performing their design functions.

REFERENCES

(1) "Structures to Resist the Effects of Accidental Explosions" Dept. of the Army. The Navy, and the Air Force (Manual No. TMS-1300/NAVFAC P-397/AFM 88-22) June 1969.

(2) "High Wind Levels British Cooling Towers", Engineering News - Record, November 25, 1965 page 45.

(3) "Report of the Committee of Inquiry into the Collapse of the Cooling Tower at Ardeer Nylon Works, Ayrshire on Thursday, 27th September 1973", Imperial Chemical Industries Limited, Petrochemicals Division.

(4) K. P. Buchert, "Buckling of Shell & Shell-Like Structures", K. P. Buchert & Associates, 1973.

(5) T. J. Der and R. Fidler, "A Model Study of Buckling Behavior of Hyperbolic Shells" Proc. Institution of Civil Engineers, Vol. 41, January 1968.

(6) K. P. Buchert, "Preliminary Stability Analysis of Reinforced Concrete Cooling Towers", IASS, Calgary, Canada, July 1972.

(7) K. P. Buchert, "Stress and Buckling Analysis of Cracked Reinforced Concrete Shells Using Split Rigidity Concept", IASS Symposium, Darmstadt, 1978.

LIMERIC PROJECT
JOB 8031

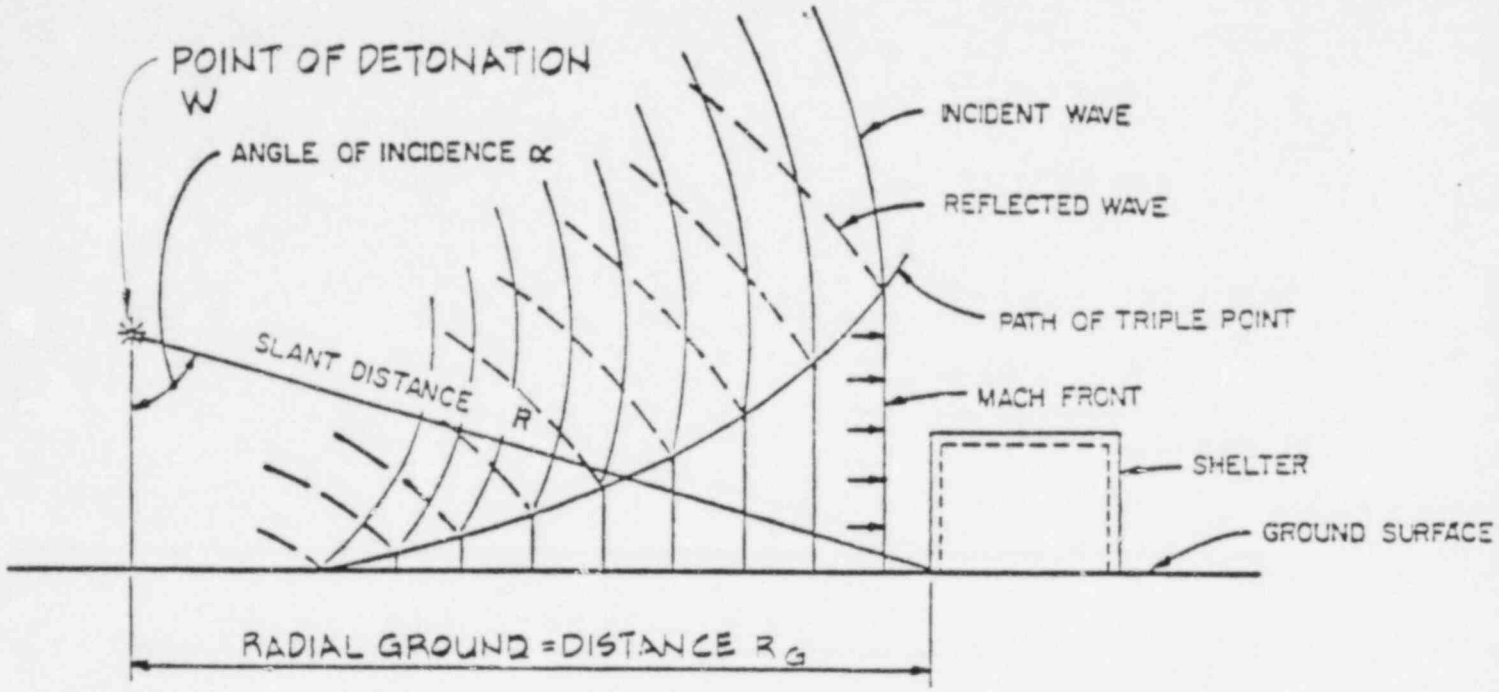
TABLE I

SUMMARY OF ACCIDENTAL EXPLOSION PRESSURES

