TENDONS

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1 ANCHORAGE ZONE DESIGN FOR PRESTRESSING TENDONS

A design for concrete and reinforcing steel in the Anchorage Zones for prestressing tendons considers two main problem groups:

(1) A prediction of the bearing stresses under the anchor bearing plate by methods which were used, along with experimentation, to set the allowable stresses under the bearing plates for prestressing tendons. In this case the allowable bearing stresses are set by ACI-318-63 and were set under the assumption that certain analytical methods would be used.

(2) A prediction of the transverse tensile (bursting) forces and a specifying of the appropriate location and amount of reinforcing steel to resist the tensile forces caused by the anchors alone or combined with those caused by the most unfavorable load conditions other than those resulting from anchorage of the tendons. |8

Problem Group 2 is the one which this Section discusses. The main problems are not related to whether the tendon is small or large and, therefore, they have been investigated, in the design of containments which use small tendons in sufficient detail to provide reasonable assurance that the designs are sufficiently conservative. It is possible, however, that there will be some small differences between the anchorage details of the large and small tendons which will require re-examination in this design. For these, it is anticipated that the same methods will be used which were used in previous designs.

The design approach and methods are briefly described as follows:

1.1 DESIGN APPROACH

The tendons are considered major strength elements of a Class 1 structure. Should gross failures occur in either the tendon end anchor assembly or the anchorage zone, the tendon would be rendered ineffective. Hence reasonable assurance must be provided that such failures are improbable. Tendon and anchor assemblies are discussed in Section 2 of this Supplement and this section discusses the reinforced concrete anchorage zone which is affected by the end anchor loads as well as the reaction to other load conditions.

There are three main questions which require answers in design. The first is, what reinforcement is required assuming that the anchorage concrete is affected only by the end anchor? The second is, what reinforcement is required for the structure in the absence of the end anchor but where the prestressing and other loads are idealized? The third question is what are the interaction effects between 1 and 2 and is more reinforcement required because of the interaction effects? The 1st and 3rd questions must be considered at some point in time since each condition described in those questions occurs at one time or another in the structure lifetime. However, only Question 3 is relevant to this discussion since it has a bearing on structural integrity during service use of the containment.

The basic approach followed is to provide reinforcement for those areas where predicted tensions could result in anchorage failure. There are uncertainties in any one method of analysis, therefore several methods are used and the design is prepared so that reinforcement is provided for the predictions of the various methods.

This approach has been shown suitable by tests described in Reference 1 and 2. In fact, the methods used can be shown to be conservative when compared to the small amount of reinforcement at anchorage zones as used for the model described and the resistance to anchor zone failure demonstrated by the model tests.

Briefly summarized, the model was purposely extensively cracked so as to create failures that could conceivably occur. One such failure was the failure of anchorages. Ref. 2 shows pictures taken while the cracks existed and the distension of the model could only result in yielding of the tendons in certain critical areas. Anchorage zones were limited in extent by cracks as large as 1" wide around the anchor blocks. Despite these large cracks and yielding of adjacent reinforcing steel, anchorage zone concrete continued to sustain the loads imposed by the end anchors.

No claim is made here that the anchor blocks for the model were in all respects identical to those to be used for this containment. However, for the average loads imposed by the anchor blocks on the model concrete, it was demonstrated that lesser amounts of reinforcing steel, than provided by the containment design, were sufficient to preclude failure of the anchorage zones.

Since the model tests cannot be related directly to the containment designs, the practice of added conservatism is followed by providing reinforcing for the combination of adverse predictions which result from the methods described.

Where tensile reinforcing cannot be arranged to follow the direction of principal tensile stresses, the relevant shear forces or stresses will be considered in the design. |8

1.2 ANALYTICAL METHODS

Four independent methods or combinations are used in predicting the anchorage stresses. The bearing stress next to the bearing plates and the transverse tensile stresses in the bursting zone are checked by the Guyon and Taylor methods. Both these methods and the plain strain finite element program (the 3rd method) reduce the three dimensional stress condition into a two dimensional one, however the three dimensional effect is also considered, especially near the base slab and ring girder, by the fourth method.

Predicted stresses, not originated by the anchorage forces, are small compared to localized stresses immediately under the anchorage. Next to the bearing plate the concrete is predicted to be in compression in all three dimensions. Tensile cracks cannot penetrate this region. Further away from this highly compressed zone, in the so-called bursting zone, compressive stresses parallel to the tendons and tensile stresses in the hoop

(relative to the tendon centerline) direction, will exist. These tensile stresses will always be reinforced for by the kind of a reinforcement found applicable for the given detailed conditions described below:

There are three distinctly different classes of stress conditions predicted for the regions of the tendon anchorages of the containment structure.

