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 P. T. KUO AND NORMAN D. ROMNEY
CONCERNING MARGINS OF STRUCTURAL CAPABILITY
OF CATEGORY 1 STRUCTURE TO RESIST BLAST OVERPRESSURE
AND MODE OF STRUCTURAL FAILURE OF THE COOLING TOWERS

Q1. Please state your name, your position, and the nature of your work at the Nuclear Regulatory Commission (NRC).

A1. My name is Pao-Tsin Kuo. I am a structural engineer and am employed by the NRC as a Section Leader in the Structural and Geotechnical Engineering Branch, Division of Engineering, Office of Nuclear Reactor Regulation. My duties include, among other things, the supervision of the professional work of the structural engineers in my section and subsequent review of their professional work product. In that regard, I have reviewed the testimony and analysis of Norman D. Romney that is set forth below. I have found Mr. Romney's testimony and analysis to be accurate and to meet the standards and regulatory review requirements of this Branch. A statement of my professional qualifications is attached.

My name is Norman D. Romney. I am employed as a Structural Engineer in the Structural and Geotechnical Engineering Branch, Division of Engineering, Office of Nuclear Reactor Regulation. My duties include the evaluation of the analyses and design of Category I (safety related) structures in Nuclear Power Plant Safety Analysis Reports for technical adequacy, completeness and conformance to the NRC Standard Review Plan (SRP). A statement of my professional qualifications is attached to this testimony.

Q2. What is the nature of your testimony?

A2. My testimony concerns the margins of structural capability of Category 1 structures to resist blast overpressure and the mode and effect of structural failure of the plant cooling towers.

Q3. Are you familiar with the Licensing Board's concerns regarding blast pressures on safety-related structures?

A3. Yes. As I understand it, the Board has two concerns. The first is to learn whether the safety-related structures can withstand the blast pressures as specified in the applicant's FSAR and enveloped by the Siting Analysis Branch (SAB) independent calculations. The second is to learn whether there are adequate margins between the maximum blast overpressure the safety-related structures can ultimately withstand and the blast overpressure the structures may be exposed to from various hazards. The concerns are related and may best be answered by addressing the Board's second concern.

Q4. Has the applicant addressed the problems of structural margins between maximum blast overpressure capability and the calculated blast overpressure?

A4. Yes. I have reviewed the information contained in the applicant's "Table 1." Table 1 of the applicant's testimony defines the actual pressures imposed on the critical elements of safety-related structures as a result of various sources of explosions. The applicant compared the actual blast pressure on a structural element with the maximum pressure that element can withstand and remain functional. The applicant expressed the difference between actual and maximum pressure as a percentage of margin.

Q5. You used the expression, "Critical Element." Will you explain what you mean by "Critical Element."

A5. In the analysis of structures, engineers sometimes refer to a "critical element" or a "critical structural element." In this sense a "critical structural element" is that beam, column, wall, slab, or floor because of its geometry and/or orientation bears a significantly larger stress than other like structural elements. Figure 1, "Selection of Critical Element for Purposes of Analysis and Design," represents a concrete slab (which may be a wall, floor, or roof) which is rigidly supported on all four sides. Look first at what I have labeled as a "noncritical element" along the east edge of the slab and one notes that it is supported at both north and south edges as well as continuously along its east side. A load applied to this noncritical element would produce relatively little stress due to the continuous support. Comparing this with a "critical element" one notes that the critical element is supported only at the north and south sides. The portions of slab adjoining the east and west edges are assumed to offer no support since they will deflect under load equivalent to the deflection of the "critical element." Due to the absence of support along the east and west edges, a load applied to this critical element would produce forces that are higher than the forces in the noncritical element. The forces that are found to exist within the critical element are applied throughout the slab. This approach is conservative because the entire slab is designed for the higher forces in the critical element even though the noncritical elements of the same slab would have lower forces.

Q6. Are you familiar with the applicant's testimony on the "maximum blast overpressure" that a structural element can withstand?

A6. Yes.

Q7. Can you explain how the applicant used the "maximum blast overpressures"?

