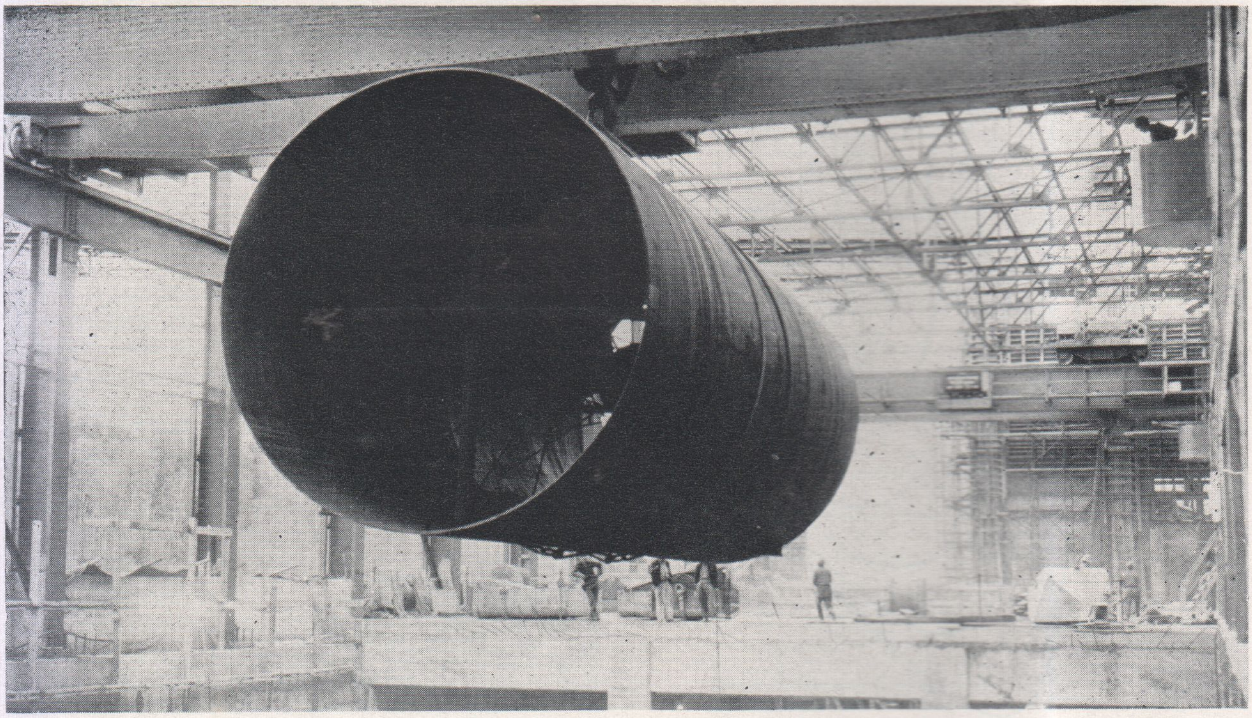

Fig. 1. Steel liner for penstock for Bersimis No. 2 development

The Buckling Resistance of Steel Liners for Circular Pressure Tunnels

The authors appraise various theoretical analyses of the buckling of encased steel liners subjected to external pressures and suggest design measures for stiffened and unstiffened liners. The practical application of the design measures is demonstrated on two examples of recent installations

By IAN W. McCAIG* and PAUL J. FOLBERTH§

UNTIL the last decade, turbines installed in medium- and high-head hydro-electric developments were, with few exceptions, of a size in which it was convenient and economical to construct the penstocks feeding them as steel pipelines supported by pedestals. The rapid increase of turbine capacity in recent years, however, made the design of such pipelines increasingly difficult, as with increase in penstock diameter the metal thickness required to resist internal pressure has become too great for riveting or welding unless special high-tensile steels are used. With the larger units, it is prudent to consider driving the penstocks as tunnels in rock and line these with steel plate encased in concrete. The concrete and rock surrounding the steel liner then take up part of the internal pressure load, and the metal thickness can be kept to a minimum.

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In the design of steel liners for pressure tunnels, both external and internal pressures must be considered. External pressures can occur either during the tunnel grouting operation or on tunnel dewatering after a prolonged period of use, and very often it is the external rather than the internal pressure that determines the thickness of the steel liner. In cases where a substantial increase in liner thickness would be required to prevent buckling due to external pressure, it may be more economical to provide anchorages or external ribs. For short steel-lined pressure tunnels, drainage arrangements to relieve the external pressure load on tunnel dewatering can be considered, but for long pressure tunnels it may not be possible to clean the drains periodically and drainage facilities cannot be relied upon.