(a) Hoop tendon anchorages in the vertical buttresses.

(b) Vertical tendon anchorages.

(c) Dome tendon anchorages in the ring girder.

1.2.1 BUTTRESS STRESSES

In analyzing the buttress anchorage stresses, the plane strain finite element program is used, since in the vertical direction the predicted strains are almost constant. The strain corresponding to the average stresses could be considered, but the localized peak stresses have only a small effect on the average before a fixed value of zero vertical strain, as assumed in the plane strain method, results in a close approximation of the actual condition. 8

The analyses developed by Guyon and Taylor are also used in these regions. The three dimensional conditions are also considered particularly at the base slab and ring girder.

Several combinations of load cases will be included in the analyses along with the effect of end anchor forces:

(a) D + F
(b) D + F + P + T
(c) D + F + 1.5 P 8
(d) D + F + 1.5 P + T

For each of these conditions analyses are made by the methods mentioned previously. Reinforcement is then provided for the adverse conditions as predicted by the various methods. This most probably results in more reinforcement than would actually be needed, however it is provided as a design conservatism. The reinforcement in general consists of radial ties between the buttress and the cylinder wall; horizontal surface buttress reinforcement additional to that required for other loads and to control any vertical cracking tendencies between the buttress and the concrete wall; and vertical reinforcement for the buttress surface. Some secondary tension forces are expected to cause strain rates to differ between the buttress and cylinder wall. The differing strain rates are considered beneficial since they will allow the buttress stresses to be controlled primarily by the average compression in the hoop direction as caused by the tendon end anchors.

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1.2.2 VERTICAL TENDON ANCHORAGES

1.2.2.1 Anchorages at the Top of the Ring Girder

Several combinations are considered more important, all of which include the local effects of tendon end anchorages: They are

(a) D + F
(b) D + F + 1.5 P
(c) D + F + 1.5 P + T
(d) D + F + T + P |8

For (a) stresses are found to be compressive except the bursting stresses in the meridional direction. For (b) the condition is but slightly altered |8 and the bursting stresses are similar to (a). For (c), tension forces are predicted in the radial direction. For (d), no significant thermal stresses are predicted to add to the condition for (a).

1.2.2.2 Anchorage at the Base Slab

These are similar to the ones described in 1.2.2.1. Heavier reinforcement is placed in the hoop and meridional direction however, since this area is circumferentially prestressed less than at the top.

1.3 DOME TENDON ANCHORAGES

The dome tendons are not perpendicular to the cylindrical surface. This condition is provided for however by spiral reinforcement which is additional to the main reinforcement and confines the volume most affected by bursting stresses, and by surface reinforcement for crack control. |8

The load conditions considered important in addition to end anchor stresses are:

(a) D + F
(b) D + F + 1.5 P
(c) D + F + 1.5 P + T
(d) D + F + T + P |8

In each combination, the various analytical methods are used and appropriate reinforcement provided.

1.4 CONCLUSION

The problems of anchor zones are complex. They are analyzed by several methods and reinforcement is provided for the adverse conditions predicted by each. Such design procedures are believed to result in conservatism when compared to the experiences and experimental results available. The predictions and reinforcements are reviewed by Bechtel Consultants.

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REFERENCES

1. Prestressed Concrete Reactor Vessel Model 1 by HTGR Laboratory Staff GA 7097

2. Prestressed Concrete Reactor Vessel Model 1 Supplemental Report by HTGR Laboratory Staff GA 8024

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2 LARGE SIZE PRESTRESSING TENDONS

Large size tendons will be used for this containment. Their ultimate strengths will be between 1,000,000 and 2,000,000 lbs. as compared to the 1,000,000 lbs. size now in use. Their usage is similar to that of the smaller size tendons. There are, however, differences of detail that will be given consideration in the selection of the tendons during the detailed design of the containment.

The differences, between the large and small size tendons, have been investigated in the detail necessary to determine that the problems for the large size tendons are not major, do not differ significantly from those of small size tendons, and that there are advantages in the use of larger size tendons. However, in the selection of the actual tendons to be used, the following items will be considered and the findings recorded.