LOADING ON STRUCTURE / BUILDING FACILITIES	DESIGN/ASSESSMENT VALUES: POSITIVE PEAK REFLECTED PRESSURE-PSIG: COLUMBIA PIPELINE NATURAL GAS EXPLOSION		ARCO PIPELINE GASOLINE EXPLOSION		READING RAILROAD BOX/TANKCAR EXPLOSION		MARGINS (%) OVER DESIGN/ASSESSMENT PRESSURE FOR EXPLOSION		N-S DIRECTION COMPARISON OF GLOBAL BLDG. RESPONSE: EXPLOSION PRESSURES		SAFE SHUTDOWN EARTHQUAKE		REMARKS
	ROOF	EXT. WALL	ROOF	EXT. WALL	ROOF	EXT. WALL	ROOF (BENDING / SHEAR)	EXT. WALL (BENDING / SHEAR)	OVER-TURNING MOMENT	STORY SHEAR	OVER-TURNING MOMENT	STORY SHEAR	
									FT-K	K	FT-K	K	
REACTOR BLDG. UNIT 1	NC (1)	NC	NC	NC	5.3	16.1	NC	19 / 15	1.37×10^7	88,630	1.51×10^7	120,440	
REACTOR BLDG. UNIT 2	5.4	10.0	1.9	1.9	NC	NC	35 / 3.3	NC					
DIESEL GEN. BLDG. UNIT 1	NC	NC	NC	NC	5.7	16.4	NC	14 / 28	1.55×10^5	8390	4.65×10^5	9,060	
DIESEL GEN. BLDG. UNIT 2	6.7	10.0	1.9	1.9	NC	NC	84 / 20	NC	1.55×10^5	8390	4.65×10^5	9,060	
CONTROL BLDG.	4.9	10.0	<1.9	<1.9	3.3	10.0	83 / 14	15 / 20	NA	NA	NA	NA	
SPRAY POND PUMPHOUSE	3.0	5.0	<1.0	<1.0	2.1	4.7	143 / 9	900 / 104	1.92×10^4	2,025	14.8×10^4	4,242	

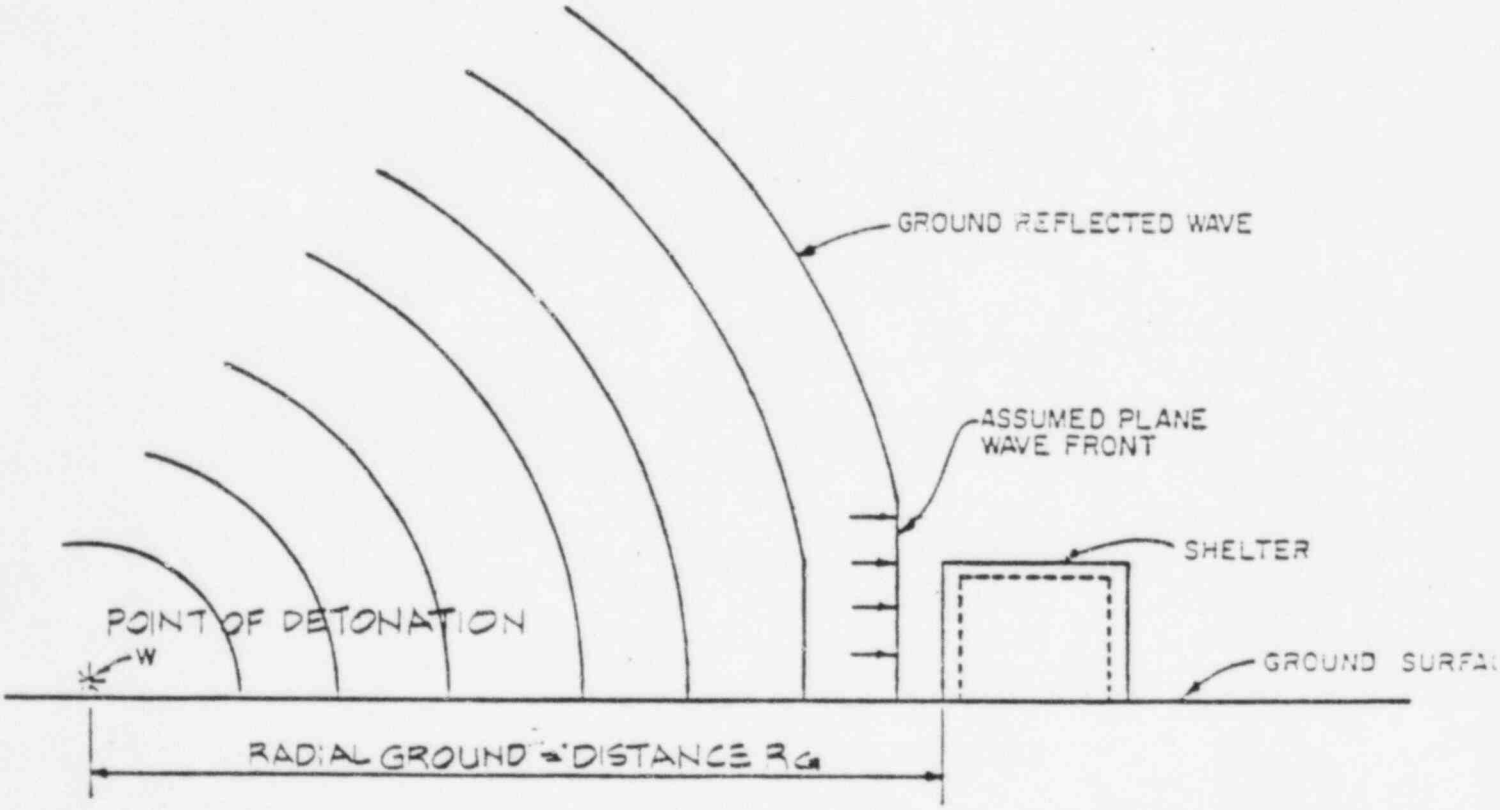
NOTES:

1. NC MEANS NOT COMPUTED. ELEMENT IS LESS CRITICAL THAN IN CORRESPONDING STRUCTURAL UNIT.
2. NA MEANS NOT APPLICABLE. THE ELEMENT OR LOADING CASE DOES NOT EXIST OR APPLY TO THE STRUCTURE UNDER CONSIDERATION.



