REVISED 2/28/84

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

DOCKETED
USNRC

Before the Atomic Safety and Licensing Board

'84 MAR -1 A11:55

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

RELATED CORRESPONDENCE

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 detonations and the margins above such values inherent in building design. (TR 5934-44). This testimony is responsive to that request and includes the following:

   o 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.

The witnesses sponsoring particular portions of this testimony are indicated on Attachment 1 hereto.

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, burst and detonation are used interchangeably.

TYPES OF BLAST ENVIRONMENT

4. The possible types of detonation loading on plant facilities can be identified as free-air burst loads, air burst loads and surface burst loads. The free-air burst environment is produced by the blast wave propogating away from the center of the explosion striking the structure without intermediate amplification of the initial shock wave (Figure 1') (Applicants' Exhibit 15). The air burst environment is produced by a detonation which occurs above the ground surface and at a 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) (Applicants' Exhibit 16) 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. The height of the intersection of

the incident wave, reflected wave and Mach front, which increases as the wave propogates away from the center of the detonation is called the triple point. A structure is subjected to a plane wave when the height of the triple point exceeds the height of the structure. Above the triple point two separate shocks will be seen, the first being due to the incident wave and the second to the reflected wave.

5. The surface burst environment is produced by a detonation which occurs at or very near the ground surface. The reflected wave merges with the incident wave at the point of detonation to form a single wave, which is essentially hemispherical in shape, and resembles a Mach front as in an air burst below the triple front (Figure 2) (Applicants' Exhibit 17).

5'. This analysis assumes that detonation of an unconfined natural gas mixture could occur although the evidence in this proceeding is clear and uncontradicted that this is not possible. Furthermore, the assumption that an elevated detonation can occur is also not credible due to the lack of an ignition source, let alone a source of energy sufficient to cause detonation. In the initial testimony a surface burst was analyzed very conservatively to determine the pressure on walls of safety related structures. When it was recognized that an "optimized" height air burst could give theoretically higher values, it was decided to attempt to more realistically, but still conservatively, evaluate this case as well as to reexamine the case previously presented.

BASIS OF ANALYSIS

6. As discussed in more detail below, the blast effects from the Columbia Gas Transmission pipeline have been recalculated using the percentage of gas-air mixture that theoretically could detonate in accordance with Regulatory Guide 1.91 Rev. 1, and with all other assumptions contained in Walsh's original testimony remaining unchanged. (Testimony of John D. Walsh related to contentions V-3a and V-3b) (Tr. ff 5411). That testimony discussed the maximum pressure that would be developed at any of the safety related structures for the Station assuming a surface burst and a detonable mixture approximately four times that suggested by Regulatory Guide 1.91 Rev. 1.

6'. A 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 for predicting the centroid of the explosion 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) (Applicants' Exhibit 18). Utilizing the distances from this line to safety related structures, the resulting pressure for each of the particular structures was determined as discussed below.

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 in this testimony. As calculated by Walsh, the maximum peak positive reflected pressure from an ARCO pipeline explosion is 1.9 psi.

8. Initially, pressures on walls and roofs were calculated assuming a surface blast and the Regulatory Guide 1.91 Rev. 1 assumption. Table II, Column 1 presents the results of this evaluation. Using Figure 4-12 of Reference 1, the reflected pressure on the wall of each of the safety related structures was obtained as a function of the scaled distance. Inasmuch as the wave front for a surface burst is perpendicular to the roof, no reflection occurs. The roof pressure was determined utilizing Equation 4-8 of Reference 1.

8'. Even though no source of ignition or detonation could occur in the open air, the case of an elevated detonation was nevertheless examined for the sake of completeness. It is helpful to discuss the relationship between surface bursts and air bursts in order to understand why, for particular conditions, an elevated burst can produce greater pressures. For a surface burst, there is instantaneous reinforcement between the reflected and incident waves. As the elevation of the burst increases, some of the energy is directed downward, resulting in a lessening of blast pressures at a given distance. This can be seen by comparing Figures 4-5 and 4-12 of Reference 1. For very small elevations the correction for ground reflection is small as shown in Figure 4-6 of Reference 1. As the height increases, two competing effects occur. First, the range increases, thus lowering the free air burst pressure in Figure 4-5 of Reference 1. Second, the reflected pressure coefficient increases to a maximum, then decreases to a constant value for peak positive incident pressures of interest here. The resultant is a maximum pressure at a specific height. For the case at hand, because the source cannot rise above 500 feet, this elevation yields the maximum resulting pressures for an air burst. For this case, the peak reflected pressure on the walls is calculated as per Section 4-7(e) of Reference 1. The results of these calculations are shown in Column 2 of Table 2.

