INTRODUCTION

The radial or tainter gate is the most common type of spillway crest gate used on large dam projects. Because of the simplicity, light weight, and low hoist requirements of a tainter gate, economy has been the basic reason for the trend toward its increased usage. However, this type of gate offers additional advantages over other types, such as permitting a very favorable design from the standpoint of hydraulics, ease of service, and maintenance.

During the past decade, the Corps of Engineers has designed tainter gates for approximately 70 projects including over 600 gates. Most of these gates have been constructed or are presently under construction. Prior to that time, it was somewhat unusual to design tainter gates in excess of 40 ft. in height. It is now not unusual to design 50-ft. wide tainter gates subjected to a hydraulic head of 60 ft. or more. For gates of this size, the direct thrust from hydraulic loading delivered to each spillway pier is approximately six million pounds. This load is transmitted from the trunnion yokes to the supporting trunnion anchorage girder and thence into the pier. While the discussion in this paper is confined primarily to the anchorages, a photograph of a tainter gate showing its principal parts is included as Fig. 1. The principal structural components of the gate are the skin plate assembly, the horizontal girders, the end frames, and the trunnion assembly.

EVOLUTION OF ANCHORAGES

While the size of tainter gates has continuously increased over the past ten or fifteen years, anchorages and anchorage girders have changed to provide more efficient and more economical supports. Although there has been quite a variation in types of anchorages used, the more general types will be presented before discussing the design of prestressed concrete anchorages.

Prior to the advent of the larger tainter gates, the anchorage system used, more than any other type, consisted of a steel girder, usually box type, cantilevered at each face of the pier to receive the trunnion loads. An example of this type is shown in Fig. 2. The loads from the trunnion girder are transmitted into the pier by an anchorage consisting of two standard rolled beams or built-up sections welded to the trun-
Fig. 1—Tainter Gate

Fig. 2—Horizontal Tension Tie Anchorage
The trunnion girder and to an embedded girder or grillage at the upstream end. These tie beams are placed with at least eight inches of concrete cover at the pier faces. The embedded girder is designed to receive the entire load from the tie beams in bearing. To allow free movement or deformation of the tie beams and prevent tension in the pier concrete, the beams are unbonded or mechanically isolated from the concrete for the entire length by $\frac{1}{2}$ inch of cork mastic.

For an intermediate pier with one gate loaded and the adjacent gate unloaded, the anchorage frame formed by the trunnion girder and the tie beams is subjected to severe racking. In addition to the direct tension in the tie beam nearest the loaded gate, there are bending stresses induced into the frame. This type of anchorage has short vertical beams embedded at the inner faces of the tie beams to transmit both the side thrust from the inclined end frames and the vertical component of trunnion reactions into the pier. Sliding at the downstream ends of the tie beams is allowed by use of bronze plates.

Another type of anchorage that had widespread use on the smaller to the moderately sized gates is one in which the trunnion loads are...
transmitted to the concrete below the pier rather than to the pier itself. This is shown in Fig. 3. In this type, the anchor ties are inclined at an angle with the horizontal large enough to place the embedded girder well below the level of the spillway crest. Columns are provided under the trunnion girder to take the vertical components of the trunnion loads. The cork mastic is also used in this case to isolate the anchorage members from the concrete except at the embedded girder and the column footings. While this anchorage requires greater amount of structural steel than the previous one, the reduction in the amount of pier reinforcing steel somewhat compensates for this.

While the above anchorages are quite satisfactory structurally for smaller gates, they are not suitable for large gates. As the size of tainter gates is increased, the problem of providing an anchorage system becomes more complex. It is generally economical to utilize the higher strength structural steels in the design of larger tainter gates. However, their use is not feasible for the anchorages because of the increase in elastic deformation due to the higher stresses, and because of in-
crease in length of embedment required. Resulting elongations of the tie beams would be excessive. The natural solution is to prestress the anchorage system into the pier. Prestressing is preferred since it will eliminate the relative movement between the pier concrete and the anchorage structure. A prestressed anchorage system appears most essential for large gates spanning between relatively narrow spillway piers.