AIR-BURST BLAST ENVIRONMENT

FIGURE 1



SURFACE-BURST BLAST ENVIRONMENT

FIGURE 2

QUESTION 410.5 (Section 3.4.1)

For those non-seismic Category I vessels, pipes and tanks located outside of buildings, discuss the effect of failure of these items and any potential flooding of safety related structures, systems and components. Provide a similar discussion for non-tornado protected vessels, tanks and piping.

RESPONSE

The failure of non-seismic Category I and non-tornado protected tanks, vessels, and major pipes located outside buildings (Table 410.5-1) will not adversely affect safety-related structures, systems, and components as discussed below.

Tank Failures

The location of tanks in the yard area is shown in Figure 3.8-58. Failure of the tanks on the west and south sides of the power plant complex (Table 410.5-1, items 1 through 5) will not cause potential flooding of safety-related structures, systems, and components. Any flooding due to a failure of these tanks will be contained within seismic Category IIA earth dikes, which will remain stable under both static and dynamic conditions. The design of the earth dikes is discussed in the responses to Questions 240.4 and 241.14.

The tanks on the north side of the power plant complex (Table 410.5-1, items 6 through 9) do not have seismically designed containments around them. Failure of these tanks could cause local flooding. This flooding would not adversely affect safety-related facilities for the following reasons:

a. Surface drainage in this area will drain water towards the Schuylkill River and Possom Hollow Run before it can reach the power plant complex.

b. Seismic Category I electrical cable and duct banks located in the vicinity of these tanks are adequate, as discussed in the response to Question 410.6.

Even if the above dikes were to fail, there would be no impact on other safety-related structures, systems, or components due to site drainage.

Failure of Cooling Tower Basin Wall (Table 410.5-1, items 10 & 11)

The failure of the cooling tower basin wall would not adversely affect safety-related structures, systems, and components as discussed below.

The runoff pattern of water from the cooling tower basin wall failure would be similar to that caused by intense storm precipitation as shown in Figures 2.4-4 and 2.4-5. Most of the flood water from the cooling tower basin would run away from the power plant complex. The worst-case flood conditions for the power plant complex would be created by a failure of the south side of the Unit 1 cooling tower basin wall. For this case, a portion of the cooling tower basin water would flow towards the turbine enclosure. Although some limited turbine enclosure flooding may occur, there would be no impact on safety-related components.

The seismic Category I electrical cable and duct banks and valve pits located in the flow path of the water from the failed cooling tower basin are adequately protected as discussed in the response to Question 410.6.

Failure of Circulating Water Conduit (Table 410.5-1, item 12)

Failure of the conduit within the yard area between the cooling tower basin and the turbine enclosure will cause flooding of this area. Water from the damaged conduit will erode the soil cover and flood the yard.

The runoff pattern will be similar to that shown in Figure 2.4-4. The seismic Category I electrical cable and duct banks and valve pits, located in this area are adequate, as discussed in the response to Question 410.6.

In the most severe case, all the water from the cooling tower basin could drain through the damaged conduit into the yard area between the cooling water pumphouse and turbine enclosure and cause flooding of the condenser pit. However, safety-related systems and components would not be damaged, as discussed in Section 10.4.1.3.3. See also the response to Question 410.92.

Failure of Major Yard Piping

Failure of any of the pipes identified in Table 410.5-1, items 13 through 17, may cause local flooding. However, the intensity and volume of water discharge from any of the pipes is less than that of the cooling water conduit failure discussed above and would not cause damage to any safety-related facilities. Soil erosion caused by failure of these pipes is discussed in the response to Question 410.47.

TABLE 410.5-1

YARD TANKS AND MAJOR PIPING (NON-SEISMIC)

ITEM NO.	TANK OR PIPE DESCRIPTION	CAPACITY OR FLOWS	LOCATION	TYPE OF CONTAINMENT	TORNADO PROTECTION
1	Condensate storage Unit 1 tank	200,000 gal.	West of power plant complex	Earth dikes	None
2	Refueling Water	550,000 gal.	West of power plant complex	Earth dikes	None
3	Condensate storage Unit 2 tank	200,000	South of power plant complex	Earth dikes	None
4	Fuel oil storage tank	200,000	South of power plant complex	Earth dikes	None
5	Aux. boiler fuel oil tank	50,000	South of power plant complex	Earth dikes	None
6	Neutralizing tank	15,000	Water treatment plant north of power plant complex	Reinforced concrete walls	None
7	Neutralizing tank	15,000	Water treatment plant north of power plant complex	Reinforced concrete walls	None
8	Clarified water tank	200,000	Water treatment plant north of power plant complex	None	None
9	Demineralized water tank	50,000	Water treatment plant north of power plant complex	None	None
10	Cooling tower Basin 1	7×10^4	North of power plant complex	Reinforced concrete walls	None
11	Cooling tower Basin 2	7×10^4	North of power plant complex	Reinforced concrete walls	None
12	Cooling water pipes 8.0 ft diam. 4 pressure pipe each unit	450,000 gpm per unit	Between cooling tower and turbine enclosure	Underground	Soil cover
13	36" ϕ makeup water pressure pipe	30,000 gpm	From Schuylkill River to cooling tower	Underground	Soil cover
14	36" ϕ blowdown gravity pipe	18,000 gpm	From cooling tower to Schuylkill River	Underground	Soil cover
15	36" ϕ makeup water pressure pipe	30,000 gpm	From Perkiomen Creek to cooling tower	Underground	Soil cover
16	36" ϕ service water pressure pipe	35,000 gpm	From cooling tower to turbine enclosure	Underground	Soil cover
17	12" ϕ fire loop pressure	2,500 gpm	Around plant complex	Underground	Soil cover

QUESTION 410.6 (Section 3.4.1)

Provide a discussion of the protection afforded safety-related systems and components, including underground cables, with respect to flooding or wetting caused by (1) groundwater, (2) design basis flood, (3) design basis precipitation or (4) failure of an outdoor tank or tank truck.