2.1 PRESTRESSING TENDON CONSIDERATIONS

The tendons are major strength components of a Class 1 structure. Their selection and usage must therefore be done such that there is assurance that the tendons will satisfy the general design criteria for the containment structure (which includes relevant ACI and Prestressed Concrete Institute Codes and specifications) and the performance criteria from Supplement 5. The tendon and system selection and utilization criteria includes requirements that consideration be given to analytically and, or experimentally determining the following:

2.1.1 CONSERVATIVE RELAXATION LOSS VALUES

Providing assurance of conservative relaxation loss provisions is actually related to other interrelated prestressing losses provided for as well as to other design features. For clarity however, only relaxation loss values will be discussed here.

Steel relaxation losses can range from less than 5% to more than 14% both as extrapolated to 30 years from 1000 hour tests and based on the initial prestress load resulting from an axial prestressing steel stress of 0.7f's. (Ref. 2.1 and 2.2) The major determining factor in establishing the loss to be provided for is the source of prestressing steel supply. In effect the source of supply establishes the subrange, within the range of 0 to more than 14%, to be provided for.

The subrange will be determined from tests records of laboratory experiments on steel as produced by the prestressing steel supplier for the containment. Since this subrange is derived by laboratory experiments on single wire specimens, it can be seen that it is independent of the tendon size.

It has been hypothesized that the differences between tendon and laboratory load conditions may result in relaxation greater than predicted by the use of laboratory test results. Tests (Ref. 1) have shown that the increases may be large when compared to average laboratory test results and when steel

in the low end of the relaxation range is used. It is also apparent that the differences reported are within the laboratory test scatter since the increase, if any, in relaxation was 1% of the initial prestress load when tests on straight tendons were compared with tests on curved tendons with 5 curves of less than 10 ft. radius. The qualifying phrase "may be" is used since the increases in curved tendon relaxation have not been verified by other tests and may be the result of problems similar to those which cause scatter in the laboratory data.

Other tests (Ref. 3 and 4) have been made for large size tendons (2,000,000 lb. ultimate strength). The final results have not as yet been reported by the Atomic Energy Commission subcontractor but it is understood that the report is in final editing. It is however unlikely that the increases in relaxation losses, if any, for curved as compared to straight tendons, will exceed the scatter of laboratory test results even though the curvature radii are about 20 feet.

2.1.2 CONSERVATIVE FRICTION LOSSES

As for relaxation loss provisions, the friction losses must be considered in the total context of the provisions for other losses and the containment design features. It is discussed separately here for clarity.

Tests reviewed and reported in the preparation of Appendix J show that the friction factor can be determined with sufficient confidence for design. It will be a design objective to assure, as required by ACI 318-63, that the friction factors for the structure are not greater than those provided for in the design.

Assurance that the friction factors do not exceed those provided for is gained by installing the tendons after concreting so that cement grout leakage, if any, can be detected and corrected. Assurance is also provided by following the provisions of ACI-318-63 and verifying, during stressing, that the friction factors used in the design are indeed within the range provided for by the design.

It is considered important that, during stressing, the friction factors used in the design are verified as suitably conservative. The reason is of course that field conditions can be worse than test conditions. Some provisions for this are made by suitably increasing the friction factors as determined by tests. The worst historical friction factors cannot, however be provided for in the design since they, combined with conservatism in relaxation could result in overprestressing the concrete if lower friction factors are not detected by verification during stressing operations. Further, the occurrence of the worst historical friction factors is so unlikely that it is best to detect and correct the situation, if it exists, rather than to arbitrarily provide for excessive friction factor.

The test friction factors are used with the exponential formula given by ACI-318-63 instead of the straight line formula, that may be used if the curvature friction factor is less than 0.3. This practice allows some conservatism in the design. Fig. 2.1 however, shows that the difference

between a straight line and the exponential curve is not large, a factor which is evident from the equations involved. Should the curvature or wobble coefficients be smaller, the curve of course would even more closely approximate a straight line.

Fig. 2.1 shows the effect of two end jacking of a tendon with 240 degrees of central angle which is the approximate equivalent of one end jacking for a tendon with 120 degrees of central angle. Appendix J describes tests which included one end jacking of tendons with that angle. Further tests planned (Ref. 3 and 4) have been made for large size tendons that had an included central angle of curvature of 180 degrees and radii of about 20 feet. The test results have not yet been completely reported so cannot be directly cited here. However, the indications are that the results were similar to the results reported in Appendix J .

