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Dalwyn R. Davidson
VICE PRESIDENT
SYSTEM ENGINEERING AND CONSTRUCTION

April 19, 1982

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

Perry Nuclear Power Plant
Docket Nos. 50-440; 50-441
Response to Draft SER
Structural Engineering Branch

Dear Mr. Schwencer:

This letter and its attachment is submitted to provide additional information regarding the intake tunnel structure.

It is our intention to incorporate this response in a subsequent amendment to our Final Safety Analysis Report.

Very Truly Yours,

Dalwyn R. Davidson

Dalwyn R. Davidson
Vice President
System Engineering and Construction

DRD: mlb

cc: Jay Silberg
John Stefano
Max Gildner

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Perry Nuclear Power Plant
Intake and Discharge Tunnels

Responses to Additional Information
Requested by the NRC - March 30, 1982

1. With respect to the design of the 10 inches concrete tunnel lining provide justification and basis for taking exception on the limitations as imposed on Section 1.4.3 of ACI 322-78 code.

There is no code that has been developed that reflects the design consideration of tunnel supports and linings. The fact that the tunnel lining is not a simple flexural or compression member makes application of existing codes for reinforced (ACI 318-77) and structural plain concrete (ACI 322-72) inadvisable. The tunnel is not a determinant structure because of its complex interaction with the surrounding rock mass and therefore the ACI code, as developed for beams, columns and slabs for buildings, is not appropriate for describing tunnels. The general failing of most building codes to consider tunnels has prompted the United States Department of Transportation, Urban Mass Transit Administration to fund the development of a more specialized "tunnel code". This research has been carried out by the Department of Civil Engineering at the University of Illinois. The results are currently in draft form only, subject to review by some industry representatives and the American Society of Civil Engineers' Tunnel Lining Design Committee, and not available for reference at this time.

The design of the concrete lining for the Perry tunnels did use the existing ACI codes for guidance, but recognizing the deficiences in the code with respect to tunnel lining design, the codes were not the sole source of justification for the design. In particular, the behavior of an unreinforced concrete-lined tunnel under seismic loads is not considered by ACI 322-72. The limitation of paragraph 1.4.3 is thus not appropriate for tunnels.

A two-fold approach to the design of the tunnel lining under seismic loads was used in the case of the Perry tunnels. First, seismic loads were applied via computer analyses to the proposed tunnel lining. The tensile stresses generated by the seismic loads were then compared to the tensile strength of the concrete, a function of the modulus of rupture. The second approach to the consideration of the behavior of a concrete-lined tunnel under seismic loads was to study the history of other existing tunnels which have been subjected to seismic loads. This history was well-documented in STRAAM Engineers, Inc. report, "Design of Concrete Final Lining," November 1979, with the resulting conclusion that all tunnels, whether reinforced concrete-lined, or unreinforced concrete-lined, behave well under seismic loads with damage being very minimal.

2. With respect to using allowable concrete tensile stress for tunnel lining design, it is believed that under the range of stress computed in your response, cracking will probably develop in the plain concrete lining. Please provide an assessment this anticipated cracking on the integrity of tunnel lining and structure. Also provide a consequence analysis of the tunnel structure assuming conservatively the collapse of some segment of concrete lining during an SSE and demonstrate that the structural function and integrity of the tunnel is maintained.

Cracking of the concrete is in itself not necessarily detrimental to the integrity of the tunnel lining. A tunnel lining is a very redundant structure owing to the interaction between the lining and the surrounding rock mass. Cracking, to some degree, is beneficial to the distribution of stress in a tunnel liner. Cracks allow the liner to deform slightly, the crown-invert dimension shortens while the spring-line dimension lengthens. This deformation releases the moments and associated tensile stress developed in the lining and allows the lining to carry the loads in compression for which concrete is more suitable.

The mechanical interlock between the irregular rock surface and the the cast-in-place concrete prevents the accumulation of longitudinal strains and consequently large cracks from developing. Cracks which do occur are generally well distributed.

The unlikely scenario of a segment of concrete lining collapsing into the tunnel will in any case have little effect on the function of the tunnels. The tunnel is self-supporting in a good quality shale. Thus, the loss of concrete lining in the crown serves only to open up the space to the rockline of the original machine-driven rock tunnel. The cross-sectional area of the tunnel is not reduced, therefore the flow quantity will be maintained. The roughness of the tunnel is increased due to the segment pieces lying in the invert and due to the lack of coherent lining in the crown. The velocity of flow is technically reduced by the increased roughness but practically the velocity remains nearly unchanged.

3. a) Provide information regarding the design basis for use of reinforced tunnel lining as well as unreinforced tunnel lining which were built in the United States.

   b) Provide an evaluation based on available data on the performance of unreinforced tunnel lining under earthquake and other pertinent loads.

a) In general, there is no simple basis for deciding to use reinforcement in a tunnel liner. The use of reinforcement in a concrete tunnel liner is based on a variety of considerations, the most important being the ground conditions surrounding the tunnel, the size of the tunnel, the shape of the tunnel, and the depth of the tunnel. In good quality rock some tunnels are left unlined.

Reinforcement is often found at tunnel intersections with other tunnels or shafts as in the case of the Perry tunnels. The unusual shape of the opening in these areas causes the stresses to be distributed unequally. Tunnels may show reinforced concrete at the portals. This is usually a result of hillside movements, not a consideration of ground support.

Attached is a list of 46 tunnels and typical statistics which are routinely recorded by the Bureau of Reclamation. Of the 46 tunnels on the list, only one shows any significant reinforcement in the lining.

b) The performance of tunnels under seismic loads is documented in STRAAM Engineers, Inc., "Design of Concrete Final Lining", November, 1979.

4. We reviewed your responses on the seismic analysis of tunnel structures, the following additional clarifications are needed for our final evaluation.

   a) Your basis for using 1957 Golden Gate earthquake, instead of R.G. 1.60 response spectra anchored to SSE as input motion.

   b) Your basis for selecting the shear modulus of water as 31000 psi.

   c) Your basis for selecting only six mode in the seismic analysis. What are the cumulative participation factors for the modes selected?

   d) The basis for use of 5% damping for both OBE and SSE.

   e) Why the difference in system stiffness and responses of the tunnel structure when different methods of analysis (i.e., Time Marching vs. Modal) are used.

   a) The time-history analysis was used only to obtain information that would help construct a model for extracting modes for a modal analysis and to provide a check on the modal analysis. It was not used for the final design analysis. In an effort to reduce the computer costs for the time-history analysis, the Golden Gate record was used since only about 3 seconds of the record was required to match the horizontal criteria spectrum of the safe Shutdown Earthquake between 4 and 20 Hz as required by Regulatory Guide 1.60.

The NRC requirement published in the 1975 Standard Review Plan is that the time-history must provide a spectrum that will match or exceed the criteria spectrum throughout the range of frequencies. The liner frequencies of concern are in the 8 to 17 Hz range and the range of 4 to 20 Hz was studied on this basis.

b) The properties of water were based on two assumptions, the bulk modulus of elasticity, K, for ordinary temperature and pressures is 300,000 psi (Streeter, V.L. and Wylie, E.B., Fluid Mechanics, 6th ed., McGraw-Hill Book Company, New York, 1975, p. 17), and Poisson's ratio, ν, is 0.45. The shear modulus, G, falls out of the relationship between these quantities

$$G = \frac{3\,(1-2\nu)\,K}{2\,(1+\nu)} = 31{,}000 \text{ psi}$$

c) The Rayleigh-Ritz approach was used to extract the modal shapes and associated frequencies of the tunnel liner under seismic loads. In using the Raleigh-Ritz method a set of prescribed force patterns is used as input data. The resulting displacements considered were independent sets and contained the lower mode shapes of the system. Twenty-four force patterns were used on the Perry tunnel liner analysis. The computations were terminated when six lower frequencies and mode shapes were obtained. Frequencies and mode shapes were limited to the lower range (< 20 Hz) of the criteria spectrum. The maximum frequency suggested by the Reg. Guide 1.60 criteria spectrum is 35 Hz. The stress contribution above 20 Hz frequency was neglected since the magnitude was small.