The penstocks for the Bersimis No. 1[1, 2] and Bersimis No. 2[2] developments in Canada were driven as tunnels, with finished internal diameters of 10 ft and 17 ft respectively. The static head for these developments was 875 ft and 385 ft. The construction of surface penstocks at these sites would not have been economical, and it was only by the use of steel-lined pressure tunnels in rock that it became possible to carry in one pipeline the high flows required for the 200,000-h.p. turbines of these schemes. At Warsak[3], in Pakistan, the use of a 39-ft-diameter power tunnel divided at a manifold into six 18-ft-diameter steel-lined pressure tunnels, provided the most economical power conduit arrangement at this difficult site. Here, the topography did not lend itself to surface pipelines.

At the Bersimis No. 1 development, external pressures were of minor consequences. At Bersimis No. 2, however, the thicknesses of some portions of the penstock steel liners were governed by external pressures. At Warsak, the external pressures had to be given detailed consideration. Anchors were provided to resist external pressures during the grouting operation and relief drainage to lower external pressures on tunnel dewatering. For this development these measures were economical as they allowed an appreciable reduction in steel liner thickness. The steel-lined pressure tunnels for the Bersimis and Warsak developments are described in detail at the end of this paper.

External Pressure

External pressures acting on the steel liner may become critical either during construction or on dewatering after prolonged use.

During construction, external pressures may be exerted by the concrete while the space between the liner and the tunnel sides is being filled. However, such forces are normally resisted by providing internal bracing as a temporary measure. Additional stiffening is sometimes provided by the anchors required to hold the liner in place during the concreting operation. After the concrete has set, ground-water pressures may build up unless precautions are taken. Normally, adequate relief against this condition is provided by leaving open the grout plugs in the liner.

Frequently, the grouting operation is the critical design case, as considerable pressures are required for this operation to be completely effective in closing the gap between the liner and surrounding concrete. Usually, internal stiffeners cannot be left in place during the grouting operations.

Even after the installation has been in service, the steel liner may again become subject to external water pressures. In this event, the pressures are due to water which has infiltrated from the conduit, entering fissures or other openings in the rock. Given ideal conditions, the external pressures due to this water could be as high as the internal pressure within the conduit. However, it is reasonable to assume that in most instances the fissure system is interconnected and a seepage path to the ground surface exists. Thus, the maximum pressure head will not normally exceed the depth below the ground surface.

Design Measures

The effects of external pressures and modes of failure will be discussed for the following cases:

(a) Unstiffened steel liners.
(b) Steel liners with anchors acting as longitudinal stiffeners.
(c) Steel liners with stiffening rings.

Measures to relieve external pressure will be considered under subsection (*d*).

(a) *Unstiffened Steel Liners.* If the external pressure acts uniformly on the liner and the latter behaves elastically, failure would occur when the hoop stress reached the yield stress f_y. The pressure at failure p_y would then be given by the equation:

$$p_y = \frac{2f_y}{K}$$

where K is the ratio of the diameter to thickness of the steel liner.

In most cases, however, collapse of the liner will be due to elastic instability or buckling, and failures of this nature have been studied by Vaughan[4], Borot[5], and Amstutz[6].

In their analyses, both Vaughan and Borot assumed that the steel liner would buckle into an even number of half waves around the circumference, where the number of waves m may be determined from the well-known formula for the buckling pressure of a free-standing pipe under uniform external pressure, namely:

$$p = \frac{2E'}{3K^3}(m^2 - 1)$$

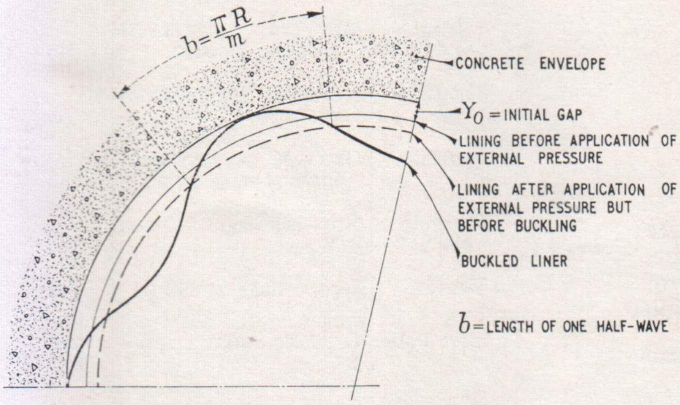

Fig. 2. Buckling according to Vaughan and Borot

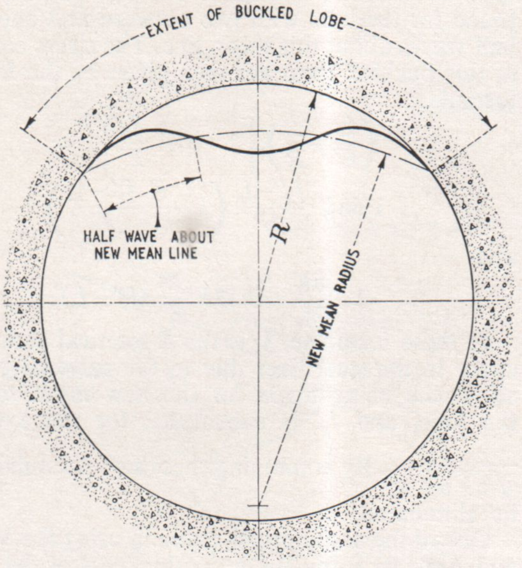

Fig. 3. Buckling pattern according to Amstutz

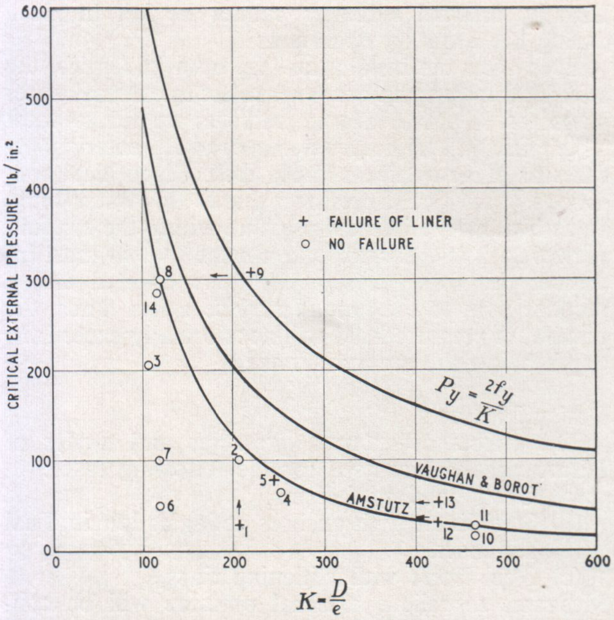

Fig. 4. Vaughan and Borot, and Amstutz analyses for various values of K

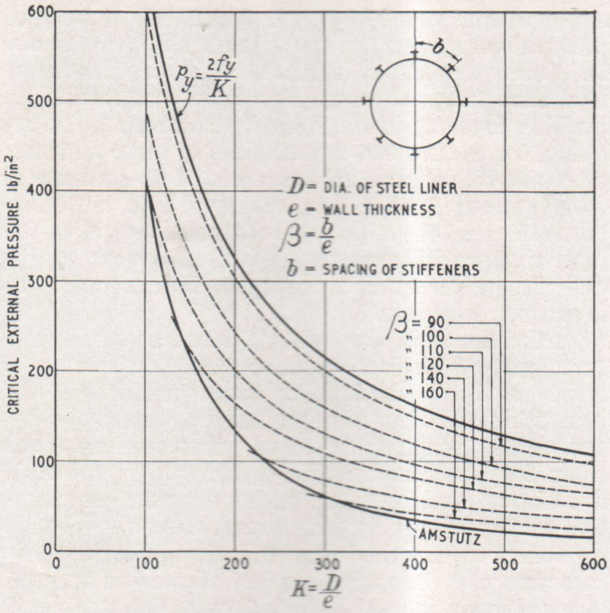

Fig. 5. Evaluation of the formula deduced by the authors

In this formula p designates the external pressure at buckling, E' is Young's modulus divided by $1-u^2$, where u is Poisson's ratio for the material, and K is defined as above. After buckling, the liner would thus have the form pictorially represented in Fig. 2.