Fig. 2.1 does not show the effects of penetrations which of course can increase the total angle of curvature. However this factor, and the establishment of a conservative, experimentally derived friction factors will be considered in this design as it has been for the smaller size tendons.

2.1.3 REALISTIC VALUES FOR TENDON EFFICIENCY AS A FUNCTION OF CURVATURE

The real measure of efficiency is found in economics, the details of which are not a subject for this discussion. There are two other measures of efficiency affected by curvature. One is based on the reduction in tendon ultimate load capability and the other based on friction for curved tendons as compared to straight tendons. Both efficiencies will be investigated during the design to provide assurance that the design criteria are met, a common part of the design procedures. To make the detailed study, final design work is required.

2.1.4 BRITTLE FRACTURE CHARACTERISTICS

Brittle fracture has not been found to be a problem as stated in Supplement 5, pages 5.9 to 5.11. However, this consideration requires reassessment for each prestressing system as compared to site needs and will not be neglected.

The characteristics will therefore be examined in the selection and use of the prestressing system for this containment.

2.1.5 PRESTRESSING STEEL COMPONENT REDUNDANCY

A more complete investigation of the redundancy for this design will be made when the prestressing system is selected. Initial investigations show that the redundancy, under the more important conditions, increases when the number of buttresses decreases.

This effect results primarily from the increased number of prestressing components per tendon and is caused by the increased friction for the 3 buttress configuration.

2.2 PRESTRESSING TENDON COMPONENTS

Subsection 2.1 concentrated attention on the tendon because the component, considered in the context of its usage, is more important than when components are considered as isolated cases out of context.

However, components have to be considered as isolated cases so as to provide assurance that the components which make up the tendons (evaluated as described in subsection 1) are representative of the components to be manufactured and used in making up tendons for the containment.

The components are not named here since the names vary from prestressing system to prestressing system.

However, the intent is to include all components which are loaded as the force is carried from the end of the prestressing steel components through the end anchor and load adjusting devices, to and through the devices which distribute the end anchor load into the concrete. For wire tendons this would include the wire and buttonheads, the end anchor and any devices which maintain the desired end anchor distance from the concrete surface of the interface to which the end anchor load is delivered.

2.2.1 BRITTLE FRACTURE CHARACTERISTICS

The brittle fracture characteristics will be assessed for test and load conditions which are representative of those found when used for containment tendons.

2.2.2 CAPABILITY OF TENDON COMPONENTS TO DEVELOP THE ULTIMATE STRENGTH OF THE TENDON

The criteria require that the tendons have a known ultimate strength and tendon experiments are required to show that this is the case. The tendons must however, be made from representative components and that representativeness must be determined by examining the critical components and insuring that they are considered in the components used in the tendon tests.

For the tendon system selected, assurance will be provided that the components used in experiments can be correlated with components to be produced for the containment tendons. This will be done by analyses which define the critical production tolerances* and material characteristics and assuring that they are sufficiently covered in the tendon tests and any component tests. All materials shall be identified, to the extent necessary for the specific application, by conformance to ASTM specifications, to industry standards of similar detail, or by measured chemical and physical properties necessary to define the materials to a degree equivalent to that given by such specifications or standards. 8

*Production tolerances includes manufacturing tolerances, field erection tolerances, and alignment conditions for the tendons as installed in the structure. 8

It is considered that such ultimate testing of the tendon hardware does in fact give sufficient assurance that the hardware will develop the strength of the wire. However, for this project, an attempt will be made to analytically determine deflections of the tendon hardware, bearing plate, and concrete for the test conditions. Such predictions will then be compared with actual deflections measured during the test. The analysis will be done by axisymmetric finite element techniques to allow the consideration of non-linear stress-strain properties for both the concrete and steel. No attempt will be made to consider the stress concentrations within the stressing washer due to the buttonheads or the wire holes. 8

Upon completion of the tendon anchorage qualification test, a report will be prepared and submitted for information. This report will include a description of the analytical methods, material properties used, results of the analysis results of the test, and a discussion of the comparisons.

4.3 DESIGN BASIS

The design of a prestressed concrete containment includes matching the capabilities of a prestressing system to the needs of the structure. It also includes determining that the reaction of the structure to prestressing and other loads does not cause the prestressing tendons to be exposed to conditions for which their capability cannot be reasonably proven. In essence, neither the structure nor its prestressing system should be considered out of context with the other.