Of the six modes analyzed, the responses were combined when frequencies of the modes were within 10%. As directed by Reg. Guide 1.92 the absolute sum of the closely spaced modes were combined and the square root of these sums taken.

d) Damping used in the seismic computer analysis is taken as a function of the mass and stiffness matrices in the form:

$$[c] = \alpha\,[M] + B\,[K]$$

where x and B are related to the critical damping ratio by

$$\lambda_n = \frac{\alpha}{2\,W_n} + \frac{W_n}{2} = B$$

Since the predominant frequencies of earthquake acceleration records normally occur between 1 and 8 cps, values of α and B were selected for the first trial so that λn varied from 4 to 8 percent and averaged 5 percent.

e) The time-marching solution was not used as a basis for design. The time-marching solution provided information on the response of the tunnel lining to a specific earthquake record, the Golden Gate S80E component. This general information was used to develop the model for the modal analysis only. The details of the time-marching solution were not refined as was the modal solution and the criteria spectrum was not used as input. These basic inconsistencies and the fact that the programs themselves were developed on the basis of different assumptions would of course result in different responses of the tunnel structure. The magnitude of that difference was not considered inconsistent.

5. We need additional information and reference materials pertaining to your longitudinal tunnel analysis procedures. Also assess and discuss your analysis approach with respect to the applicable requirement of SRP Section 3.7.3.

The longitudinal analyses of the tunnel lining is discussed in "Stress Analysis of Perry Nuclear Power Plant, 10-ft Diameter Cooling and Emergency Service Water Tunnel System, Vol. II." by Agbabian Associates, July 1975. The appropriate sections are reproduced herein.

It is again noted that the dead load D, the grout load G, and the ground load H' are not included in these cases for the shaft analyses. For this reason, Case 1 is not a real loading condition. Also, it will be noted that the shaft stresses are equal to or lower than the tunnel stresses for all cases.

5. STRESSES PARALLEL TO THE LONGITUDINAL AXES OF TUNNELS AND SHAFTS

It is pointed out in Section 4 of Volume I that stresses produced by both the static and dynamic loads are of greatest significance in planes normal to the longitudinal axes of the tunnels. For this reason, detailed analyses have been provided in Volume I of typical transverse sections of the tunnels and extrapolated to the shafts in the previous section. Plane strain finite element models, which restrain deformations parallel to the longitudinal axes (z-direction) of the tunnels were used for these analyses. Therefore, except for one seismic load condition, the stress (σ_z) parallel to the longitudinal axes is given directly in the computer printout of stresses for the grout load G, the ground load H', and the temperature stresses T_o and T'. The σ_z stresses derived from these analyses are shown in Figures 10 through 16. The longitudinal stresses resulting from the dead load are small and have been neglected. Also, for the external and internal fluid pressures, the longitudinal stresses of the tunnel are low and approximately uniform circumferentially. Stresses of +20 psi tension result from an internal water pressure of 50 psi and, -60 psi compressive stresses result from an external water pressure of the same magnitude. Plots of stresses are not shown for the fluid pressure loads.

Stresses developed parallel to the longitudinal axes of the tunnels from seismic loads can be divided into two categories for presentation. First, the plane strain models used for the transverse modal analysis (see Figs. 41 and 42 of Volume I) indicate longitudinal stresses σ_z, which are provided in Figures 17 and 18 for the OBE and DBE, respectively. These stresses are of low intensity, 30 psi or less, and can be characterized as being stresses