A7. Certainly. First there are terms which I will be using that must be defined:

a. Elastic/Plastic deformation - Figure 2 is a typical load-deformation curve for an idealized elastic/plastic system which shows the rate at which a material deforms in response to an increase of load. This figure is for purposes of this testimony and is not intended to represent the specific behavior or characteristics of material used at Limerick. For purposes of this discussion the term "load" as used in

Figure 2 may be forces or moments; the term "deformation" may be displacement or rotation as the material responds to the respective load applied to it. A material or structural element that has deformed within the elastic range will return to its original shape when the load causing the deformation is removed Loads above point "B" on the load axis will cause the material to deform beyond its elastic limit and into the plastic range. Materials or structural elements that have deformed into the plastic range will not return to their original shape.

b. <u>Ductility</u> - is the property of a material to deform beyond its elastic limit without rupturing. The limit of ductility, or maximum deformation, is defined by the ultimate limit or rupture point of the material.

c. <u>Ductility ratio</u> - is the measure of a material's total deformation to a given load beyond its elastic limit. For example, referring to Figure 2, a material has an elastic range of deformation limited to B; additional deformation into the plastic range may have a deformation of C; the ductility ratio is then C divided by B.

In the applicant's calculations of maximum blast overpressure, an upper bound limit of ductility ratio was set at 3. This limit for the structural elements under consideration is within that allowed by Appendix C of the Code Requirements for Nuclear Safety Related Concrete Structures, American Concrete Institute (ACI 349-76) code regarding impactive loads that are time dependent and is therefore acceptable to the staff. Once the applicant established upper bound limit of deformation as a ductility ratio value of 3, the applicant performed a calculation to determine the amount of dynamic pressure necessary to achieve a deformation consistent with a ductility ratio of 3.

Q8. Can you explain the methodology used by the applicant to calculate ductility ratios?

A8. Yes. The applicant used a methodology contained in Department of the Army Technical Manual, Structures to Resist the Effects of Accidental Explosions, TM 5-1300, hereinafter referred to as the Army Manual. The method used in the Army Manual considers a critical element from a slab and, based on its concrete strength amount of reinforcing, etc., calculates a maximum deflection at the mid span of the critical element due to a blast overpressure. This maximum deflection may cause plastic deformation at the support of the critical element. Then an "elastic deflection" is calculated, which the Army Manual refers to as that deflection which is consistent with the supports of the critical element reaching their elastic limit. From these two deflections, the ductility ratio is determined by dividing the maximum deflection by the elastic deflection.

Q9. Is the applicant's use of the methodology described in the Army Technical Manual TM 5-1300 to determine the limit of structural capability acceptable to the staff?

A9. The staff accepts the methodology employed by the Army Manual. However, it is not apparent how the Army Manual method relates deflection at the mid-span to strains occurring at the supports. The strain or deformation of a critical element with fixed supports and with the type of load that occurs during blast overpressure is not uniform across the length of the critical element. The relationship between deformation at the support and deflection at the mid-span is nonlinear with the deformation being highest at the supports. A complete analysis would require making an assessment of ductility directly at the supports where the strains are highest and most critical.

Q10. What is your judgement as to the adequacy of the applicant's calculation?

A10. It is my judgment that the ductility ratio at the support is higher than 3, which is the maximum value at the mid-span but less than the value of 10, which is the maximum the Staff would find acceptable.

Q11. Given the limits of structural capability established by the above method, what did the applicant do to determine the margins between actual calculated blast pressure and the limits of structural capability?

A11. As previously mentioned the applicant used the Army Manual method to determine the blast overpressure that would cause structural elements of safety- related structures exposed to blast sources to deform up to a ductility ratio of 3 at the mid-span. The blast pressure thus determined would be the maximum pressure the structural elements could withstand. Next, the applicant compared this maximum blast pressure with the calculated blast pressures from the various sources of explosions. The applicant expressed the difference between actual pressure and maximum pressure as a percent (%) of margin between actual pressure and the amount of pressure that would cause a deformation of the structure equal to a ductility ratio of 3.

Q12. Do you accept this method of comparison?

A12. Yes. The percentage margin gives an index of how much the blast pressure must be increased before the limit of structural capability is reached.

Q13. Did the applicant make an assessment as to the effects of blast pressure on the entire structure, i.e., global effects, as well as various structural elements?

A13. Yes. For each of the safety-related structures, with the exception of the Control Building, the applicant applied the maximum blast overpressure from the various sources of explosion to an entire safety related building and determined the overturning moments and story shear for that building. The overturning moments thus determined were compared with the overturning moments and story shears determined for the Safe Shutdown Earthquake for the respective buildings. The overturning moments and story shears for safety-related structures under SSE conditions were previously found acceptable by the staff. In each case the overturning moments and story shears for the blast pressure were found to be less than those caused by the SSE condition.