To assure a proper mating of structure and prestressing system, the work will consider the reaction of the structure to combinations of unfavorable load conditions which could affect the prestressing tendons. The load conditions would include thermal transients which could affect the anchoring components of the tendon.

The design requirements for this containment are such that the tendons will be affected by only a small amount by such considerations. This results primarily from the fact that a sliding surface is provided to control the interaction between the concrete and tendon (except of course at the end anchors) and some relative motion is thus provided for. Variations in tendon temperature are minimized by the insulating affect of the concrete and the small temperature differences between the concrete and tendon, caused by thermal cycling, can be accommodated by slip between the tendon and concrete, a factor preproved by the tensioning process. The sum of the relevant factors indicate that the anchor load to the concrete reduces with time but may vary slightly with seasonal changes and changes in operating conditions of the plant.

The friction factor is one factor which can affect the quantitative value for the qualitative statements in the previous paragraph. Tests on tendons from small to large, with small to large radii of curvatures and small to large central angles of curvatures have shown that the friction factors are not grossly affected by the variables mentioned. For the prestressing system selected for this containment, experimental evidence will be provided which shows that the friction factor values are adequately bounded.

2.4 MISCELLANEOUS PROVISIONS

2.4.1 The provisions made for corrosion protection of all prestressing tendon components will be indicated in the report made on this work.

2.4.2 The fabrication and quality control methods will be indicated for all tendon components and tendons. The most important one is of course, the final one where the tendon is jacked to 0.8f's before anchoring.

2.5 SUMMARY

The information obtained and provided will provide assurance that, under the most probable unfavorable conditions, rupture of the tendons and anchors is improbable and that the factor of safety is sufficient.

REFERENCES

1. A study of stress relaxation in prestressing reinforcement.

   D.D. Magura, Mete Sozen and Chester P. Siess

   Prestressed Concrete Institute Journal, 1964 (April).

2. Prestressed Concrete Pressure Vessels - Groun D Papers and Discussions

   Reports from a conference sponsored by the Institution of Civil Engineers.

3. Amendment 2 Attachment A, page V.14 to the Application of the Public Service Company Of Colorado For Construction Permit and Class 104 License For the Ft. St. Vrain Nuclear Power Plant - Docket No. 50-267

4. Amendment 4 Attachment A page V.15 of the Application described in Ref. 3.

3 DESIGN OF LINER PLATE ANCHORS

3.1 DESIGN CRITERIA

The anchors will be designed to preclude failure when subjected to the worst possible loading combinations. The anchors will also be designed such that in the event of a missing or failed anchor the total integrity of the anchorage system will not be jeopardized by the failure of adjacent anchors.

3.1.1 LOADING CONDITIONS

The following loading conditions will be considered in the design of the anchorage system:

1. Prestress
2. Pressure
3. Shrinkage and creep of concrete
4. Thermal gradients
5. Dead load
6. Earthquake
7. Wind or tornado
8. Flood
9. Vacuum

3.1.2 FACTORS AFFECTING ANCHORS

The following factors will be considered in the design of the anchorage system:

1. Initial inward curvature of the liner plate between anchors due to fabrication and erection inaccuracies.
2. Variation of anchor spacing
3. Misalignment of liner plate seams
4. Variation of plate thickness
5. Variation of liner plate material yield stress
6. Variation of Poisson's ratio for liner plate material
7. Cracking of concrete in anchor zone
8. Variation of the anchor stiffness

3.1.3 DESIGN CONDITIONS

The anchorage system will satisfy the following conditions:

1. The anchor will have sufficient strength and duct ductility so that its energy absorbing capability is sufficient to restrain the maximum force and displacement resulting from the condition where a panel with initial outward curvature is adjacent to a panel with initial inward curvature.

2. The anchor will have sufficient strength to resist the bending moment which will result from Condition 1.

3. The anchor will have sufficient strength to resist radial pull-out force.

3.2 MATHEMATICAL METHOD OF ANALYSIS

When the liner plate moves inward radially as shown in Figure 1, the sections will develop membrane stress due to the fact that the anchors have moved closer together. Due to initial inward curvature, the section between 1 and 4 will deflect inward giving a longer length than adjacent sections and some relaxation of membrane stress will occur. It should be noted here that section 1-4 cannot reach an unstable condition due to the manner in which it is loaded.