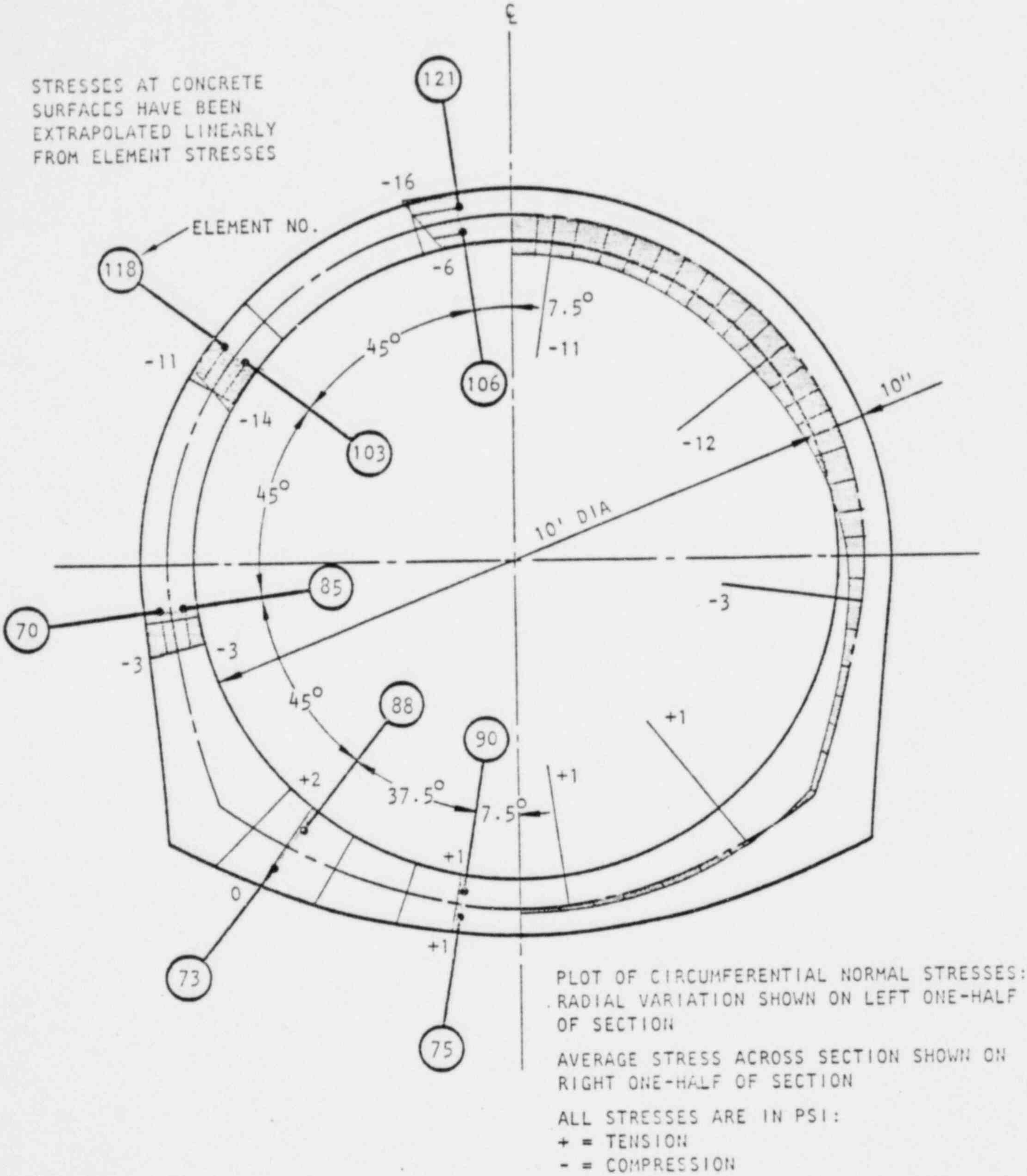


FIGURE 10. LONGITUDINAL STRESSES RESULTING IN PERMANENT LINING FROM GROUT LOAD G

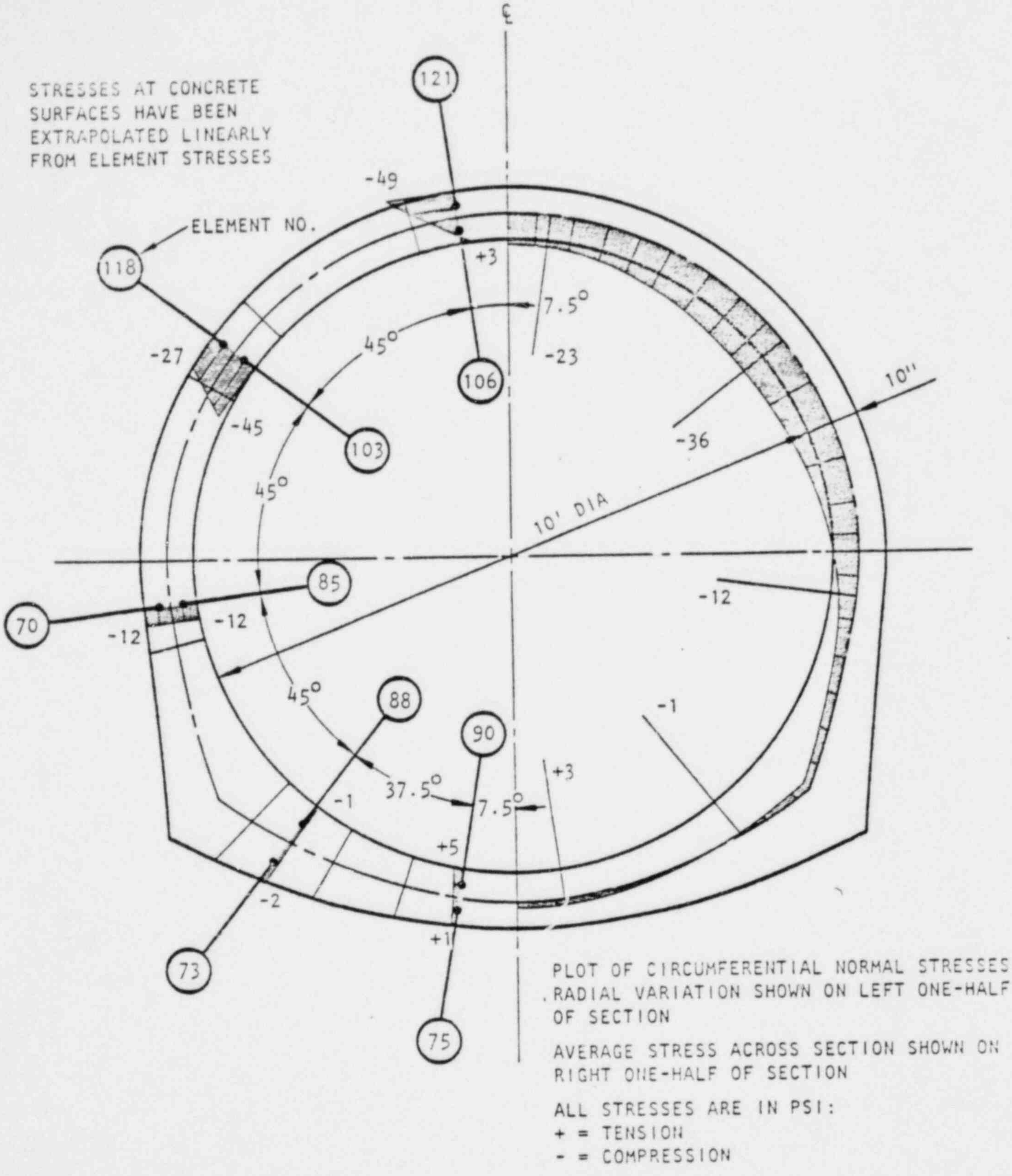


FIGURE 11. LONGITUDINAL STRESSES RESULTING IN PERMANENT LINING FROM GROUND LOAD H'

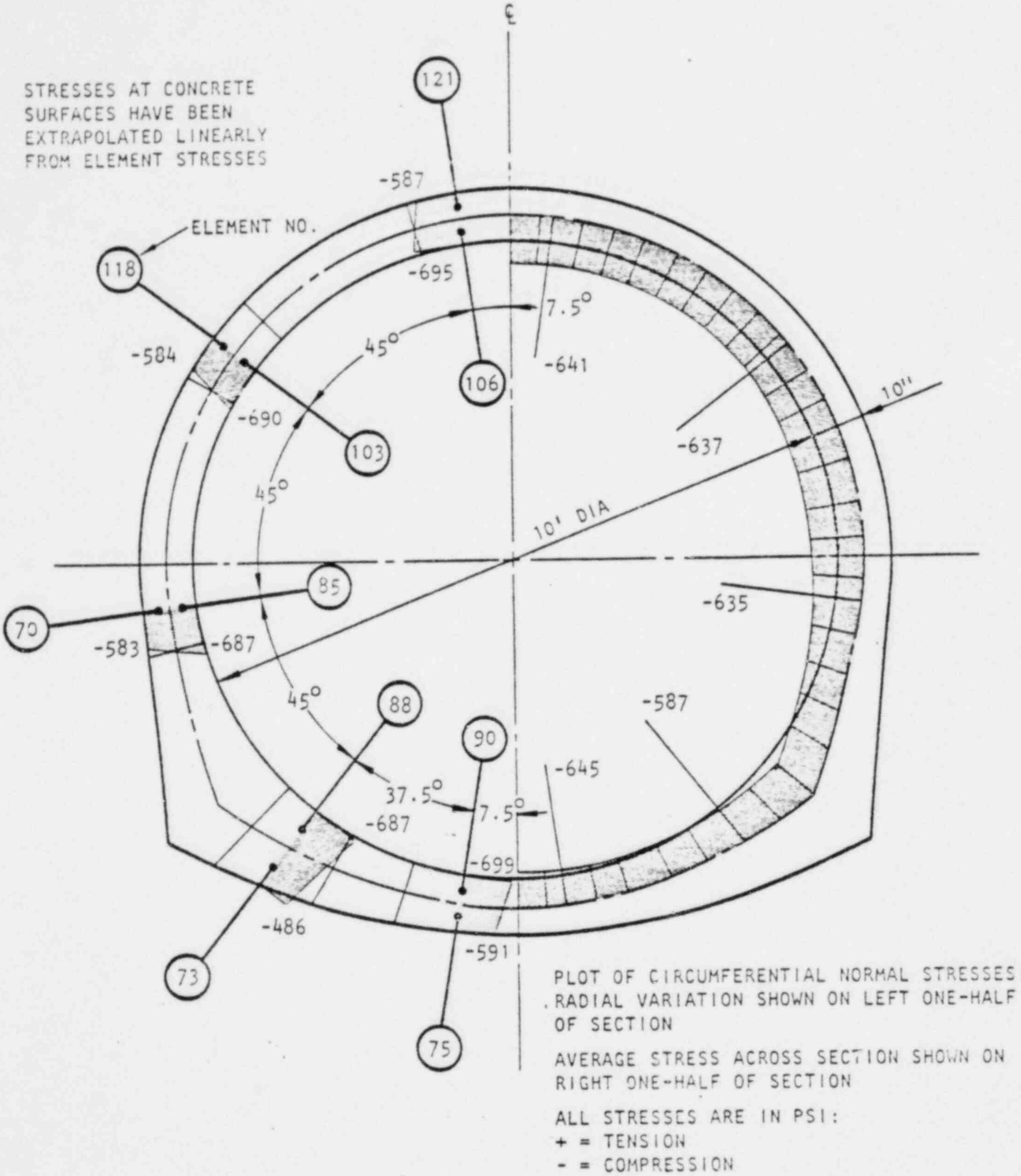


FIGURE 12. LONGITUDINAL STRESSES RESULTING IN PERMANENT LINING FROM SEASONAL TEMPERATURE RISE T_o' (T_o = +57°)