Q14. Can you explain what is meant by overturning moment?

A14. Yes. An overturning moment is the product of the amount of lateral force applied to the center-of-gravity of a structure and the perpendicular distance from the axis of rotation to the center-of-gravity of the structure. The overturning moment tends to cause the structure to tip over as a rigid body about the axis of rotation.

Q15. Can you explain what is meant by story shear?

A15. Yes. Story shear is the amount of force that is necessary to cause a story or floor of a building to deform relative to an adjacent story or floor.

Q16. Is the applicant's comparison of global building response acceptable to the staff?

A16. Yes. Comparison of overturning moment and story shears caused by blast pressure with that caused by SSE conditions allows for comparison with values of overturning moment and story shear previously accepted by the staff. The story shears and overturning moments found for the blast condition were less than those determined for the SSE condition and therefore acceptable.

Q17. Are you satisfied that the applicant considered global response on each of the affected safety-related structures?

A17. The applicant considered overturning moment and story shears for the Reactor Building, Diesel Generator Building, and Spray Pond Pumphouse. The Control Building is structurally integral with the Reactor Enclosure Building and a separate overturning moment does not need to be calculated. It is my judgement that the control building would not be overstressed in shear.

Q18. In the applicant's assessment of structural capability of safety-related structures to resist blast overpressures, was the minimum code-allowable 28-day concrete strength used?

A18. The applicant used actual concrete strength as determined from cylinder tests of concrete at 28 days of curing. This is acceptable engineering practice provided that the appropriate ACI Code and ASTM Specifications are followed. The applicant referenced the appropriate ACI Codes and ASTM Specifications in the FSAR. The applicable ASTM Specifications are ASTM C-172, C-31, and C-39. The ACI Code 214 is also applicable.

Q19. Are you familiar with the Board's concerns regarding the failure of the plant cooling towers?

A19. Yes. As I understand it, the Board has two concerns, namely, the possibility of the cooling towers falling onto a Category I structure and the possibility of debris from a collapsed cooling tower penetrating approximately 4 ft of soil and impacting Category I buried piping or electrical duct banks in the vicinity of the cooling towers.

Q20. Have you reviewed the applicant's testimony regarding these concerns?

A20. Yes.

Q21. What is your opinion of the applicant's response to the Board's concern regarding collapse of the cooling tower onto Category I structures?

A21. The applicant did not specifically address this concern. However, the nearest Category I structure to the cooling towers is the Spray Pond Pumphouse which is 520.5 ft from the base of the cooling tower. Although it is highly unlikely, assuming that the cooling tower rotated as a rigid body about the edge of its base, the top of the tower could not reach the Spray Pond Pumphouse because the tower is only 507.5 ft high and the distance to the Spray Pond Pumphouse is 520.5 feet.

Q22. What is your opinion of the applicant's response to the Board's concern regarding debris from the tower penetrating buried piping?

A22. The applicant made two assumptions based principally on engineering judgment. The applicant assumed that debris from a collapsed cooling tower will be confined within a target area with a radius equal to one tower base diameter measured from the center of the tower. Next the applicant assumed that the cooling tower failure would produce a concrete fragment of 5 ft x 5 ft x 1 ft falling within the previously defined target area. The applicant calculated that the postulated concrete fragment would fall freely 550 ft and penetrate 2.8 ft into the soil. The minimum soil cover is 4 ft.

Q23. What is your assessment of the degree of conservatism associated with the choice of the size of debris impacting the ground cover over the safety related piping and electrical duct bank?

A23. I find that it is conservative because the thin shell characteristics of the upper portions of the tower, the close spacing of reinforcing bar within the cooling tower structure, and the likely mode of failure by buckling and collapse would be likely to produce individually sized debris significantly smaller than the applicant's assumed 5 ft x 5 ft x 1 ft size.

Q24. What is your assessment of the applicant's calculation of 2.8 ft penetration into the 4 ft of cover over the safety related piping and electrical duct banks?

A24. The applicant calculated the 2.8 ft penetration using methodology similar to that used by the NRC staff for the calculation of tornado missile penetrations. The 2.8 ft. penetration calculated by the applicant is reasonable and acceptable to the Staff.

Q25. What is your opinion of the applicant's testimony regarding the likely failure mode of the cooling towers, including the assumption that all resulting debris would fall within a target area with a radius of one tower base diameter?

A25. I have reviewed the applicant's testimony including the references provided with it on this subject. Based principally on engineering judgment and the lack of any substantial evidence to suggest otherwise, I agree with the applicant's findings.