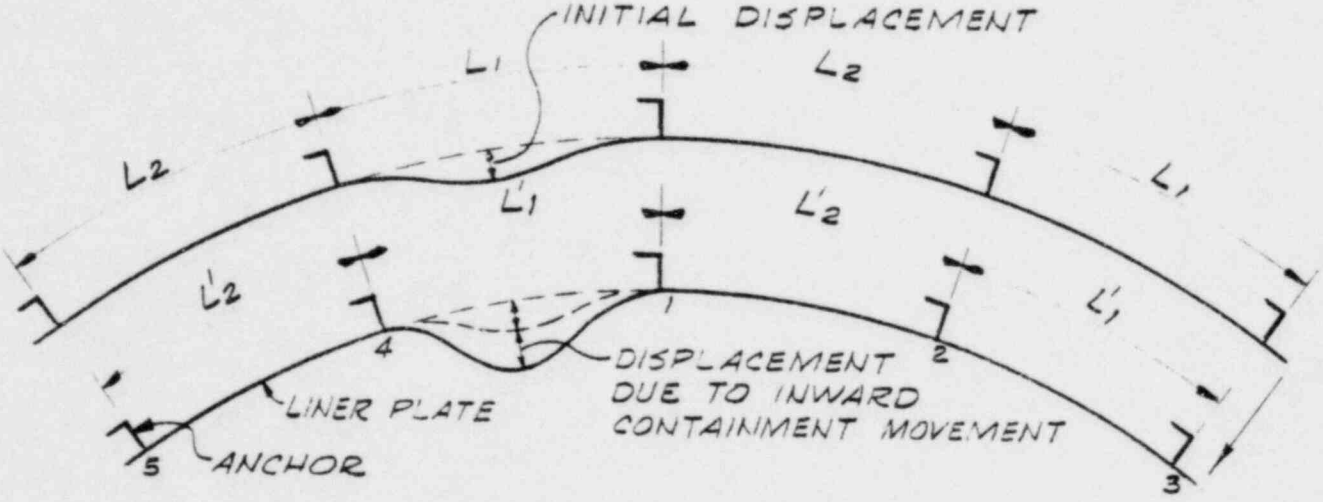

Figure 1

The first part of the solution for the liner plate and anchorage system is to calculate the amount of relaxation that occurs in section 1-4, since this value will also be the force across anchor 1 if it is infinitely stiff. The above solution can be obtained by solving the general differential equation for beams and the use of calculus to simulate relaxation or the lengthening of section 1-4. Figure 2 shows the symbols for the forces that result from the first step in the solution.

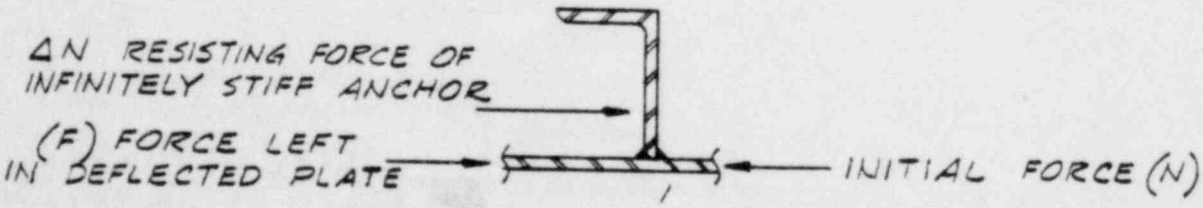

Figure 2

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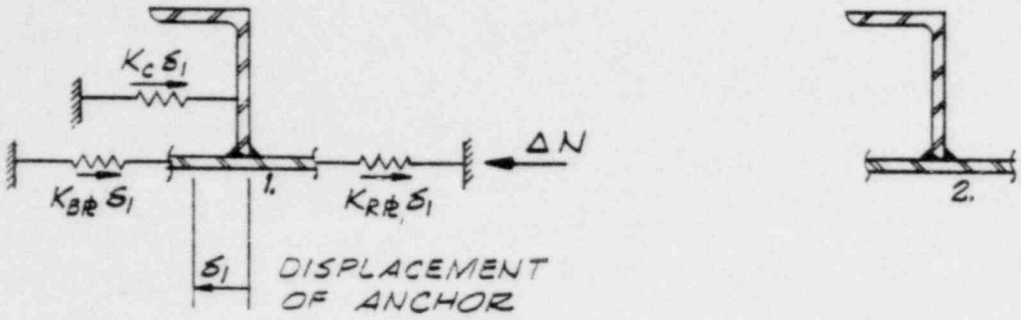
Using the model shown in Figure 3 and evaluating the necessary spring constants, the anchor can now be allowed to displace.