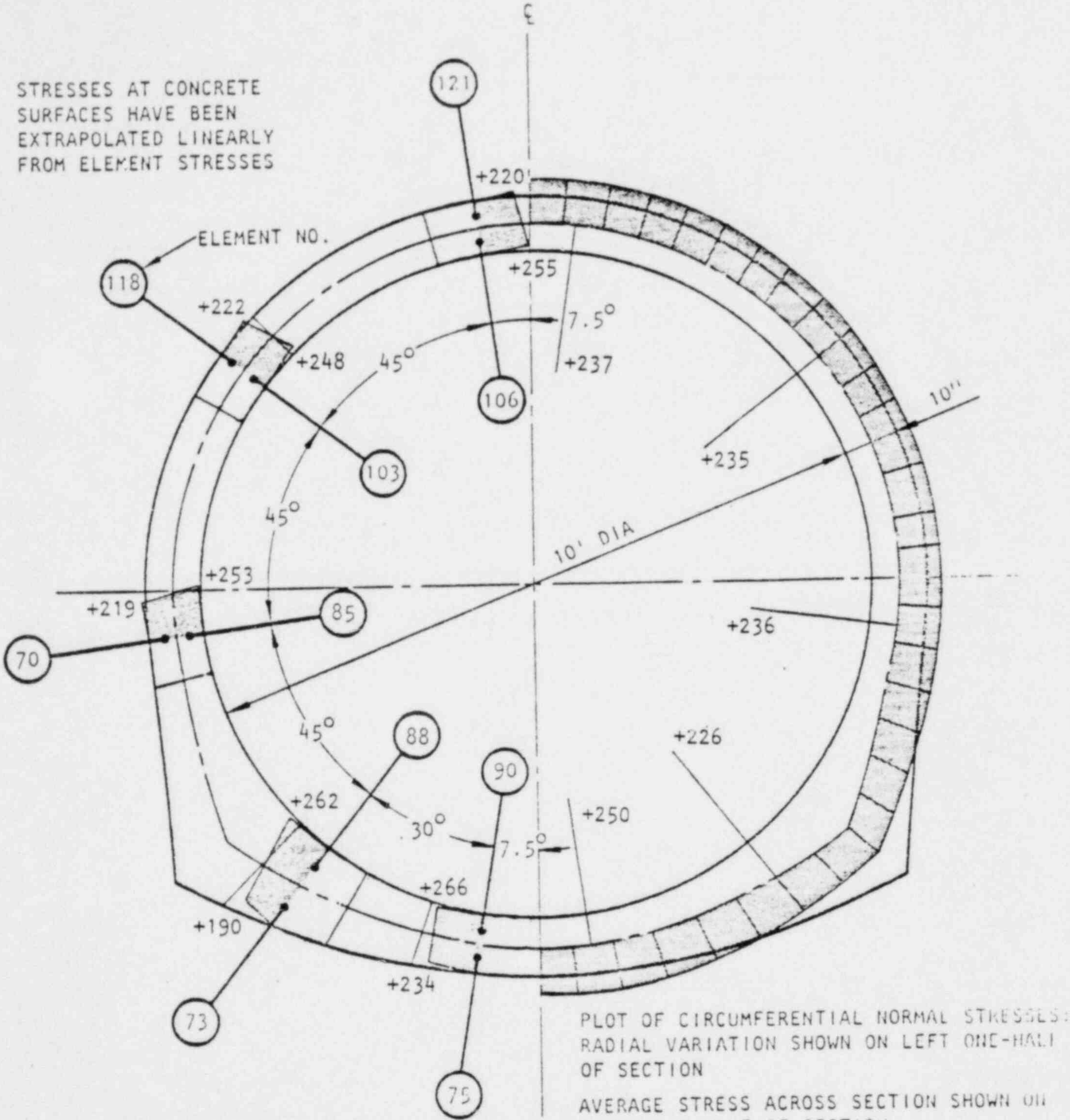


FIGURE 13. LONGITUDINAL STRESSES RESULTING IN PERMANENT LINING FROM SEASONAL TEMPERATURE DROP T_o (T_o = -23°) (WITH PARTIAL ROCK RESTRAINT)

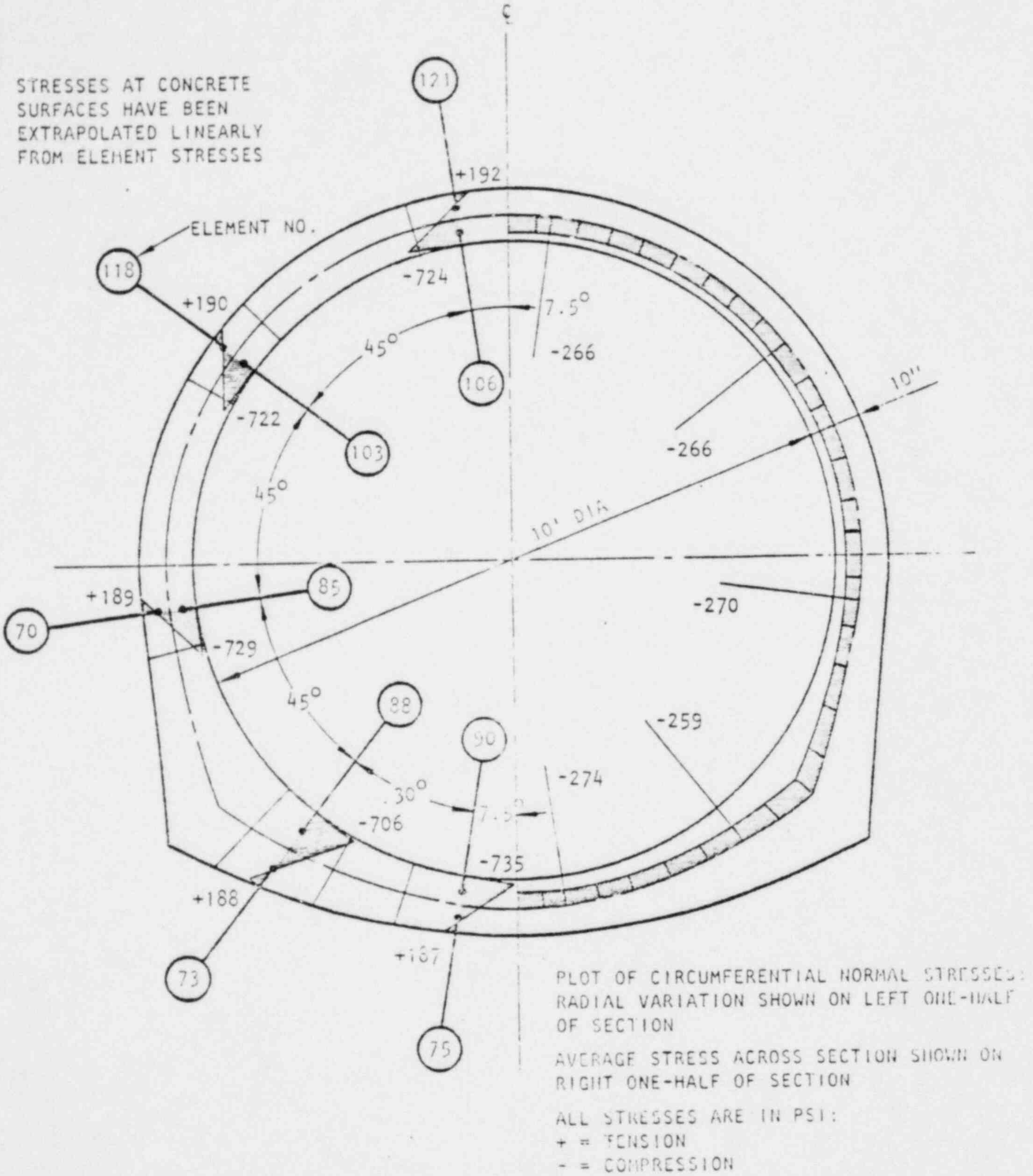


FIGURE 14. LONGITUDINAL STRESSES RESULTING IN PERMANENT LINING FROM RAPID TEMPERATURE RISE T' (T' = +57°)