Professional Qualifications of

Norman D. Romney
Structural and Geotechnical Engineering Branch
Division of Engineering

My name is Norman D. Romney. I am a structural engineer in the Structural and Geotechnical Engineering Branch, Division of Engineering, Office of Nuclear Reactor Regulation, U.S. Nuclear Regulatory Commission. My duties include the review and evaluation of structural engineering aspects of safety-related structures and components as proposed in Safety Analysis Reports for technical adequacy, completeness, and conformance to the NRC Standard Review Plan and other requirements.

I received a Bachelor of Science in Civil Engineering from Howard University in 1974. I have completed graduate courses in Reinforced Concrete Design, Foundation Design, and Economics. I have also completed Continuing Engineering Education courses in Concrete Technology, Construction Management, Dynamic Analysis of Ocean Structures, and BWR Reactor Fundamentals.

From 1974 to 1980 I was employed as a structural engineer with the Bechtel Power Corporation in Gaithersburg, Maryland. Typical duties with Bechtel included the structural steel and reinforced concrete design of nuclear power plant buildings, design of masonry walls, pipe supports, pipe whip restraints, and platforms. Structural engineering project assignments included the Grand Gulf Nuclear Power Plant, Davis-Besse Unit 1, Joseph M. Farley, and Millstone. I joined the Nuclear Regulatory Commission in 1980.

I am a Registered Professional Engineer in Virginia. I am a member of the National Society of Professional Engineers.

PROFESSIONAL QUALIFICATIONS

PAO-TSIN KUO

U.S. NUCLEAR REGULATORY COMMISSION

STRUCTURAL AND GEOTECHNICAL ENGINEERING BRANCH

DIVISION OF ENGINEERING

I am a Section Leader in Division of Engineering, responsible for review and evaluation of design criteria for structural systems, static and dynamic analyses, design, and testing of safety-related structures, and the criteria for protection against the adverse effects associated with natural environmental loads and postulated failures of fluid systems for nuclear facilities.

I received an Engineering Diploma in Civil Engineering from Taipei Institute of Technology in 1958, a M.S. degree in Civil Engineering from North Dakota State University in 1966, and a Ph.D. degree in Civil Engineering from Rice University in 1974. I completed my graduate studies all under scholarships and fellowships. My major fields of studies included structural dynamics, engineering mechanics and earthquake engineering in particular. I was elected to be a member of Sigma Xi honor society in 1970. Currently, I am a member of both Earthquake Engineering Research Institute and American Society of Civil Engineers. I am also a registered Professional Engineer in the State of Maryland.

From September 1958 to June 1960, I served as a commissioned lieutenant officer with Chinese Marine Corps. During the last eight months of this period I also served as a field engineer involved in the reconstruction of a reinforced concrete dam destroyed by a record flood.

From June 1960 to June 1961, I was employed by Taiwan Water Conservancy Bureau as a civil engineer involved in embankment line layout.

From July 1961 to March 1965, I joined Keelung Harbor Bureau in Taiwan, China. I served as a field structural engineer responsible for construction of a number of harbor structures including both steel and reinforced concrete structures.

From April 1965 to September 1965, I was employed by John A. Mackel and Associates in Los Angeles, California as a Designer responsible for analysis and design of highrise commercial buildings.

Immediately after I received my MSCE degree, I was employed as a Senior Design Engineer by Cushing and Nevell Technical Design Corporation on contract to Ebasco Services, Inc. in New York City from July 1966 to August 1967. During this period I was primarily concerned with the structural analysis and design for a commercial nuclear power plant.

From March 1971 to May 1975, I was associated with Bechtel Power Corporation In Gaithersburg, Maryland. Between the years of 1971 and 1973, I served as a Senior Engineer in charge of seismic analyses for a commercial nuclear power plant. I was also responsible for reviewing and approving the seismic qualifications of mechanical and electrical equipment by either analytical means or laboratory testing. During this period I was also engaged in impact analysis for cask drop and aircraft impact and in developing design criteria and methods for pipe whip restraint design.

Between the years of 1973 to 1975, I served as an Engineering Specialist responsible for reviewing and establishing criteria for seismic analyses of structures, performing specialized investigative studies in the seismic analysis area, and advising the Chief Engineer concerning problems related to seismic analyses and design.