RESPONSE

Safety-related structures, systems, and components, including underground cables, will not be adversely affected by flooding or wetting caused by the items listed above.

The safety-related structures, systems, and components located within the yard area are shown in Figure 3.8-58. Excavation for seismic Category I structures, pipelines, electrical duct banks, manholes, and underground diesel oil storage tanks are shown in Figure 2.5-37.

Protection of structures against the intrusion of groundwater is discussed in the response to Question 410.8. Groundwater does not affect the yard facilities because the design groundwater elevation (Section 2.4.13.5) is generally below yard facilities.

The design basis flood is not applicable to structures and yard facilities. The design basis flood level with respect to the Schuylkill River is 207 ft (Section 2.4.2.2), which is 10 feet lower than the lowest grade level entrance to any of the safety-related facilities.

The protection of safety-related structures, systems, and components with respect to flooding caused by the design basis precipitation is discussed in Section 2.4.2.3. The runoff patterns for flood water caused by precipitation are shown in Figures 2.4-4 and 2.4-5. Below grade parts of structures are protected from water intrusion as stated in the response to Question 410.8.

The following seismic Category I yard facilities may be susceptible to flooding:

a. 5 Valve pits for RHR and ESW piping

b. 8 Valve pits for diesel oil storage tanks

The valve pits may be temporarily covered with water during an intense storm or major failure of an outdoor tank. However, the

following protective features have been provided to resist resultant flooding of these facilities:

a. They are built as reinforced concrete boxes and are equipped with solid steel manhole covers with gaskets.

b. The tops of slabs of these facilities are elevated 3 to 12 inches above grade.

c. All valve pits are equipped with drain pipes leading seepage water into normal waste drainage system.

All electrical cables are designed to operate under water. Water absorption characteristics for all cables have been reviewed to confirm that even under flooded conditions, electrical cabling in manholes and duct banks will continue to operate properly. In addition, all electrical conduits that travel to electrical manholes outside the structures are sealed to prevent water from entering the structures through the electrical duct banks (Table 3.4-1).

The design features described above will also protect seismic Category I structures and yard facilities against internal flooding in case of failure of an outdoor tank or tank truck. The response to Question 410.5 gives additional discussion of flooding due to failure of outdoor tanks.

QUESTION 410.47 (Section 9.2.1)

Verify that the failure of one buried non seismic Category I pipe in a SSE will not result in failure of safety related buried pipes by soil erosion.

RESPONSE

The integrity of safety-related seismic Category I buried pipe will not be impaired through soil erosion by a failure of one buried non-seismic Category I pipe. This conclusion was based on the following conditions.

a. All but approximately 170 feet of common trench has been constructed in rock, where erosion of the supporting medium would be insignificant.

b. The 170 feet of trench not constructed in rock is in Type 1 fill.

   The non-pressure pipes (gravity lines) in this section, such as the blowdown line and waste and storm lines, do not pose a significant erosion problem. The non-seismic Category I pressure lines in this section consist of a 36-inch Schuylkill River makeup water line and a 12-inch fire line (Figures 410.47-1 and 410.47-2).