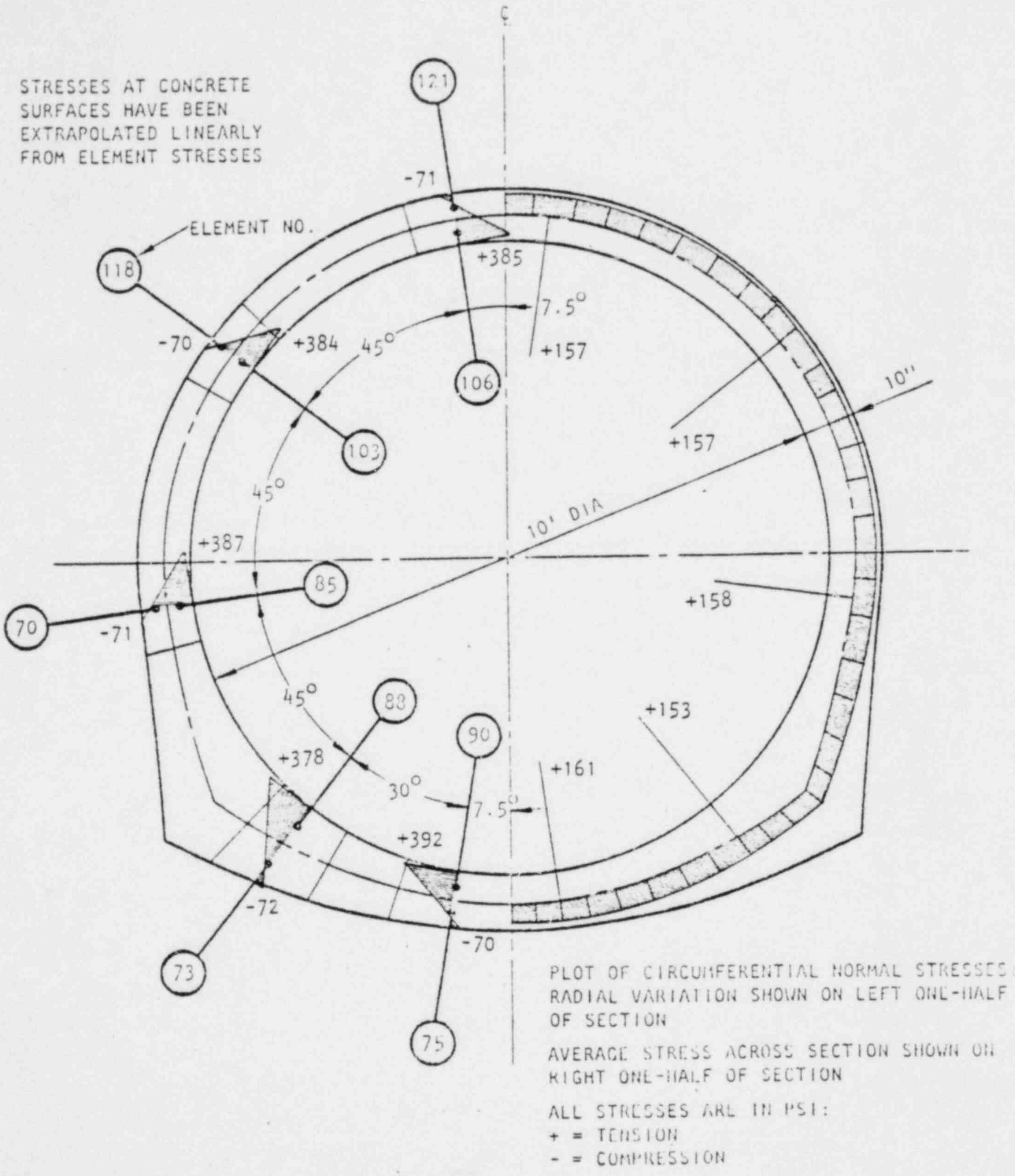
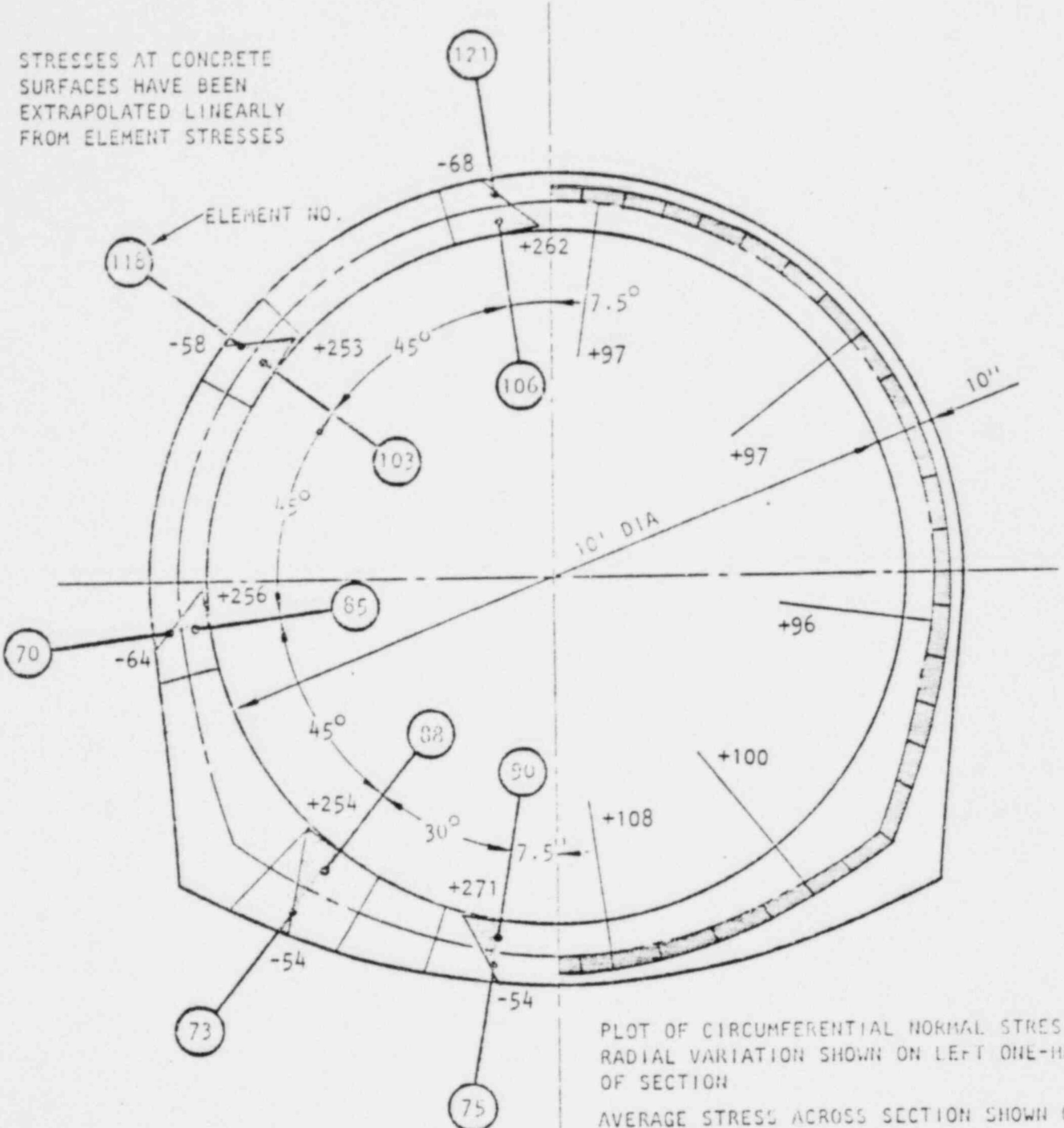


FIGURE 15. LONGITUDINAL STRESSES RESULTING IN PERMANENT LINING FROM RAPID TEMPERATURE DROP T' (T' = -23°) (WITH FULL ROCK RESTRAINT)



PLOT OF CIRCUMFERENTIAL NORMAL STRESSES:
RADIAL VARIATION SHOWN ON LEFT ONE-HALF OF SECTION

AVERAGE STRESS ACROSS SECTION SHOWN ON RIGHT ONE-HALF OF SECTION

ALL STRESSES ARE IN PSI:
+ = TENSION
- = COMPRESSION

FIGURE 16. LONGITUDINAL STRESSES RESULTING IN PERMANENT LINING FROM RAPID TEMPERATURE DROP T' (T' = 32°) (WITH PARTIAL ROCK RESTRAINT)