Two types of prestressed anchorage systems that have received general usage consist of steel trunnion girders and post-tensioning into the piers. The system shown in Fig. 4 is one in which the trunnion loads are taken by separate vertical cantilever girders. While this type of anchorage is not considered economical, there is a construction advantage in that each anchorage can be aligned independently without affecting adjacent gates. A more conventional type consists of a welded structural steel box girder, similar to those used without prestressing, cantilevered out on each side of the pier to receive the trunnion loads. The first designs in the Corps of Engineers to incorporate post-tensioned anchorages were for Markland and Greenup projects on the Ohio River and were prepared by the Seattle District.

For very large tainter gates in which the total thrust anchored into the pier amounts to more than, say 6,000,000 pounds, the amount of structural steel involved in a trunnion girder becomes excessive. In 1961 a prestressed concrete anchorage was conceived and developed for the spillway tainter gates on Lower Monumental Lock and Dam on the Snake River. The design of Lower Monumental spillway gates was also prepared by the Seattle District. The gates on this project are 50 ft. wide by 61 ft. high. The girders are of high strength concrete, cast-in-place, post-tensioned, square in cross-section, and abut the downstream face of the pier. Prestressing into the pier is similar to that previously used for structural steel girders.

**MAJOR DESIGN CONSIDERATIONS**

The prestressed concrete anchorage consists of two systems of post-tensioned tendons; one for prestressing the girder itself, and one for prestressing into the pier. For discussion in this paper, the system of prestressing of the girder will be referred to as the "transverse" prestressing and the prestressing into the pier as "longitudinal." This anchorage system is depicted in Fig. 5.

A compressible filler is installed over the central portion of the vertical contact surface between the pier and the girder. This gives a more efficient arrangement by providing a larger moment couple for the prestressing forces and reducing the bending in the girder. After prestressing is completed, the prestressing tendons are grouted and concrete caps are placed over the exposed ends of both the transverse and longitudinal tendons.

There are usually two major design conditions for the intermediate piers: one in which both gates are closed, and the other condition in which one gate is closed with the adjacent gate open. The latter condition governs the design of the longitudinal prestressing.

The longitudinal prestressing elements are arranged in two groups, each group being placed as near the pier face as practicable, allowing about 9 inches of cover for the outside rows of tendons so that con-
Conventional pier reinforcing steel may be placed outside the prestressing steel. The amount of prestressing steel is determined assuming the unbalanced condition of one gate loaded with the adjacent gate unloaded.

While the bearing stress distribution is not exactly known, longitudinal prestressing that will provide an average residual bearing stress between the girder and the pier on the loaded side of about 20 percent of the final prestress will generally be satisfactory. However, it may be necessary to investigate the interface pressures to assure that no tension exists under unbalanced loads.

The girder is considered a rigid member with a straight-line variation in the stress distribution. However, for the girder design, the uniform stress distribution assumption will be conservative.

Another consideration in the design of the longitudinal prestressing into the pier is to unload the prestressing load at the embedded end gradually rather than abruptly on a single vertical plane. This is accomplished by staggering the cutoffs of tendons and will alleviate the objectionable concentration of large quantities of reinforcing steel that would otherwise be required to control vertical tension cracks in the pier concrete. Where practicable, the gate trunnion assembly is positioned so that the vertical component of thrust is negligible with the gate closed and full pool. Theoretically, this places the trunnion at about one-third the height of the gate above the sill which permits placing the pier prestressing elements horizontally.

A zone of high strength concrete is incorporated into the pier for the entire length and height of the prestressing elements. Placing the prestressing elements horizontally is a construction convenience in that fewer lifts of the high strength con-

Fig. 5—Prestressed Concrete Anchorage
crete are required and also high steel support frames for the down-stream portion of the prestressing elements are eliminated.