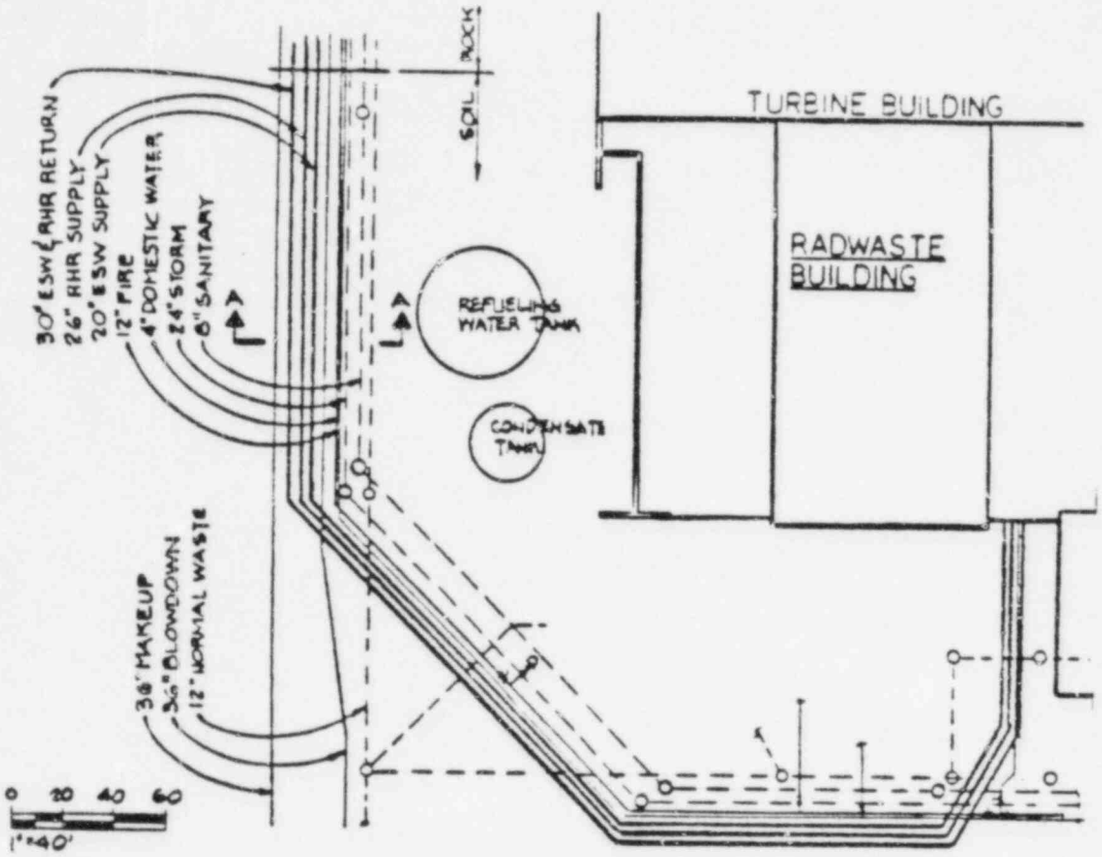
   Failure of the non-seismic Category I pressure pipe may create progressive erosion in the Type 1 fill. It is anticipated that water under pressure would penetrate to the surface, creating a progressively enlarging crater. However, because the water will flow in the direction of least resistance, once the water penetrates to the surface, the crater will be enlarged at a relatively slow pace. The span capacity needed to support the weight of the safety-related pipes in the trench is conservatively estimated to be in excess of 30 feet, based on the maximum allowable spans given in the ASME code. A considerably long time would be required to erode a crater large enough to exceed this span capacity.

c. Instrumentation would give indication in the control room if a break occurred in the non-seismic Category I pressure pipe. Loss of flow from the makeup water line to the cooling tower would result in an alarm in the control room when low level is reached in the cooling tower basin. It is conservatively estimated that low level would be reached within 30 minutes.

Low pressure in the 12-inch fire line, following a break, starts a fire pump that gives an alarm in the control room without a fire signal.

Following an SSE, if either alarm described above is activated, personnel will investigate for evidence of a faulty condition in the pipelines described in (b) above and will initiate any necessary corrective action.

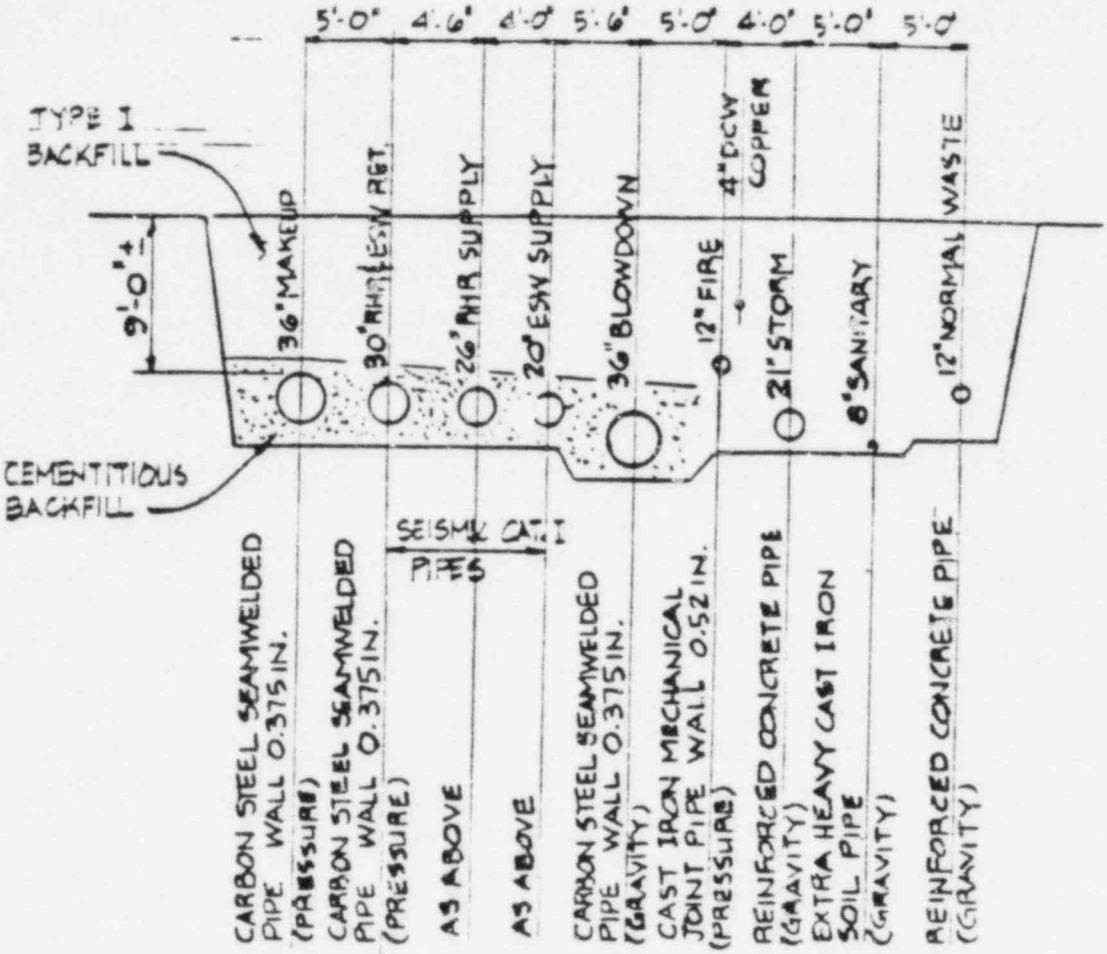
d. The procedures for operator response to a seismic event will include the requirement that, within two hours after an SSE, personnel will investigate for evidence of a faulty condition in the pipelines described in (b) above and will initiate any necessary corrective action.