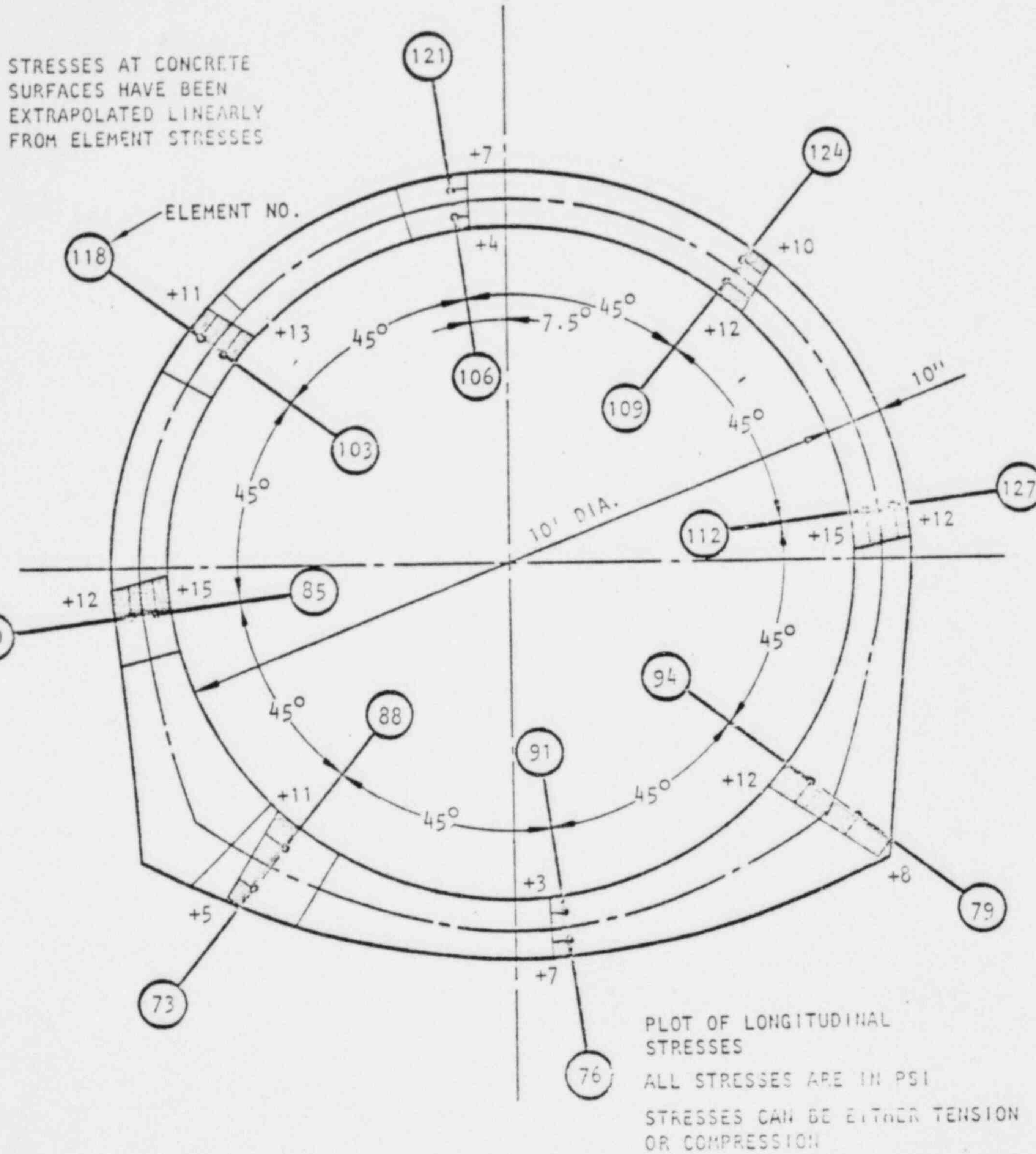


FIGURE 17. LONGITUDINAL STRESSES IN PERMANENT LINING FOR OPERATING BASIS EARTHQUAKE RESULTING FROM TRANSVERSE MODAL ANALYSIS

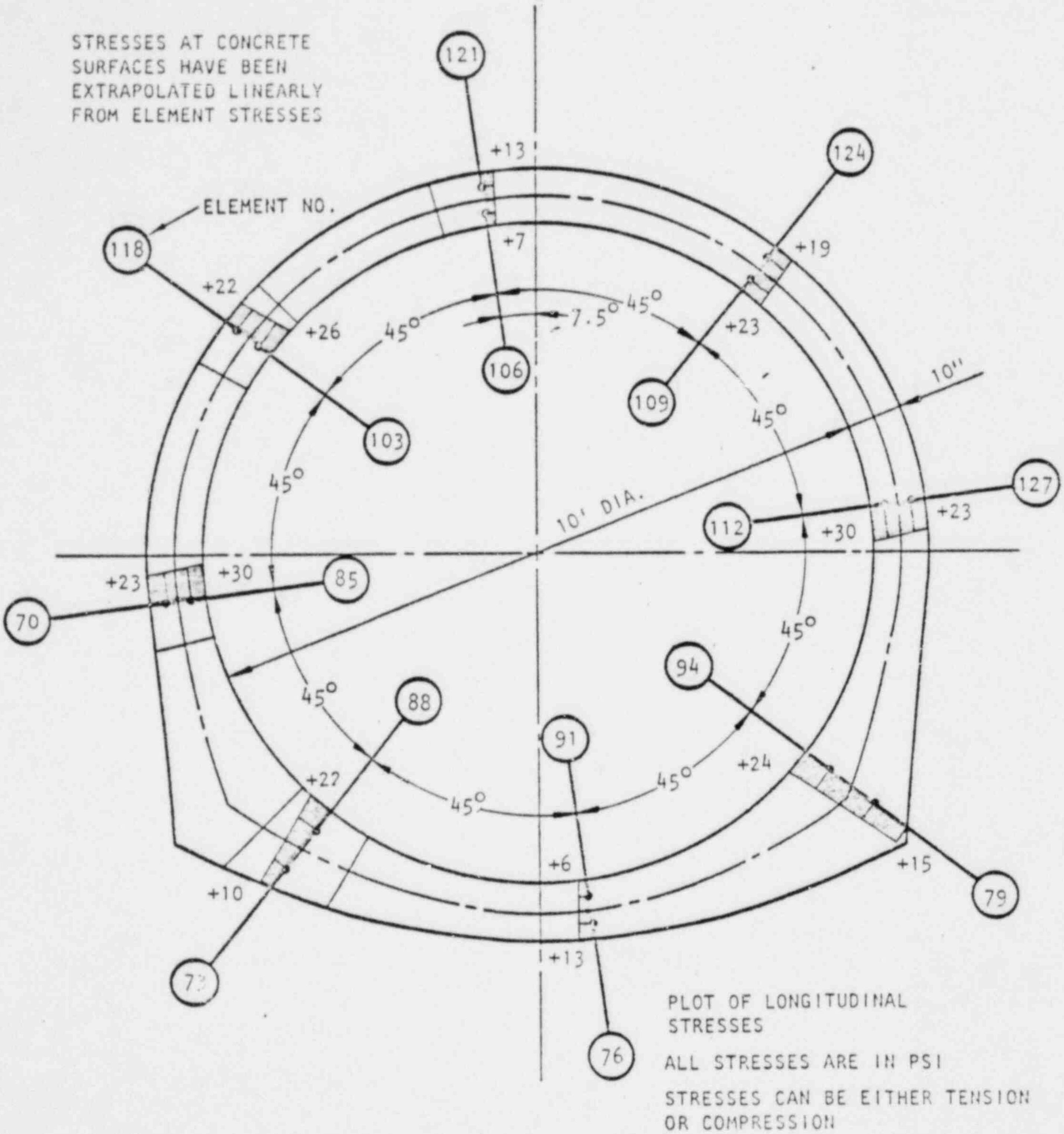


FIGURE 18. LONGITUDINAL STRESSES IN PERMANENT LINING FOR SAFE SHUTDOWN EARTHQUAKE RESULTING FROM TRANSVERSE MODAL ANALYSIS

that result from the Poisson effect of seismic loads resisted by the tunnel in the transverse direction due to circumferential loading of the tunnel. Additional stress, or a second category of longitudinal seismic stress, results from the fact that the tunnel section is translated laterally and deformed longitudinally as the earthquake stress waves engulf the tunnels and pass through the shale surrounding the tunnels. Additional discussion of the phenomenology associated with this second category of stress is required and follows in the next paragraphs.