8". For an air burst, the pressures on the roof are calculated in one of two ways. For the case where the elevation of the triple point exceeds the elevation of the roof, equation 4-8 of Reference 1 is used. The roof pressure is calculated as per Section 4-14(c) of Reference 1. Otherwise, it is calculated as a free air burst using Figure 4-5 of Reference 1. While it is ultraconservative to assume that four times the Regulatory Guide 1.91 Rev. 1 mixture would detonate, the surface and air burst pressures were calculated in the manner described above using Reference 1 methodology. These cases are presented in Columns 3 and 4 of Table

II. It should be noted that when comparing Column 3 with Column 5 (pressures used in structural assessment in Table I) which is a comparable case, the differences result from differences in the interpolation of the figures in Reference 1 and Regulatory Guide 1.91 Rev. 0.

9. Various points along the line of the possible explosion, as indicated in Figure 3 (Applicant's Exhibit 18), were examined to determine the pressures applied 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 evaluated, the critical element of that structure could be determined based upon the peak positive reflected pressure as determined for each 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, except for the reactor building

where actual 28-day concrete strengths 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 wall pressures on critical locations from this event contained in Table I were compared to the maximum for these structures as presented in Table II. For these structures, the Reading Railroad explosion was found to bound the Columbia Gas Transmission pipeline explosion for the structure walls.

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 TO 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.

SUMMARY OF RESULTS OF ANALYSIS

13. Table I presents the pressures for each safety related structure as contained in the original testimony. As previously discussed above, it is appropriate to compare the values of Table II, Column 1 with the controlling pressure of Table 1. Because there were already margin present, the lower pressures of Column 1 would indicate a significantly greater margin. While Column 2 of Table II represents an air burst which is not considered to be possible, margins compared to the pressures in Table I also exist. Merely to show the amount of margin, the results of Table II, Columns 3 and 4, were compared to Column 5. There are two cases where the pressures exceed those which were previously used for structural

assessment. For these cases, the margins were recalculated and margins do exist. 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. One additional item sho be noted. The peak calculated pressure resulting from the railroad car explosion is listed in Table I 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 the reactor building analysis, 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. None of the analyses utilized 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.

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 (Applicant's Exhibits 19, 20, 21 and 22) 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 (Applicant's Exhibits 23 and 24) 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 adaquately 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) (Applicant's Exhibits 25 and 26). 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) (Applicant's Exhibit 21) 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) (Applicant's Exhibit 27) 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) (Applicant's Exhibit 28). 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.

ATTACHMENT 1

WITNESS RESPONSIBILITY BREAKDOWN FOR CONTENTION V-3a AND V-3b REVISED TESTIMONY

Witness	Responsiblity by Paragraphs
John W. Benkert, Albert K. Wong and Ranga Palaniswamy	1-6[1], 8-27
John W. Walsh	1-11, 13, 27
Gordon K. Ashley, II	1-6[1], 8-9, 13, 27
H. William Vollmer	1-6[1], 8-27
Kenneth P. Buchert	17-19

Mr. Vincent S. Boyer will be the lead witness regarding this testimony.

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/TANK CAR 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	ROOF SHEAR	EXT. WALL BENDING	EXT. WALL SHEAR	OVER-TURNING MOMENT (FT-K)	STORY SHEAR (K)	OVER-TURNING MOMENT (FT-K)	STORY SHEAR (K)	
REACTOR BLDG. UNIT 1	NC(1)	NC	NC	NC	5.3	16.1	NC	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	NC					
DIESEL GEN. BLDG. UNIT 1	NC	NC	NC	NC	5.7	16.4	NC	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	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.

TABLE II
SUMMARY OF PRESSURES RESULTING FROM
A NATURAL GAS PIPELINE DETONATION

	COLUMN 1		COLUMN 2		COLUMN 3		COLUMN 4		COLUMN 5	
Pressure (PR) PSI	REG. GUIDE 1.91 REV. 1 SURFACE BURST		REG. GUIDE 1.91 REV. 1 AIR BURST		4 x REG. GUIDE SURFACE BURST		4 x REG. GUIDE AIR BURST		PRESSURES USED IN STRUCTURAL ASSESSMENT	
BLDG.	ROOF	EXT. WALL	ROOF	EXT. WALL	ROOF	EXT. WALL	ROOF	EXT. WALL	ROOF	EXT. WALL
DIESEL GEN.	1.9	5.8	3.5	8.3	4.0	13.0	2.5	16.0	6.7	16.4
REACTOR BLDG.	1.2	5.8	2.8	8.3	2.6	13.0	5.2	16.0	5.4	16.1
CONTROL STRUCTURE	1.6	5.0	2.8	6.9	3.3	11.0	4.7	14.0	4.9	10.0
SPRAY POND PUMP HOUSE	0.8	2.5	1.2	3.3	1.8	5.0	1.4	6.0	3.0	5.0