By considering bending stresses caused by buckling, Vaughan and Borot derived optimum conditions for deflection of the buckling lobe. The external pressure p producing critical buckling stress f_y can be calculated from the following formula:

$$\frac{13K^2}{4E'}\,p^2+2p\left\{1+3K\,\frac{y_o}{R}-\frac{f_yK}{2E'}\right\}-\left\{\frac{4f_y}{K}-\frac{f_y{}^2}{E'}\right\}=0$$

where y_o represents an initial gap that may exist between the steel liner and surrounding concrete and R the radius of the steel liner.

Amstutz, in his analysis, assumed more correctly that buckling of the steel liner would not occur in uniform waves around the circumference of the steel liner, but a single lobe would form in one particular spot, as indicated in Fig. 3. The new mean radius is found for the lobe with two outward and one inward half waves about this mean line. The stress conditions in this buckled form are then given by the following formulae:

$$\left(\frac{f_n}{E'}+\frac{y_o}{R}\right)\left[1+3K^2\,\frac{f_n}{E'}\right]^{3/2}=$$
$$1{\cdot}68K\,\frac{f_y'-f_n}{E'}\left(1-\frac{K}{4}\,\frac{f_y'-f_n}{E'}\right)$$

and

$$1-\frac{pK}{2f_n}=0{\cdot}175\,\frac{K}{E'}\,(f_y'-f_n)$$

In these formulae, f_n is the theoretical compressive stress in the steel liner due to the external pressure, allowance being made for the new mean radius of the lobe, and f_y' is substituted for the expression $\frac{f_y}{\sqrt{1-u+u^2}}$. By combining these two formulae, f_n can be eliminated.

One of the assumptions made in using the Vaughan and Amstutz formulae is the thickness of the gap between the steel liner and the surrounding concrete. The smaller the gap, the greater the critical external pressure on buckling. The size of the gap depends on site conditions and the construction techniques employed.

Results of the analyses of Vaughan and Borot, and also that of Amstutz, are shown in Fig. 4 for various values of K. The upper limit for yield without buckling is also indicated in this figure. It can be seen that the theoretical collapse pressures as determined by the Amstutz analysis are considerably lower than those for the analysis by Vaughan and Borot. On account of this, a search was made for technical data that might substantiate one or other of these analyses.

TABLE I

Point No.	Development		Buckling failure
1	Shira	Accidental overpressure during grouting	Yes
2	Shira	Measured ground-water pressure	No
3	Shira	Maximum specified grouting pressure	No
4	Whatshan	Estimated ground-water pressure	Yes
5	Whatshan	Observed pressure during first-stage grouting	No
6, 7 and 8	Bersimis No. 1	Grouting Stages 1, 2 and 3	No
9	Kemano	Second-stage grouting	Yes
10 11	Ladore Falls	Grouting Stages 1 and 2	No
12 13	Nilo Pecanha	Grouting Stages 1 and 2	Yes
14	Calancasca	Grouting	No

lar point in the direction of the arrow. It can be seen that a large number of failures occurred below the curves derived by the theories of Vaughan and Borot, whereas the successful installations and failures straddle the curve of Amstutz.

(b) *Steel Liners with Anchorages Acting as Longitudinal Stiffeners*. If the steel liner is provided with longitudinal rows of closely spaced anchorages, it is obvious that the extent of the lobe during buckling is limited to the spacing of the stiffeners. Introducing this criterion into the theory of Amstutz, we find by transformation:

$$f_n = \frac{E'}{12K^2}\,(9m^2 - 1)$$

in which $m = \frac{\pi D}{2b}$ and b is the extent of the buckled lobe, equivalent to the spacing of the stiffeners (see Fig. 3). Since $9m^2$ in the above equation is much greater than unity, the above expression may be simplified as follows:

$$f_n = \frac{3E'm^2}{4K^2}$$

Inserting this term into Amstutz's formulae and expressing the spacing of the stiffeners in terms of the wall thickness of the liner:

$$b = \beta e \qquad m = \frac{\pi K}{2\beta}$$

we may find:

$$p = E'\,\frac{4{\cdot}38}{\beta^2}\left[\frac{4{\cdot}38}{\beta^2} + \frac{3{\cdot}39}{K} - \frac{f_y'}{E'}\right]$$