Stresses resulting parallel to the longitudinal axes of the tunnels from translation and deformation of the tunnel axes were calculated by the approach given by Keusel (Ref. 1). In this method the strain in the longitudinal direction of the tunnel induced by an earthquake stress wave having a wave length L, and a single wave amplitude A, is given by:

$$\varepsilon = \varepsilon_s + \varepsilon_b = \frac{2\pi A}{L} \sin\psi \cos\psi + \frac{2\pi^2 AW \cos^3\psi}{L^2}$$

Here,

ε_s = Axial strain

ε_b = Bending strain

W = Depth or width of structure

ψ = Angle between the direction of wave propagation and longitudinal axis

This equation shows that when $\psi = 45^\circ$, ε_s reaches a maximum value and when $\psi = 0^\circ$, the bending strain ε_b is a maximum. In addition, it indicates that for a structure having a width W much smaller than the wave length L, the axial strain is predominant.

To solve this equation, the relationship of the length L and the amplitude A of an earthquake wave traveling in the medium in which the tunnel is located must be determined. Relationships between A and L for an earthquake wave traveling in sand and clay media are given by Kuesel and are shown in Figure 19. For Chagrin shale, the relationship between wave amplitude and wave length has been estimated based on the A versus L curves for clay and sand, and from the moduli of the three materials (Chagrin shale, clay, and sand). The A versus L and A/L versus L curves estimated for Chagrin shale are given in Figure 20.

Having determined the A and L relationship of earthquakes in the Chagrin shale, the seismic stress in the longitudinal direction was computed in the following manner:

From Figure 20,

$$(A/L)_{max} = 19 \times 10^{-6} \text{ at } L = 9000 \text{ ft}$$

Since the wave length, L, is much greater than the tunnel width, W, which is only 10 ft, only the first term in the equation is important and the equation may be reduced to

$$\varepsilon \doteq \varepsilon_s = \frac{2\pi A}{L} \sin \psi \cos \psi$$

From this equation, it is evident that the strain ε_s reaches a maximum when $\psi = 45^\circ$ and is equal to the following:

$$(\varepsilon_s)_{max} = \frac{2\pi A}{L} \sin 45^\circ \cos 45^\circ = \frac{\pi A}{L}$$

Substituting the $(A/L)_{max}$ value

$$(\varepsilon_s)_{max} = 19 \times 10^{-6} \times 3.1416 = 59.7 \times 10^{-6}$$

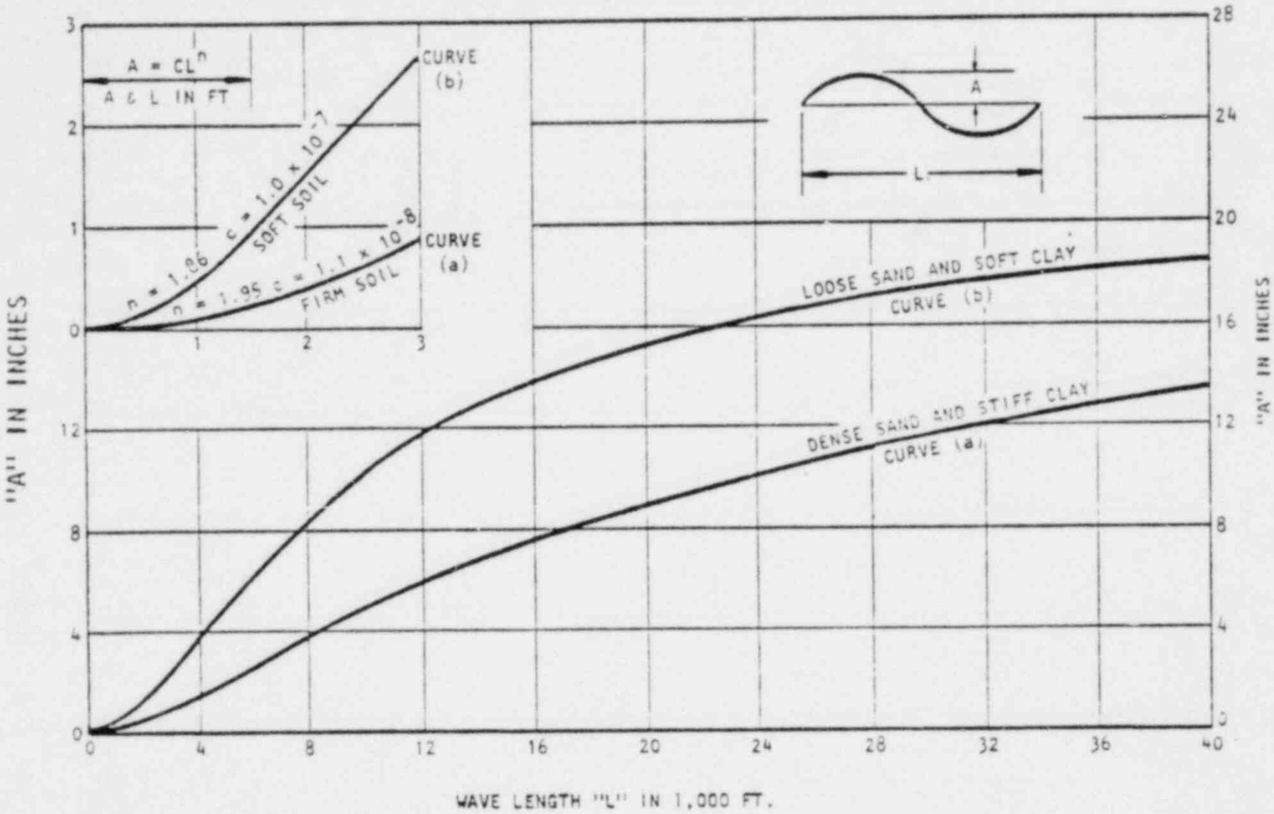


FIGURE 19. TRANSVERSE GROUND DISPLACEMENT SPECTRUM (FROM REF. 1)