The design of the post-tensioning elements into the pier can be accomplished without regard to the trunnion girder design requirement. After the longitudinal prestress has been determined, all external loads on the trunnion girder are known. Design of the girder actually involves two steps: (1) choice of the geometry of the girder and (2) the analysis of the trunnion girder to determine the unit stresses and establish the amount and arrangement of transverse prestressing.

The facility of selecting a member of proper section comes with experience. Usually a concrete of 5,000 psi strength is required except for larger gates which may require higher strength concrete. The size of the girder required is determined by the magnitude of the bending, shear (diagonal tension) and torsion. Generally, the minimum width of girder (vertical height) will be established by the space required to accommodate the longitudinal pre-stress tendons. Maximum torsion usually will occur in the girder when the gate is partially raised and the pool is at maximum level. When it is practicable to position the trunnion yoke on the girder to minimize the effects of torsion under the maximum loading condition, the combined shear, including torsion, will not generally be critical for other loading conditions. The flexural stresses in most cases will be considerably below those permitted by specifications.

The ordinary beam theory or straight-line distribution of flexural stresses does not consider the effects of the normal pressures from the external loads and reactions at the top and bottom edges of the beam. For deep beams, however, these effects are substantial and cause a non-linear distribution of bending stresses and the shear stress distribution is not considered parabolic. Also, it has been shown in ordinary shear investigation that even in shallow beams the maximum principal tensile stress is exaggerated, and that the critical section for inclined tension is not at the reaction but some distance from the support. The reason for this is that, near the reaction, the values of the vertical compressive stresses are large. An analysis by the finite element method in plane stress analysis indicates that the effects of the trunnion load reduce the principal tension substantially.

For the reasons expressed in the above paragraph, it is assumed that the critical section for principal tension is near the upstream edge of the girder. This appears to be the logical area for any cracks to develop. With this assumption, the principal stress obtained from combining average shearing stress with one normal stress is considered conservative, i.e., neglecting any compressive stress caused by the trunnion reaction.

Limiting values for principal tensile stress vary considerably. Based upon working load, the limiting values of principal tension, as given in T. Y. Lin’s Design of Prestressed Concrete Structures range from about 0.015 $f'_c$ to 0.033 $f'_c$ for beams without web reinforcement. Considering ultimate load, the range is from 0.045 $f'_c$ to 0.08 $f'_c$ without web reinforcement, and to about 0.11 $f'_c$ with web reinforcement. The codes for prestressed concrete in the United States make no reference to
limiting values of working load principal tension and generally the shear design is based on ultimate strength. A limiting principal tensile stress of \(2\sqrt{f_c'}\) under working load is considered reasonable when no web reinforcement is provided. This is quite comparable to the British Standard Code of Practice. When web reinforcement is provided, a limiting value of \(3\sqrt{f_c'}\) is conservative. This may serve as a guide; however, the design should in the end be investigated for ultimate strength and reinforcement provided accordingly.

For the small to moderate size gates, it is practicable to limit the computed principal tension to a value for which theoretically no web reinforcement is necessary. However, for the larger gates, the principal tension, as computed often governs the design, and it may not be practicable to limit the principal tension to that in which no shear reinforcement is required. While this method of determining principal tension is admittedly conservative, it is considered reasonable for design until more is known on this subject. In any case, it is considered prudent to provide a nominal amount of web reinforcement and this may be computed by rule of thumb. The minimum percent of web reinforcement desirable is about 0.003 \(bs\), in which \(b\) is the width of girder and \(s\) the spacing of web reinforcement.