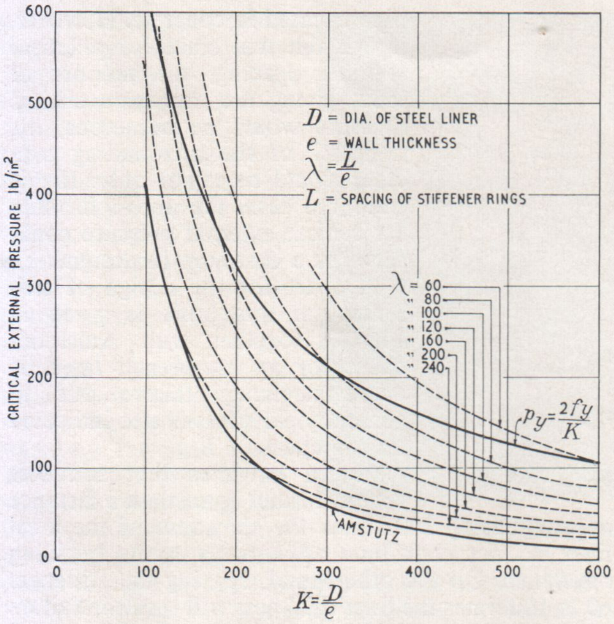

Fig. 6. Chart for the design of stiffeners

The data collected are given in Table I, and have been plotted in Fig. 4.

The circles in Fig. 4 indicate that at these pressures no failure occurred; the crosses indicate failure due to buckling. Where data are incomplete, arrows have been drawn indicating a probable shift of the particu-