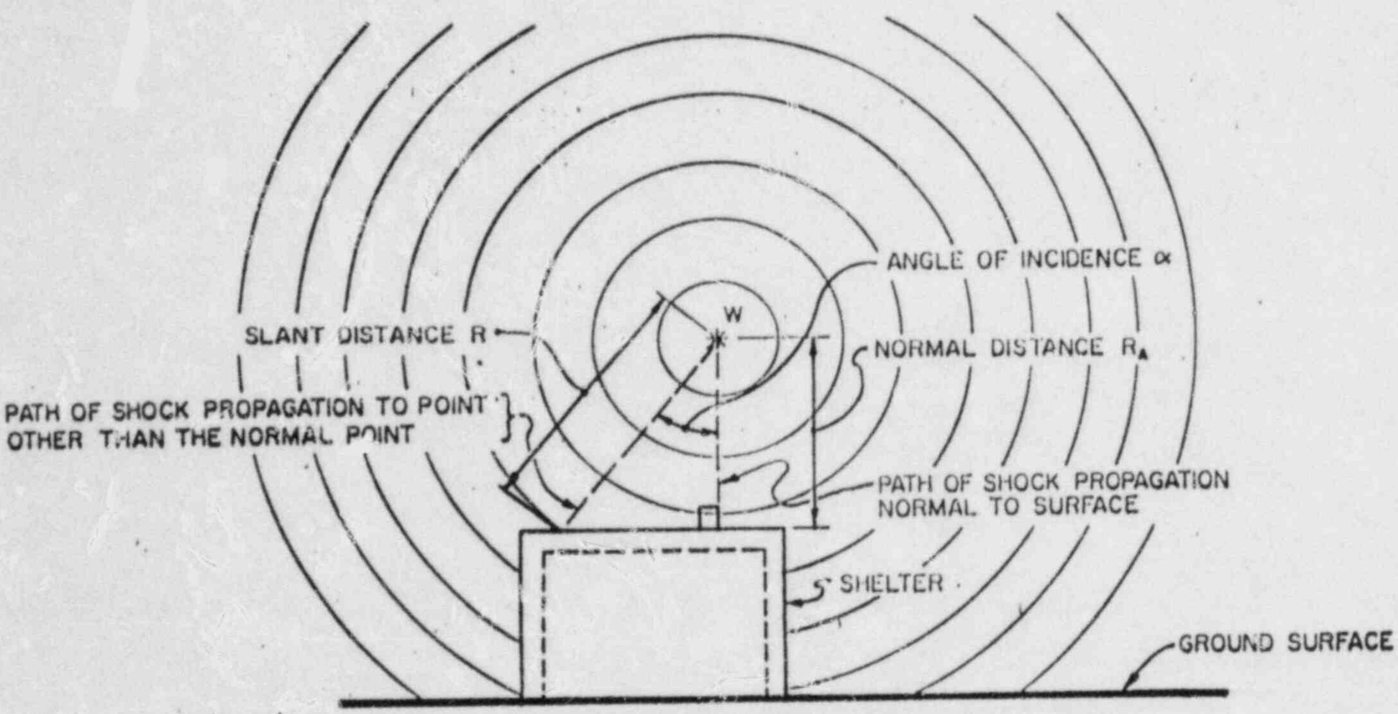
ANGLE OF INCIDENCE α
SLANT DISTANCE R
W
NORMAL DISTANCE R_A
PATH OF SHOCK PROPAGATION TO POINT
OTHER THAN THE NORMAL POINT
PATH OF SHOCK PROPAGATION
NORMAL TO SURFACE
SHELTER
GROUND SURFACE

FIGURE 1'

RELATED CORRESPONDENCE

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UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

OFFICE OF SECRETARY DOCKETING & SERVICE BRANCH

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 "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," dated February 28, 1984, in the captioned matter have been served upon the following by deposit in the United States mail this 29th day of February, 1984:

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