K_c - Spring constant of anchor

K_{BP} - Spring constant of deformed plate

K_{RP_1} - Relaxation of section 1-2 due to δ_1

Figure 3

The solution will now yield a force and displacement at anchor 1, but the force in section 1-2 is now $(N)-K_{RP_1}\delta_1$ and anchor 2 is not in force equilibrium.

The model shown in Figure 4 may be used to allow anchor 2 to displace and then find the effects on anchor 1.

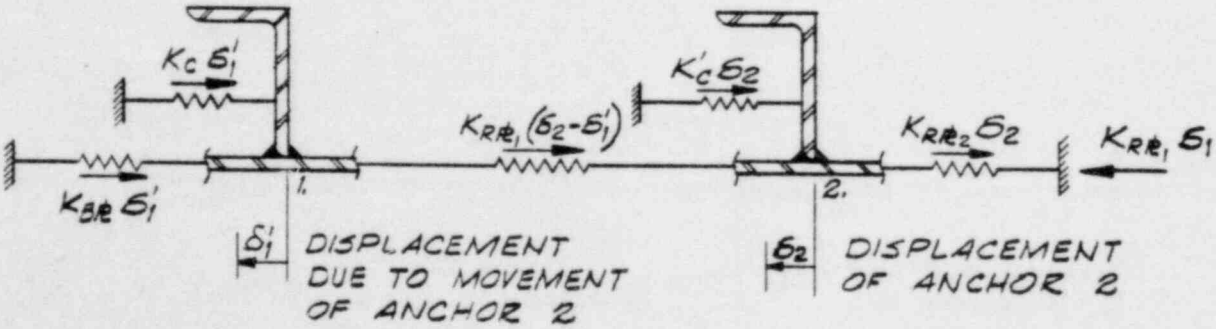


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Figure 4

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The displacement of anchor 1 is now $\delta_1 + \delta_1'$ and force on anchor 1 is $K_c(\delta_1 + \delta_1')$. Now anchor 3 is not in force equilibrium and the solution may continue to the next anchor.

After the solution was found for displacing anchor 2 and anchor 3, the pattern was established with respect to the effect on anchor 1 and by inspection, the solution considering an infinite amount of anchors was obtained in the form of a series solution.

The preceding solution will yield all necessary results. The most important results will be the displacement and force on the anchor 1.

3.3 FINAL METHOD OF ANALYSIS

The method outlined in section 3.2 produces an equation that is a very useful tool in designing the anchorage system. By varying the different variables which are contained in the equation, their effect on the design may be determined. If the conservative assumption is made that the spring constant K_{BR} is small relative to the anchor spring constants K_c and K_c', then the solution is fully dependent on the stiffness of the anchor.

Since the capacity of the anchor is both a function of its displacement and the applied force, the design must be based on energy considerations.

References 1, 2, and 3 can be used to evaluate an anchor spring constant. By using the equation obtained previously with the chosen spring constant, the amount of energy required to be absorbed by the anchor may be evaluted.

By applying reasonable variations to the anchor spring constant, the most probable maximum energy may be found.

References 1, 2, and 3 may also be used to conservatively evaluate the amount of energy that the anchor system will absorb.

By dividing the amount of energy that the system will absorb by the most probable maximum energy the result will then yield the factor of safety.

3.4 RESULTS OF ANALYSIS

By considering the worst possible loading condition which results from section 3.1.1 and the conditions stated below, Table 1 was obtained

Case I — Simulates the perfect plate with a yield stress of 32KSI and no variation of any other parameters.

Case II — Simulates a 1.25 increase in yield stress and no variation of any other parameters.

Case III — Simulates a 1.25 increase in yield stress, a 1.16 increase in plate thickness and a 1.08 increase for all other parameters. This case should adequately simulate the worst condition on the liner plate.

Case IV — Simulates a 1.88 increase in yield stress with no variation of any other parameter. The occurence of this situation is considered highly unlikely since the maximum ultimate strength of the liner plate material is 72KSI. This case is not considered as a design situation, but the anchor is still adequate.

Case V — Is the same as Case III except the anchor spacing has been doubled to simulate what happens if an anchor is missing or has failed.