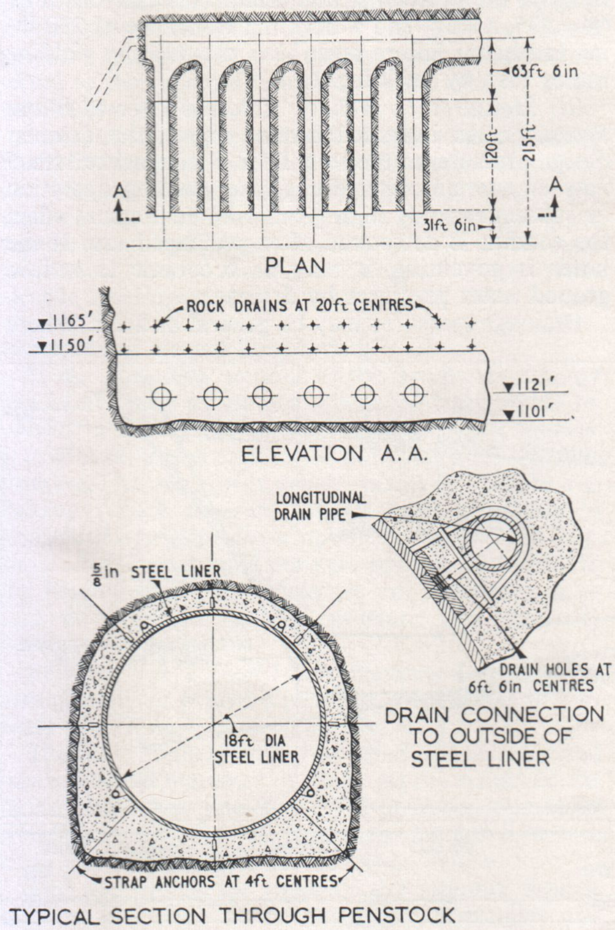

Fig. 7. Drainage measures employed at Warsak

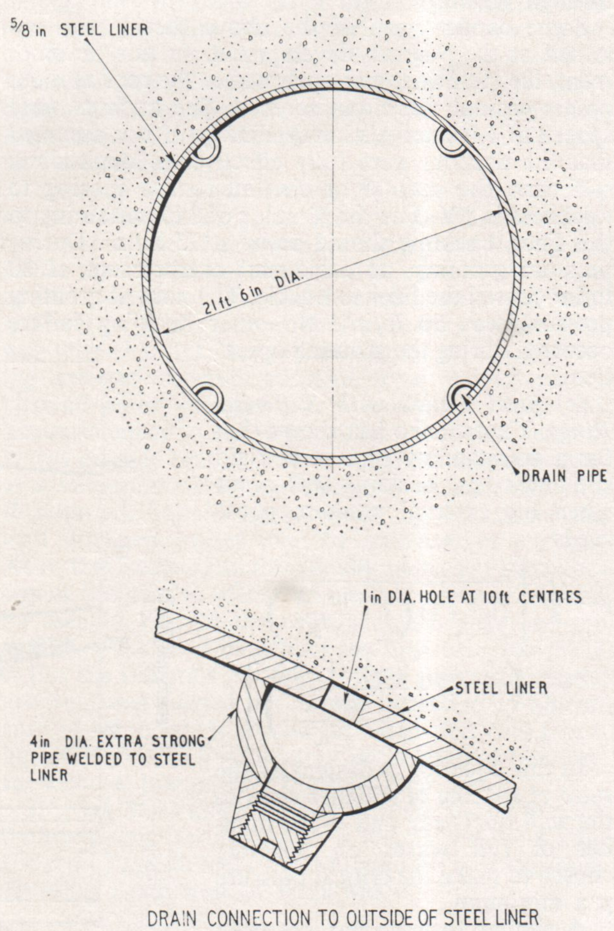

Fig. 8. Drainage detail at Upper Campbell Lake

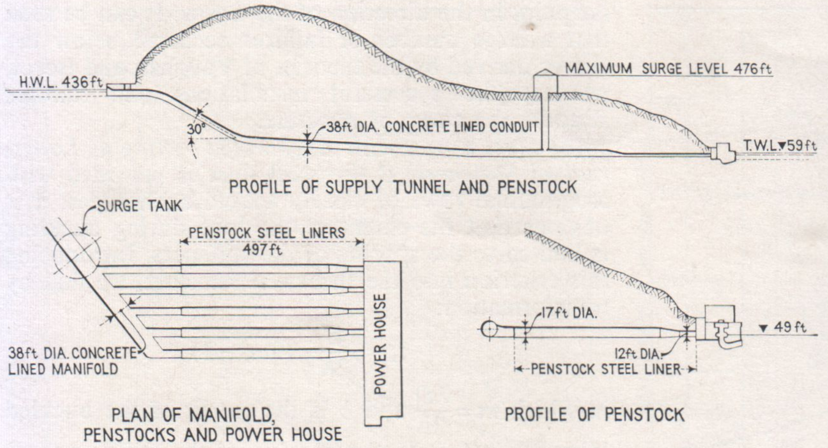

Fig. 9. Arrangement of pressure conduits at Bersimis No. 2

An evaluation of this formula is given in Fig. 5. It can readily be observed how the buckling strength of the steel liner increases as the spacing of the stiffeners is decreased.

The curve for $\beta=160$ is near the Amstutz curve, and it may be concluded that longitudinal stiffeners spaced at a distance of 160 times the wall thickness of the steel liner or further do not increase its resistance against buckling. The upper limit is given approximately by $\beta=86$, at which spacing the resistance against buckling of the steel liner is equal to its yielding under external loads. Closer spacings of the stiffeners are not necessary, as the steel liner would fail through yielding.

Some confirmation of the above theory was obtained at the Warsak development. In this development, the steel liner was anchored to the rock at eight points around the circumference. The anchors were spaced at 4-ft intervals longitudinally. If it is assumed that the anchors were fully effective as longitudinal stiffeners, the ratio β of circumferential spacing to thickness is 136. For the K value of 346 pertaining to this liner, buckling should occur at 75 lb/in^2. During pressure grouting, an accidental overpressure of 80 lb/in^2 caused the liner to buckle. The normal grouting pressure was 50 lb/in^2. No other buckling failure occurred during the grouting operation.

(c) Steel Liners with Stiffening Rings. Timoshenko has shown that for a freestanding pipe with rigid stiffener rings, buckling will occur when the external pressure p exceeds

$$p=\frac{2E'}{K}\left[\frac{1-u^2}{(m^2-1)\left(1+\dfrac{4m^2\lambda^2}{\pi^2K^2}\right)^2}+\frac{1}{3K^2}\left(m^2-1+\frac{2m^2-1-u}{1+\dfrac{4m^2\lambda^2}{\pi^2K^2}}\right)\right]$$

In this equation λ designates the ratio of spacing of stiffener rings to the wall thickness, and m the number of full waves on buckling chosen to make the critical pressure p a minimum.

A similar formula has not been developed for the case of a stiffened pipe encased in concrete. However, it was felt that since for practical stiffener spacing, the number of lobes giving the minimum critical pressure would be numerous, the influence of the surrounding concrete would be minor only. If this were the case, the above formula for critical external pressure could serve as a first step in design.

A chart for the design of stiffeners on this basis is given in Fig. 6, together with Amstutz's curve for an unstiffened liner. A curve depicting failure due to general yield stress is also indicated on the chart.

Fig. 6 indicates that stiffeners spaced farther apart than a distance approximating 240 times the thickness of the steel liner do not contribute appreciably to its buckling resistance. On the other hand, spacing the stiffeners 60 to 100 times the liner thickness will result in failure of the liner by yielding rather than buckling, and closer spacing cannot be justified.