LIMERICK GENERATING STATION
UNITS 1 AND 2
FINAL SAFETY ANALYSIS REPORT

YARD PIPING

FIGURE 410.47-1 REV. 17, 02/83



SECTION A-A

LIMERICK GENERATING STATION
UNITS 1 AND 2
FINAL SAFETY ANALYSIS REPORT

YARD PIPING

FIGURE 410.47-2 REV. 17, 02/83

PROFESSIONAL QUALIFICATIONS
H. WILLIAM VOLLMER
ENGINEER-IN-CHARGE
STRUCTURAL BRANCH
PHILADELPHIA ELECTRIC COMPANY

My name is William Vollmer. My business address is 2301 Market St., Philadelphia, PA 19101. I am Engineer-in-Charge of the Structural Branch. As a Branch Head, I supervise a group of engineers that provide the structural engineering for generating stations (fossil, hydro, and nuclear) and other facilities on the Philadelphia Electric Company System.

I received training as an engineer by attending Bucknell University from 1963 through 1968 and received a B.S. in Civil Engineering.

Following my graduation, I was employed by the Boeing Company as a Technical Support Engineer. The work was primarily in the field of correlating structural analysis with test results.

I have been employed by Philadelphia Electric Company since April of 1970. In my present capacity, I supervise the work of six graduate structural engineers who handle the structural engineering for Philadelphia Electric Company. Prior to my present assignment as Branch Head of the Structural Branch, I was a Design Engineer in the Structural Branch. In this capacity I was involved in various projects, including modifications to the containment structures at Peach Bottom and Limerick.

For the Limerick Station, I supervise Philadelphia Electric Company's review of structural design and analysis. My group reviews and approves all design specifications related to civil design matters.

I am a member of The American Concrete Institute. I am a Professional Engineer registered in Pennsylvania.

PROFESSIONAL QUALIFICATIONS
DAVID MARANO
ENGINEER-IN-CHARGE
CIVIL SECTION
PHILADELPHIA ELECTRIC COMPANY

My name is David Marano. My business address is 2301 Market St., Philadelphia, PA 19101. I am the Engineer-in-Charge of the Civil Section of the Mechanical Engineering Division of the Engineering and Research Department. In that position, I supervise a group of engineers responsible for civil engineering review of Philadelphia Electric projects, such as fossil fueled power plants, hydroelectric power plants, and nuclear power plants, including the Limerick Generating Station.

I attended Villanova University from 1944 to 1948 and received a degree in Civil Engineering in 1948. I also received a Master of Science in Civil Engineering from the University of Pennsylvania in 1953.

After graduating from Villanova in 1948, I worked in various jobs, including Highway Construction, Power Plant Engineering, and Chemical Plant Engineering. I joined Philadelphia Electric Company in 1955 as a Civil Engineer and worked as a civil engineer on projects such as Eddystone Generating Station, a coal-fired 2 unit power plant, Muddy Run Pumped Storage Hydro Project, a project incorporating a major earth dam, powerhouse and penstock system; Peach Bottom Atomic Power Plant, a 2 unit nuclear power plant; enlargement of Conowingo Project, a program which doubled the electrical capacity of an existing hydroelectric project; preliminary engineering of the Fulton Nuclear Power Plant, a nuclear project; and the Limerick Generating Station.
For the Limerick project, I am responsible for coordinating the Company's

review of all structural related matters. I was appointed Engineer-in-Charge of the Civil Section in 1972 and have served in that capacity since then.

I am a member of The American Society of Civil Engineers. I am a Professional Engineer registered in Pennsylvania, New Jersey, and Maryland.

PROFESSIONAL QUALIFICATIONS
KENNETH P. BUCHERT
CONSULTANT & PROFESSOR AT SOUTHERN ILLINOIS
UNIVERSITY AT EDWARDSVILLE

My name is Kenneth P. Buchert. My business address is 33 Pat Drive, Collinsville, IL 62234. I am a professional engineer registered in Missouri and California. I am a consultant to Bechtel in the area of structures, shells, construction and structural systems.

I received a B.S. in Civil Engineering from the University of Missouri-Columbia in 1945 and served in the Corps of Engineers in the Army and built bridges, buildings and roads in World War II.

I received a M.S. in Civil Engineering from the University of Missouri-Columbia in 1948, and afterward, I worked in the Structures Research Division of the National Advisory Committee for Aeronautics (now NASA) for two years. I worked on buckling of structures and applied the results to supersonic aircraft and missiles.

I then worked as an engineer for Sverdrup & Parcel in St. Louis, and later, for the U.S. Corps of Engineers on the design and construction of aeronautical and propulsion test facilities. I then worked for the Stanley W. Newman Construction Co. in Mobile, Alabama building airport and other buildings. Later I worked for U.S. Steel Corporation in design and construction of blast furnaces, pipe mills, test facilities and other special structures.