The ultimate strength of girders should be investigated both for flexure and shear. The ultimate load on the girder, based upon 1.8 times the normal trunnion working load, should not exceed the ultimate flexural strength produced by the following formula given in the ACI Code:

\[
M_u = \phi \left[ A_v f_{sv} d (1-0.59q) \right]
\]

The principal tension in a prestressed concrete member may increase quite rapidly compared to the unit shearing stress for an increase in the external loading. For this reason, it becomes necessary to investigate the principal tension under ultimate shear load conditions. This is accomplished by computing the unit shears stress resulting from a principal tension of \(4\sqrt{f_c'}\) which is considered the diagonal cracking stress. The difference between the shear value required for ultimate strength and that for the shear cracking load should be taken care of by providing web reinforcement. This is computed by the formula:

\[
A_v = \frac{(V_T + \phi V_c) s}{\phi d f_v}
\]

The areas of concrete beneath the prestressing loads in both the girder and the pier end anchorage zones or end blocks are subject to tensile stresses. The stress distribution is rather complicated in the end anchorage zone of the pier with two general areas of tension: (1) the center of the section which is termed the “bursting zone”, (2) on the sides and end surface called the “tensile spalling zone”. Guidance for this type of behavior is discussed in Guyon’s *Prestressed Concrete*. While there remains considerable latitude for the designer’s judgment, it is considered that generally conventional grid reinforcement placed as near the surface as practicable to take about 4 percent of the total prestressing force will be adequate in the spalling zone. To prevent spalling of the concrete at the corners of the pier, the outer layer of reinforcement is welded to angles embedded along the vertical edges of the pier. Tensile stresses in the bursting zone are evaluated by Guyon and the ratio of the bursting...
tension to the prestressing force is shown to be a maximum of about 0.18 for the typical pier load arrangement. Reinforcement for these stresses should be provided for a distance in from the downstream pier face of approximately one-half the width of the pier. The theoretical stress pattern for these zones is included in the above reference.

The actual final load on the prestressing steel should not exceed 60 percent of the minimum ultimate strength of the steel. The initial tension on the steel, immediately after seating of the anchorages, should not exceed 70 percent of the ultimate strength of the steel. The computed losses in steel stress due to elastic shortening of the concrete, shrinkage, creep, and plastic flow will be less for gate anchorage systems in massive piers than with normal beam or slab prestressing. Therefore, the initial prestress will generally be somewhat below 70 percent of the ultimate strength to assure that the final stress does not exceed 60 percent.

To minimize integral action between the girder and the pier, the girder is prestressed prior to longitudinal prestressing.

**ANALYSIS OF DESIGN**

The steps in design procedure are:

1. Determine magnitude of the longitudinal prestressing (into the pier) required for the different loading conditions which will complete the applied loads for the girder.

2. Determine the shear and moment diagrams for the appropriate loading conditions.

3. Assume a girder cross section and compute its properties.

4. Determine the magnitude and arrangement of prestressing force to provide compressive stresses to offset tensile stresses caused by bending moments and to limit the principal tensile stresses.

5. Investigate stresses at both the critical section for moment and the critical section for shear with two combinations of conditions, namely, final prestress plus full design load and initial prestress plus dead load only.

6. Compute ultimate strength of girder to see if it meets the requirements.

7. Design shear steel by ultimate strength method.

8. Determine end block requirements.

Design of the longitudinal prestressing is based upon an unbalanced loading condition of one gate closed with the adjacent gate open, see Fig. 6. To maintain a residual bearing stress of 20 percent of the final prestress, the prestress after losses is

\[ F = T \times \frac{A}{B} \times 1.25 \]

and the number of tendons required in each prestress group is

\[ N = \frac{F}{f_{ts}} \]

where \( f_{ts} \) is the final prestress per tendon (maximum value of 60 percent of the ultimate strength). The prestress force is then corrected for the actual number of prestressing tendons selected. The loading conditions for the girder are now known and the shear and moment diagrams may be computed as indicated in Fig. 6. Based on a trial section, the girder can now be analyzed. The width (vertical height) of the girder is somewhat governed by the requirement of the longitudinal prestress force.