Representing the Gaithersburg Division, I also served as a member of the Bechtel Seismic Task Force Committee during the period from 1972 to 1975. The Committee had the responsibility of establishing the corporate standards related to seismic analyses and design. We co-authored the Bechtel topical report, BC-TOP-4A, entitled "Seismic Analyses of

Structures and Equipment for Nuclear Power Plants" which is widely referenced by the nuclear industry.

In June 1975, I joined the Nuclear Regulatory Commission and have remained with this organization since. During this time, I have participated in the review and evaluation of many construction permits and operating licenses and in the generic review of topical reports, seismic analysis methodology, and structural aspects of suppression pool dynamics. I have also participated in the NRC sponsored confirmatory research activities related to seismic analyses.

I have also served as a member of AISC Nuclear Specification Task Committee III responsible for writing the nuclear specification (ANSI N690).

Professional Qualifications of

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Structural and Geotechnical Engineering Branch
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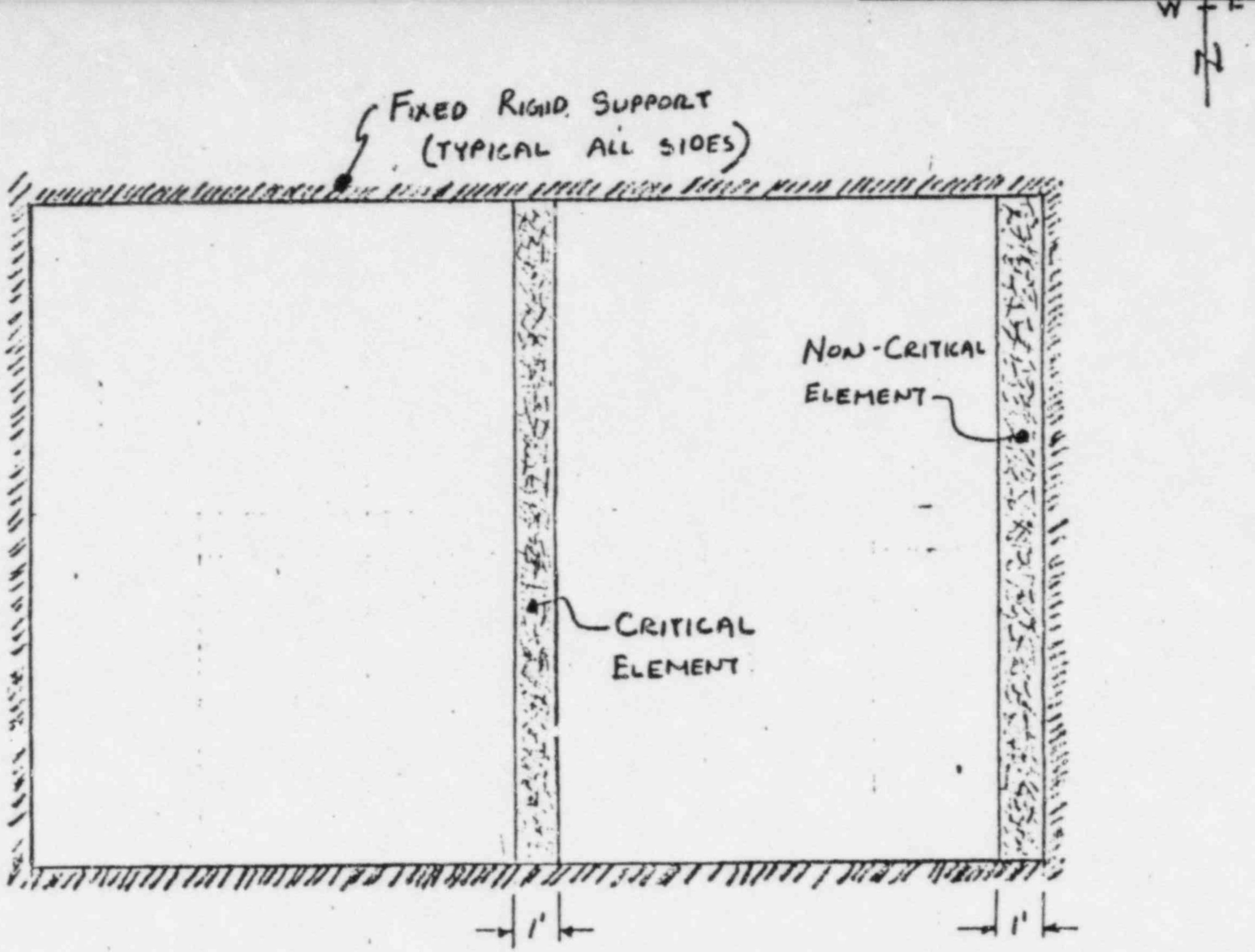