Experimental justification for the use of Fig. 6 as a design chart is obtained from scale-model experiments on the penstock steel liner of the Picote development. It is reported that the model of this 18-ft-diameter penstock steel liner, which had a prototype thickness of $\frac{25}{32}$ in with stiffening rings at 4 ft $4\frac{1}{2}$ in, failed under an external pressure corresponding to 216 lb/in^2 in the prototype. Fig. 6 indicates that this liner ($K=275$, $\lambda=67$) had a buckling resistance of 290 lb/in^2 and hence failure must have occurred by yielding rather than by buckling.

(d) Measures to Relieve External Ground-Water Pressures. As indicated further above, the external design pressure on the steel liner of a pressure tunnel may be exerted either during the grouting operation or subsequently by high ground-water pressure when the conduit is dewatered after prolonged use. If the latter is governing, it may be economic to relieve ground-water pressures by drainage.

Drainage measures may be general or local. Where

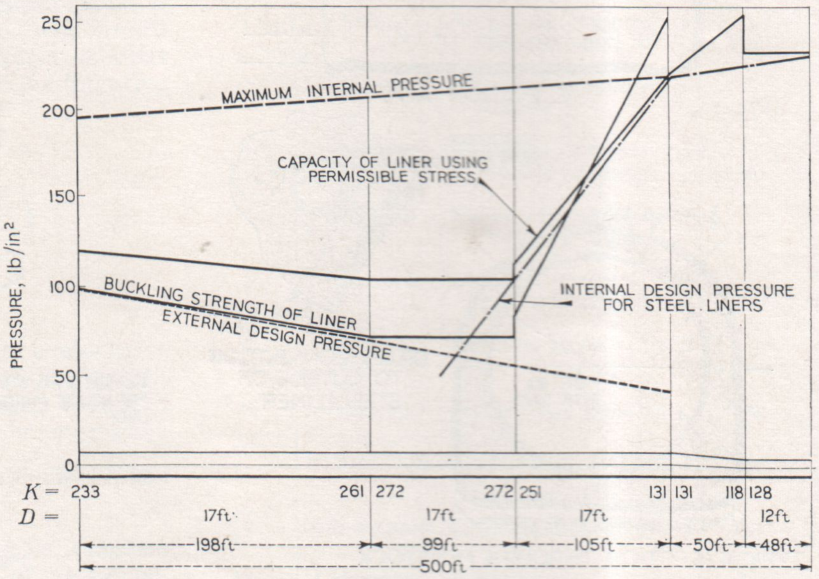

Fig. 10. Effect of external load on liner design

a steep rock face exists behind the power house, general drainage of this face may be required to reduce the risk of rock falls. In such developments, drain holes will be drilled deep into the rock face, and this will relieve external pressures on the steel liner of a pressure tunnel emerging from the face.

Fig. 11. Penstock portals of the Warsak development

Local drainage is usually provided by pipes set in the concrete surrounding the steel liner. These pipes, which are sealed during the grouting operation, are later pierced by drilling outwards through tapped holes provided in the steel liner.

Fig 7 shows the drainage measures employed at the Warsak development. Two rows of drain holes at 20-ft centres were drilled deep into the face to reduce the ground-water level generally. In addition, pipes located by brackets welded to the liner were set in the concrete around it. After completion of the grouting operation, the drainage pipes were pierced to provide local drainage around the liner.

For the Warsak development, the steel-lined pressure tunnels were only 120 ft long, so periodical clearing of drilled drain holes and penstock drain pipes is a straightforward operation. For longer or for curved pressure tunnels, effective maintenance of drains external to the pressure tunnel would be difficult if not impossible.

Installation, inspection, maintenance, and flushing of drains designed for local ground-water relief only, can be facilitated by arranging the drain within the conduit. Such an arrangement is shown in Fig. 8, representing the drainage detail employed on the penstock steel liner of the Upper Campbell Lake hydro-electric development in Canada.

Examples of Recent Installations

In the preceding sections of this article, an attempt has been made to set out the general principles involved in the design of steel-lined pressure tunnels. It is now proposed to describe the application of these principles to two recent developments in Canada and Pakistan. Both pressure tunnels have now been in successful operation for a number of years. The Bersimis* No. 1 steel-lined pressure tunnels mentioned at the beginning of this article are not described, as in that development internal pressure was the only governing criterion.