The girder prestressing arrangement should be such that the downstream fibers of the girder under an
unloaded condition will not be in tension due to prestressing and dead load. It is suggested that a minimum compression stress of 0 to 100 psi be maintained for the unloaded condition to insure compression across the entire section even when the pool is below the level of the gates. Fig. 7 indicates the normal stress distribution on the section of maximum shear due to prestressing and moment at that section. At the section of maximum moment, no tension should result in the upstream fibers under maximum working load condition. At the section of maximum shear, a fairly uniform distribution of normal stresses across the section usually results from bending and prestressing when the prestressing is based on principal stress limitations.

For the conditions in which torsion is a consideration, the torsional shear stress should be combined with the direct shear stress. Where it is required that torsional stress be calculated, Bach’s method may be used as presented in Advanced Mechanics of Materials by Seely and Smith:

\[ S_b = \frac{9 M_t}{2 b d^2} \]

\[ S_d = \frac{d S_b}{b} \]
The principal tension may be expressed by:

\[ f_t = \frac{f_3}{2} - \sqrt{\left(\frac{f_3}{2}\right)^2 + v^2} \]

\[ v = \frac{V}{bd} \]

with a limitation of \( f_t = 2\sqrt{f_c} \) when no design web reinforcement is provided and \( f_t = 3\sqrt{f_c} \) with web reinforcement. The above equation is solved for \( f_3 \) which is then expressed by:

\[ f_3 = \frac{P}{A} - \frac{P_{ec}}{I} - \frac{Mc}{I} \]

where \( e < d/6 \)

and finally the required girder pre-stress force, \( P \), is determined.

The above computations theoretically represent the condition at the section of maximum shear stress. It is now necessary to investigate the stress conditions at the section of maximum moment as indicated in step 5 of the design procedure to insure that allowable stresses are not exceeded.

The ultimate flexural strength is determined by the formula for \( M_u \) as previously stated. However, the ratio of prestressing steel used should be such that \( \frac{f_{au}}{f'_c} \) does not exceed 0.30. If this factor exceeds 0.3, then the \( M_u \) is computed by a different formula and is given in the ACI Code (Formula 26-8). The load factor of 1.8, as stated earlier in this paper, is considered adequate for this type of prestressed member where the working loads are quite definite. The shear steel, \( A_v \), is then determined based upon ultimate shear strength. Maximum shear reinforcement should be provided as previously discussed.

**RESULTS OF MODEL TESTING**

Because of its configuration, the trunnion girder is beyond the range of the usual beam or girder design. Since the trunnion girder is cantilevered at each side of the pier and...
the span is short, the behavior is similar to that of a deep beam. Stress distribution is quite complex as a result of transverse prestressing of the girder, longitudinal prestressing into the pier, and gate loadings which may be either symmetrical or unsymmetrical. The interaction between the girder and pier and the compression in the beam between the yoke and the longitudinal downstream anchor plate further complicate the pattern of stress distribution and is quite indeterminate.

The applicability of the assumptions for conventional beam flexure and shear to this type of structural member was questioned. As a result, structural model tests were performed to determine the structural strength and behavior, verify the original design, and to provide information for future designs. It is beyond the scope of this paper to discuss the model tests in detail, and only a brief résumé of the results is included.

The test program was performed under the direction of Dr. Arthur R. Anderson of Anderson, Birkeland, Anderson & Mast of Tacoma for the Seattle District of the Corps of Engineers. The model tests included nine beams which were tested to failure. The prototype girders for Lower Monumental tainter gate anchorages are 8 ft. by 8 ft. in cross section by 23 ft. 2 in. long. A special facility for testing one-fourth scale models of the trunnion girders was constructed with a block of concrete to simulate the spillway pier. The girder models were fastened to the pier by post-tensioned high-tensile strength steel rods. The girder models were cast-in-place concrete of 6,000 psi strength. A cross head was built into the pier to jack against for applying thrust (trunnion load) to the girder. The test facility is shown in Fig. 8.