FIG. 1 - SELECTION OF CRITICAL ELEMENT FOR PURPOSES OF ANALYSIS AND DESIGN

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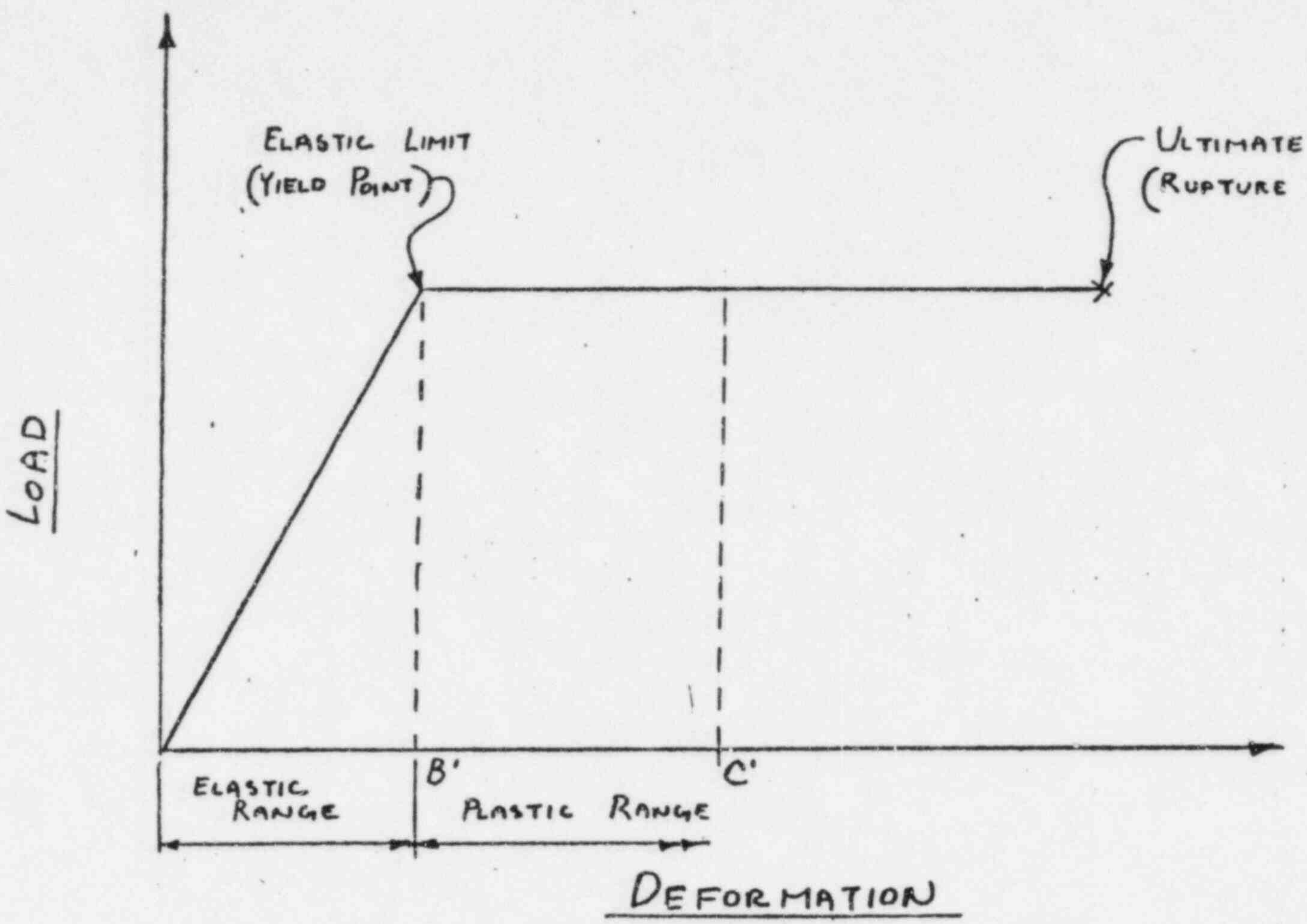

FIG. 2 - TYPICAL LOAD DEFORMATION CURVE -

IDEALIZED ELASTIC-PLASTIC SYSTEM

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