In 1961 I returned to the University of Missouri-Columbia and

received my PhD in 1964. I remained as a Professor until 1975. I taught shell structures, construction and other courses. I did consulting on domed roofs, such as the Irving Stadium in Texas. I also did failure analysis and gave expert testimony.

In 1975 I joined Bechtel. I held positions as Chief Civil/Structural Engineer in Gaithersburg, MD and San Francisco, CA. I also served as Chief Civil/Structural Engineer for the Bechtel Power Corporation. Later I was named a Principal Engineer. I also was Project Field Engineer on the Grand Gulf Nuclear Station. I recently retired and formed my own consulting service and have a half time Professorship at SIU-E.

I am a Fellow in ASCE, a member of ASME, a member of the International Association for Shell and Spatial Structures, chairman of the Shell Committee of the Structural Stability Research Council, a member of the American Concrete Institute and a Corresponding Member of the Shell Committee of the European Committee for Constructional Steel.

K. P. BUCHERT

Publications

"Stability of Alclad Plates," NACA, Technical Note 1986, December, 1949.

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"Zur Stabilitat grosser doppelt gekrummter undversteifter Schalen," Der Stahlbau, February, 1965.

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"Buckling of Heads, Cylinders, and Rings," Aerospace Environmental Facility, ARO Engineering Report, Tullahoma, Tennessee, September, 1966.

"Mizzou Housing Project - Application of Turn-Key Contraction to Industrialized Urban Housing in Missouri - Summary Report," University of Missouri-Columbia, 1966 (with W. W. Milner and H. Rubey).

"Comment on General Instability and Optimum Design of Grid-Stiffened Spherical Domes," AIAA Journal, May, 1966.

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"Denting of Circular Bins with Eccentric Drawpoints," ASCE, Structural Division, ST4, August, 1967 (Discussion).

"Effect of Edge Conditions on Buckling of Stiffened and Framed Shells," Engineering Experiment Station Bulletin, Series No. 65. University of Missouri, October, 1967.

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"Plastic Buckling of Framed and Stiffened Domes," Column Research Council, Annual Meeting, May, 1968.

"Space Frame Buckling," AISC, Engineering Journal, October, 1968.

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"Local and General Buckling of Reticulated Shells," IASS, Madrid, Spain, September, 1969.

"Buckling Loads of a Free Formed Shell Structure," International Association of Shell Structures, Madrid, Spain, September, 1969.

"Application of Turn-Key Construction to Industrialized Urban Housing in Missouri," Seventh Space Congress, Cocoa Beach, Florida, April 22, 1970 (with W. W. Milner and H. Rubey).

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"Shell-Like Structures," ASCE National Conference, Phoenix, Arizona, January 11-15, 1971.

"Buckling of Hyperbolic Paraboloids," Engineering Journal, American Institute of Steel Construction, April, 1972.

"Preliminary Stability Analysis of Concrete Cooling Towers," International Association for Shell Structures, Calgary, Alberta, Canada, July, 1972.

"Buckling of Shell and Shell-Like Structures," book published by K.P. Buchert & Associates, 1973.

"Study of Steel Cooling Towers," U.S. Steel, 1973.

"Engineering Management Decisions Related to the Design and Construction of Industrialized Spatial and Shell Structures," International Association of Shell Structures, Symposium on Industrialized Spatial & Shell Structures, Kielce, Poland, June 18-23, 1973.

"Buckling of Curved Flange Shell-Like Columns," Column Research Council, Proceedings, 1974.

"Stability of Reticulated Structures," ASCE, National Meeting, Cincinnati, April 22-26, 1974.

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"Shells for Standard Floor and Roof Elements," ASCE, Structural Division, ST7, July, 1974 (Discussion).

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"Trends in the Design of Pressurized-Water-Reactor Containment Structures and Systems," Nuclear Safety, Vol. 18, No. 2, March-April, 1977 (with D. S. Mehta, H. W. Osgood, A. J. Bingaman).

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"Structural Testing of Class I Seismic HVAC Duct Specimens," Civil Engineering and Nuclear Power, ASCE Speciality Conference, Paper 11-3, Knoxville, Tennessee, September 15-17, 1980.

"Practical Applications of Shell Research," Interntional Conference on Buckling of Shells in Offshore Structures, Imperial College, London, England, April 23 & 24, 1981.

"Remember The Good Old Days? -- What About The Future?" ASCE Specialty Conference, "Managing The Client-Consultant Relationship," San Francisco, November 18-19, 1982.

PROFESSIONAL QUALIFICATIONS
ALBERT K. WONG
ENGINEERING SUPERVISOR
BECHTEL POWER CORPORATION

My name is Albert K. Wong. My business address is 221 Main Street, San Francisco, California 94105.

I received my bachelor of science degree in Civil Engineering from Washington State University in 1962, and completed the program of Master of Science in Structural engineering and structural mechanics at University of California - Berkeley in 1964.

I am a registered civil engineer and a registered structural engineer in the State of California. I am a member of the Structural Engineers Association of Northern California.

I have been employed by Bechtel Power Corporation, San Francisco, California since 1974. My present position is an engineering supervisor responsible for following up all structural engineering work for the Reactor Building, Control Building, Diesel Building, Spray Pond Pumphouse and other non-safety related structures for the Limerick Generating Station. My previous assignments with Bechtel included positions as an engineering supervisor responsible for civil and structural engineering work in a Coal-fired and an oil-fired power plant.