Various prestress combinations were employed in the model testing. The width of girder for the model also was varied to determine the effect of girder width on strain distribution in the concrete. Twenty-four SR-4 type A-12 electric strain gages were attached to the upper surface of each model. Model No. 1 was cast on the pier which had the surfaces of both the vertical face

![Fig. 8—Model and Test Facility](image-url)
and the horizontal face sandblasted. While this model was able to resist up to 2.2 times the design load, it was not possible to interpret the strains measured because of the unknown conditions resulting from shear action between the model and the pier. The other models tested were cast by preventing bond on the horizontal surface between the girder and pier with the idea that the model was free to deflect under horizontal loads and that a stress distribution conforming to that of a beam subjected to bending could be achieved.

The model tests revealed a very complex behavior of the prestressed concrete girder due to the interaction of the trunnion girder and the pier. As a result of this interaction, the strains measured were not always linear with applied load which made the interpretation of the data difficult. Before the trunnion loads are applied, the interface between the girder and pier is under high compression from the longitudinal prestress, and as the load approaches the prestress value, the pressure on this face approaches zero. The change in local concentrated stress along the upstream face of the girder is rather severe and a very steep stress gradient results. Despite the limitations of the strain measurements in the girder model, they are instructive in presenting the behavior under load of a structure of this type. The torsional effect from vertical loads on the trunnion was not serious. The crack patterns run parallel to the hoop reinforcement near the zone of maximum moment, demonstrating that this reinforcement contributes very little to the ultimate shear strength of the girders. This was verified by one model in which nearly all of the web reinforcement was omitted.

The model tests confirm that the Lower Monumental girder design is adequate and that the prestress in the girder could be reduced somewhat and yet resist cracking at 1.5 design loads. In the model tests, when a load 1.5 times the design load was applied, the transverse prestressing was reduced until cracking occurred. At this point, the tension in the concrete on the upstream face was approximately 1,000 psi, computed by the straight-line method. With this reduced prestress force applied for the design loads, a tension of 500 psi will exist based upon a straight-line assumption.

Figs. 9 and 10 represent the results of a computer program performed by the Seattle District for the models. These theoretical analyses of principal stresses are based on the finite difference method and the finite element method. Fig. 11 gives a comparison of flexural stresses based upon theoretical analyses and straight-line beam theory.

**SUMMARY**

Since the idea of prestressed anchorage systems was introduced, it has been possible to utilize larger tainter gates, in which the trunnion loads on a single pier exceed six million pounds, with far less embedded anchorage metal than previously required and without a substantial increase in pier size. The use of the prestressed concrete trunnion girders has proven to be economical. For the anchorage system of the Lower Monumental gates, the bid price indicates about a 40 percent saving over a preceding project having gates of similar size but utilizing structural steel trunnion girders. Studies and bids on subsequent projects, including smaller gates, also reflect a substantial saving though not generally
Fig. 9—Stress Contours—Finite Difference Method
Fig. 10—Stress Countours-Finite Element Method
Fig. 11—Comparison of Flexural Stress
of this degree. The prestressed concrete anchorage is proposed for several Corps of Engineers' projects. Comparison of the results of structural model tests, as well as theoretical analyses by the finite element method and finite difference method confirm that the straight-line theory for these structural members is reasonable. While the model tests demonstrate that a fairly high flexural tensile stress, say about 400 psi, appears reasonable under working load conditions and would also provide adequate ultimate strength, it is not recommended that tension be allowed in the design. This is in accordance with the codes. While it is recognized that the maximum computed principal tension will be somewhat greater than actually exists, conservative design assumptions for this type of structural member are recommended.

REFERENCES

Presented at the Tenth Annual Convention of the Prestressed Concrete Institute, Washington, D.C. September 1964