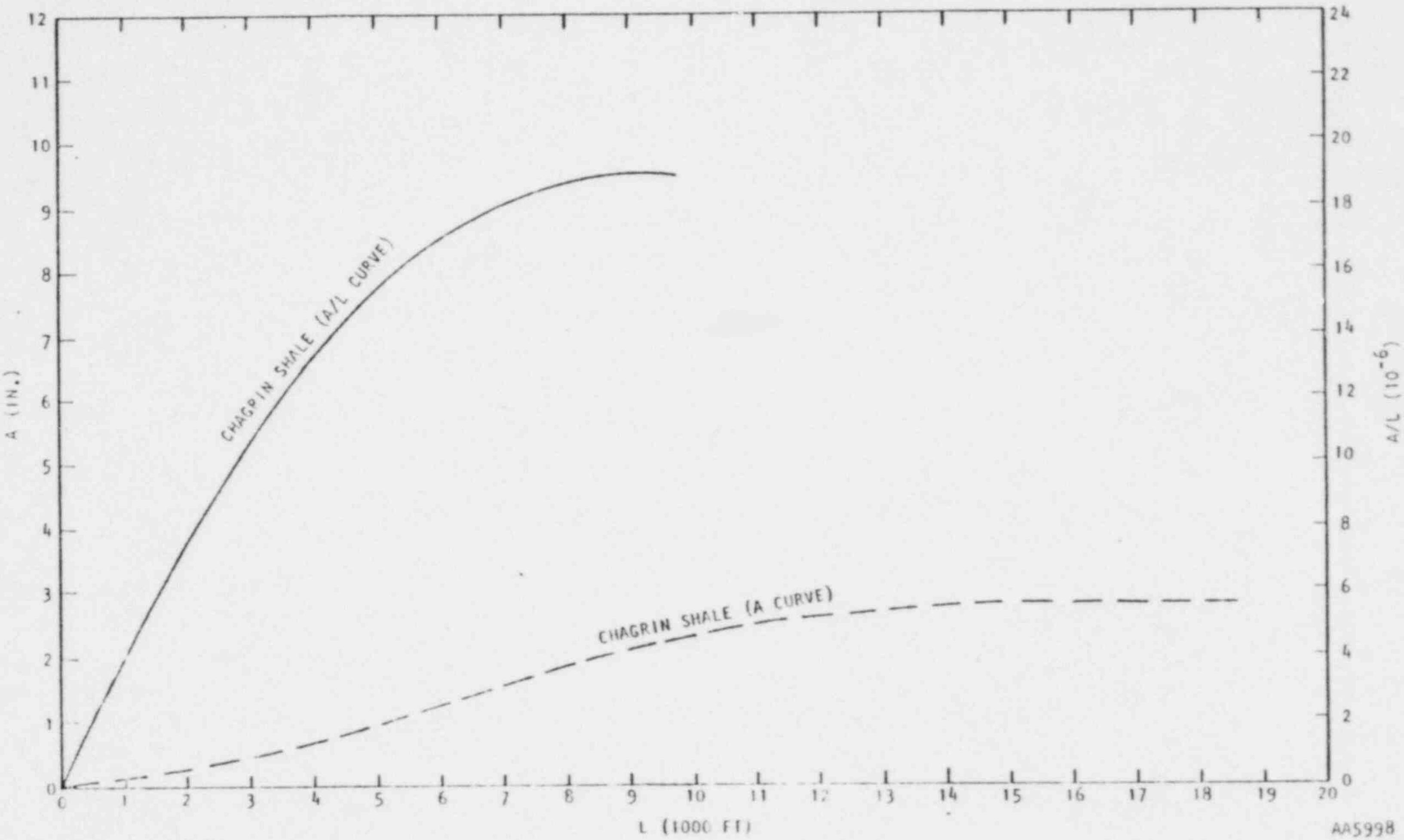


FIGURE 20. TRANSVERSE GROUND DISPLACEMENT SPECTRUM FOR CHAGRIN SHALE

The maximum strain ε_s can be either tensile or compressive, and the equivalent stress in the tunnel lining is given by:

$$(s)_{max} = E(\varepsilon_s)_{max} = \pm 215 \text{ psi}$$

which is uniformly distributed throughout the entire section. In the above calculation the short term modulus of concrete was used. This value of stress is assumed to apply to the SSE condition. One-half of this value, $\pm$110 psi is assumed for the OBE.

The maximum bending strain, ε_b, of the tunnel can be demonstrated to be insignificant in the following manner:

For $\psi = 0^{\circ}$

$$(\varepsilon_b)_{max} = \frac{2\pi^2 AW \cos^3 \psi}{L^2} = \frac{2\pi^2 AW}{L^2}$$

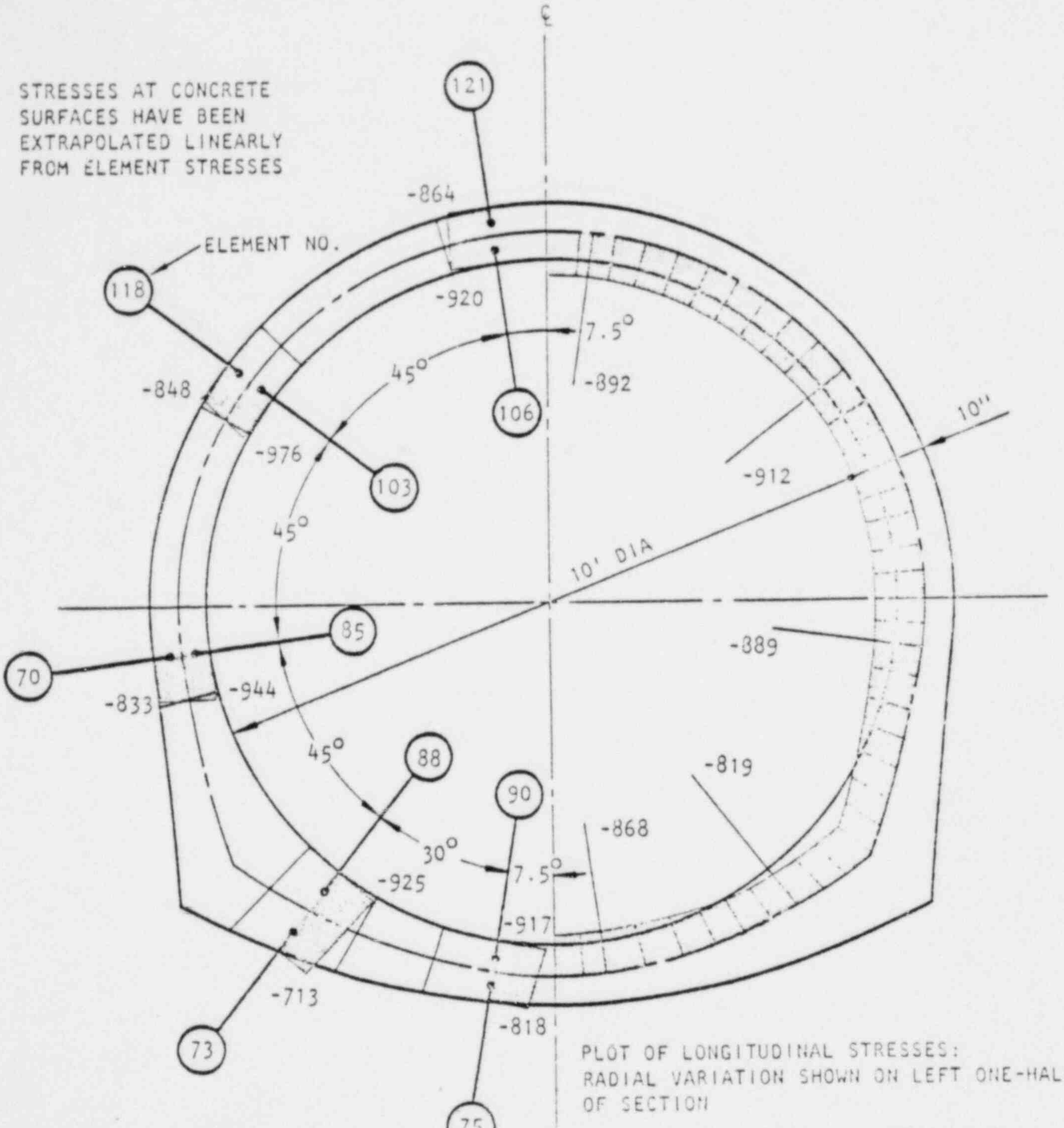
Substituting in this equation the A/L, and L values found above gives the following for W = 10 ft.

$$(\varepsilon_b)_{max} = 0.42 \times 10^{-6}$$

This is the maximum value of bending stress that could develop for the same A/L, but it does not develop simultaneously with the maximum value of ε_s as a different value of ψ has been used in the two equations. Since this value is only $\pm$2 psi, it can be neglected. It should be noted that an absolute maximum for ε_b occurs for a different value of A/L than considered above (i.e., A/L^2 should be a maximum). However, in all cases ε_b is an order of magnitude lower than ε_{smax} and can be neglected.

Comparison of the longitudinal stresses given in Figures 10 and 11 for the grout load G and the ground load H' with the stresses resulting from the fluid pressures indicates that the longitudinal static load stresses are quite low and vary from tensile stresses of less than +50 psi to compressive stresses of about -100 psi. This statement neglects temperature stress effects. The temperature and seismic stresses, however, are sufficient to cause transverse cracking. The designer is, therefore, faced with four alternatives. Recognizing that transverse temperature cracks will result in a slight local increase in stress in the transverse section, but will not impair the load-carrying capacity of the tunnel, three of the alternatives involve relieving the longitudinal temperature stresses through the use of transverse joints or cracks. These alternatives are (1) provide transverse construction joints at regular intervals along the longitudinal axes of the tunnels; (2) provide control joints rather than construction joints; and (3) provide no joints but let transverse cracks develop at random. The fourth alternative is to control the temperature cracks by the use of longitudinal temperature reinforcing steel. These alternatives are prerogatives of the designer. If the longitudinal stresses are to be carried by reinforcing steel, the stresses that must be considered are given in Figures 21 through 27 for the seven loading conditions considered in this analysis.

Since the dead load, grout load, and ground loads do not act as transverse loads on the shafts, stresses in the longitudinal direction of the shafts from static loads are less than estimated for the tunnels. It is conservative, and recommended that the same stress conditions be assumed in design of the shafts as computed for the tunnels.



SAFE SHUTDOWN EARTHQUAKE CONDITION:
$U_c = 1.00D + 1.00H' + 1.00T_o + 1.00E'$

ALL STRESSES ARE IN PSI:
+ = TENSION
- = [illegible]

FIGURE 27. LONGITUDINAL STRESSES RESULTING FROM CASE 7 LOADS