Prior to joining Bechtel I accumulated 10 years of experience in civil engineering and structural engineering. After my graduate study I worked as an assistant engineer in Tudor Engineers, San Francisco, preparing the geometric design manual and the conceptual design of aerial structures for the San Francisco Bay Area rapid transit.

From 1965 through 1973, I was employed by Skidmore, Owings and Merrill, San Francisco as a structural designer and progressed to project structural engineer. My assignments included responsible charge of structural engineering design for major high-rise buildings, university campuses, stadiums, industrial facilities, and residential buildings.

PROFESSIONAL QUALIFICATIONS
JOHN W. BENKERT
ENGINEERING SUPERVISOR
BECHTEL POWER CORPORATION

My name is John W. Benkert. My business address is 50 Beale Street, San Francisco, California 94109. I am a professional civil engineer. I am an engineering supervisor in that position. I supervise the civil/structural group assigned to the Limerick Generating Station. I am a registered Professional Engineer in the Commonwealth of Pennsylvania (PE-017066-E) and a registered Professional Engineer in Civil Engineering in the state of California (C-024342).

I attended Georgia Institute of Technology from 1959 through 1963 and received a Bachelor of Aerospace Engineering degree.

Following my graduation until 1965, I was employed by Hayes International Corporation as a design engineer performing structural design and drafting of launch support equipment for National Aeronautics and Space Administration Saturn Project.

From 1965 through 1969, I was employed by Combustion Engineering, Inc. as a structural engineer performing design of boiler houses, support structures and steam generator internal structural components for approximately 12 fossil fueled power plants.

From 1969 through 1972, I was employed by Gilbert Associates Inc. as a structural engineer primarily involved with two nuclear fueled power plants. Activities included engineering/design of various structures, development of structural portions of Safety Analysis Reports, and development of criteria and specifications.

I have been employed by Bechtel Power Corporation since 1972. My present title is engineering supervisor and I supervise the civil/structural group assigned to the Limerick Generating Station.

PROFESSIONAL QUALIFICATIONS
RANGA PALANISWAMY
ASSISTANT PROJECT ENGINEER
BECHTEL POWER CORPORATION

My name is Ranga Palaniswamy. My business address is 50 Beale Street, San Francisco, California 94105. I am a professional Civil Engineer. I am an Assistant Project Engineer for Limerick Generating Station. I am responsible for all structural analysis and design for the Limerick Generating Station.

I attended the University of Illinois at Chicago from 1968 to 1972, and received a Ph.D degree in Structural Mechanics and Materials. My M.S. degree in Structural Engineering and B.S. degree in Civil Engineering are from Indian Universities.

Following graduation from graduate school, I was employed by Sargent and Lundy Engineers in Chicago for a year. The work was primarily analysis and design of Containment Structures.

Since October 1973, I have been employed by Bechtel Power Corporation. For the first two years, I worked on the analysis and design of concrete Containment Structures.
From 1975 to 1977 I was the Group Leader for the Seismic Requalification of equipments for the Humbolt Bay Nuclear Plant. From 1977 to 1984, I was the Supervisor and subquently, the Project Engineer in Bechtel for the requalification of Mark I Containments for the hydrodynamic leads. My present title is Assistant Project Engineer (Structures) on the Limerick Generating Station. I am responsible for all structural related engineering.

Professional Qualifications

Gordon K. Ashley II, Ph.D.

Engineering Supervisor

Bechtel Power Corporation

San Francisco Power Division

My name is Gordon K. Ashley II. My business address is 50 Beale Street, San Francisco, California, 94109. I am a professional physicist and currently supervise the Mechanical Analysis Special Studies Group of the San Francisco Power Division Nuclear Staff.

I received training as a physicist at the University of Utah where I received a Ph.D.

Following the completion of my graduate studies in 1974, I was employed by the San Francisco Power Division of the Bechtel Power Corporation and assigned to the staff of the Chief Nuclear Engineer as a specialist. My personnel title is Engineering Supervisor and I work as a staff consultant to a number of Bechtel projects. Among my assignments, I have performed numerous analyses for over a dozen nuclear power plants. I have had extensive experience in the determination of the pressure response due to some waves generated by impulsive steam bubble collapse in suppression pools. I was the principle co-developer of the methodology for the U.S. Mark II-type BWR industry to assess the effects of the chugging phenomenon. This methodology has been accepted by the U.S. Nuclear Regulatory Commission and is currently also utilized by the domestic Mark III-type BWR power plants.

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

In the Matter of	)	
	)	
Philadelphia Electric Company	)	Docket Nos. 50-352
	)	50-353
(Limerick Generating Station,	)	
Units 1 and 2)	)	

CERTIFICATE OF SERVICE

I hereby certify that copies of "Transmittal of Testimony of Applicant Relating to the Ability of Safety Related Structures to Withstand the Effects of Postulated Detonation Resulting from the Ruptures of the ARCO and Columbia Gas Transmission Pipelines and Request to Set Schedule" dated February 3, 1984, "Testimony of Philadelphia Electric Company Regarding the Ability of Safety Related Structures to Withstand the Effects of Postulated Detonation Resulting from the Assumed Ruptures of the ARCO and Columbia Gas Transmission Pipelines," and Professional Qualifications of H. William Vollmer, David Marano, Kenneth P. Buchert, Albert K. Wong, John W. Benkert, Ranga Palaniswamy and Gordon K. Ashely, II in the captioned matter, have been served upon the following by deposit in the United States mail this 3rd day of February, 1984:

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