LINER PLATE CALCULATIONS - RESULTS

TABLE 1

Case	Nominal Plate Thickness (in)	Initial Inward Displacement (in)	Anchor Spacing L_1 (in)	Anchor Spacing L_2 (in)	Factor of Safety Against Failure
I	.25	.125	15	15	37.0
II	.25	.125	15	15	19.4
III	.25	.125	15	15	9.9
IV	.25	.125	15	15	6.28
V	.25	.25	30	15	4.25

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[redacted]

The following discussion is given to provide additional information with respect to the "Factor of Safety Against Failure" that was submitted in Section 3, Supplement No. 7 of the PSAR.

THE AMOUNT OF ENERGY THAT THE SYSTEM WILL ABSORB

By using the test results previously summarized in the answer to Question 11.2.19, Supplement No. 4 of the PSAR and reducing the maximum capacity of the 4-12 fillet weld from 6.67 K/in to 5 K/in, the total available energy was calculated by integrating the area under the load-displacement curve. The amount of energy that the system will absorb was found to be 700 in lbs/in. The results from Reference 2 and 3 of Section 3, Supplement No. 7 of the PSAR were used to verify that the energy value given previously would not be reduced by the composite effect with concrete (this verification was made since the Bechtel tests did not include concrete).

THE MOST PROBABLE MAXIMUM ENERGY INPUT

Refer to Section 3, Supplement No. 7 of the PSAR for the definition of the variables considered in the following discussion. The most probable maximum energy input was calculated by integrating the area under the calculated load-displacement curve using the following input information:

$K_{BHL} = 0$, $K_{RHL} = .500 \times 10^3$ K/in/in, and K_c and K_c' were varied from $.0825 \times 10^3$ K/in/in to $.35 \times 10^3$ K/in/in

THE ANCHORAGE SYSTEM FACTOR OF SAFETY AGAINST FAILURE

The anchorage system factor of safety against failure was calculated by dividing the amount of energy that the system will absorb by the most probable maximum energy input.

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REFERENCES

1. Answer to question 11.2.19, Supplement No.4 of the PSAR.

2. "Amendment No.2 to application of Public Service Company of Colorado for construction permit and Class 104 license for the Fort St. Vrain Nuclear Generating Station".

3. "Liner Design and Development for the Oldbury Vessels" R. P. Hardingham, J. V. Parker, and T. W. Spruce, Group J, Paper 56, London Conference on Prestressed Concrete Pressure Vessels.

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4 LINER PLATE ANCHORAGE TESTS 13

(Submitted with Supplement No. 11)

5 FIXED END-ANCHOR BEARING PLATE TEST
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7. TENDON ANCHORAGE TEST SCHEDULE

The following is the Containment Tendon Anchorage Test Data:

The post-tensioning system selected for Arkansas Nuclear One consists of 186 wire BBRV tendons. Hoop and dome tendons will be stressed from both ends, vertical tendons will be stressed from the upper end only. The prestressing system components have been or will be tested.

The following tests have been completed:

1. Fixed End-Anchor, Bearing Plate Test

Test Conducted: April 20, 1969
Report Printed: June 20, 1969
Result: Proved the adequacy of the bearing plate.
The anchor plate observed unelastic deflection.
Test of the redesigned anchor plate has been scheduled.
(See under 4. below)
Note: Report was submitted with Supplement No. 11.

2. Stressing (Movable) End-Anchor, Bearing Plate Test

Test Conducted: August 19, 1969
Report Printed: October 3, 1969
Result: Proved the adequacy of Bearing Plate. Stressing anchor was not made of representative material.
~~Tendon~~ was stressed up to ultimate capacity.
~~Note:~~ ~~Report~~ is submitted with this Supplement.

3. Dynamic Test

Test Conducted: July 10 and July 18, 1969
Final draft printed
Result: Tendons showed higher than specified fatigue loading capacity.

4. Fixed End-Anchor, Anchor Plate Test

Test Conducted: November 6, 1969
Report Printed: February 27, 1970
Result: Proved that fixed end-anchor plate behaved essentially elastic and that with thickened anchor plate, reduced stress level and deformations relative to the fixed end-anchor bearing plate test were noted in the anchor and bearing plates. Test showed reasonable agreement with analytical results and proved adequacy of anchor plates. 17

The following tests are scheduled:

5. Stressing (Movable) End-Anchor, Stressing Anchor Test | 17

   Tentative Date: June 1970
   Purpose: Proving the adequacy of stressing anchor made by representative material.

6. Tendon Test

   Tentative Date: June 1970
   Purpose: Proving the specified characteristics of the tendon. (Strength and elongation.)

The test reports of bearing plate tests include comparisons between test results and analytical results and between test structure and real structure.