Bersimis No. 2 Steel-Lined Pressure Tunnels. The arrangement of pressure conduits constructed at Bersimis No. 2 may be seen in Fig. 9.

The main tunnel is concrete lined and circular and has a finished diameter of 38 ft. At the manifold the tunnel branches into five 17-ft-diameter penstocks which are reduced to 12 ft in diameter just upstream from the rotary valves in the power house.

The rock along the route of the pressure tunnels may be identified as the Precambrian granites and paragneisses of the Canadian Shield. The minimum rock cover over pressure conduits that were not steel lined was set at 50% of the design maximum internal pressure head. Steel liners were provided for the final 497 ft of the penstocks adjacent to the power house.

The closing of the rotary valves exerts tension in the steel liners, and for a distance of 48 ft upstream from the power house the steel liners are maintained at a constant diameter and coated on the outside surface to prevent the transfer of bond to the surrounding rock. Transition to a diameter of 17 ft occurs immediately upstream from the coated section and extends to a point 98 ft upstream from the power house. Up to this point, the steel liner is designed to withstand the full internal pressure within the usual design limit for steel. Upstream from the transition, it was assumed that the effect of the surrounding rock would increase rapidly.

A section of approximately 200 ft in length at the upstream end of the penstocks was steel lined, chiefly for the purpose of providing a waterproof membrane rather than for taking the internal pressure. A section of this portion of the steel liner is shown in Fig. 1.

The ability of the steel liner to resist external load was then checked. For this development, the governing external pressure resulted from ground water. Ground-water pressure on a section of liner was assumed equal to the depth of cover at that section. It was decided that in order to keep tunnel excavation to a minimum, additional strengths against buckling, if required, should be provided by increasing the wall thickness rather than by stiffeners or anchors. Also, this measure automatically increased the capacity of the liner to withstand internal pressure. As can be seen in Fig. 10 the external loads governed the design of the steel liner over the upstream 300 ft.

For the Bersimis No. 2 steel-lined pressure tunnels, grouting was carried out as one operation at a pressure of 50 lb/in^2. It should be noted that this grouting pressure was a substantial proportion of the critical external pressure for the steel liner.

Warsak Steel-Lined Pressure Tunnels. The Warsak hydro-electric development* is located on the Kabul River in Pakistan near the Afghanistan border, about 19 miles north-west of Peshawar.

* This scheme was described in WATER POWER, June 1957, p. 203, and July 1957, p. 243.

* This scheme was described in WATER POWER, November 1960, p. 431, and December 1961, p. 457.

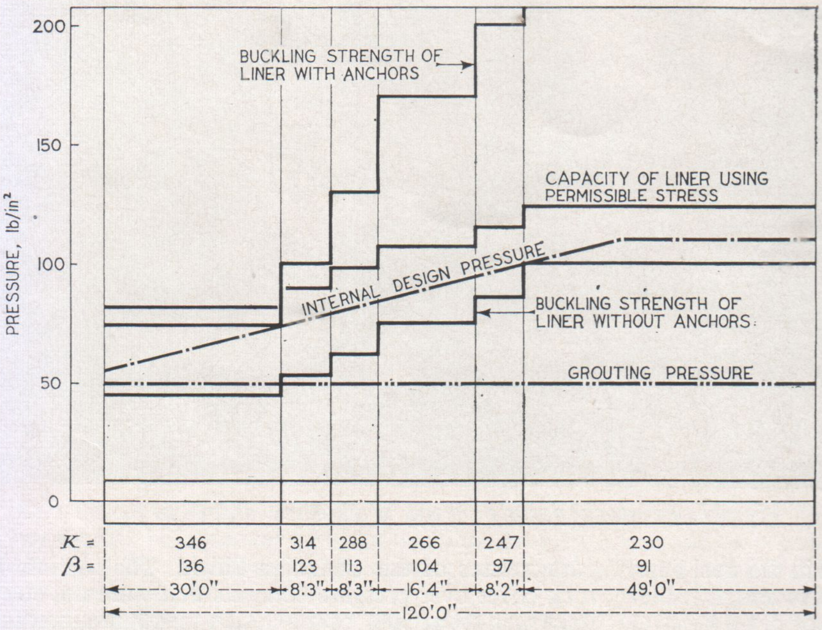

Fig. 12. Details of Warsak pressure tunnel lining

The project is in the area of metamorphic schists and gneisses in the Khyber Hills of the Safed Koh range. The local geological features are quite complex, but in general the foliation and principal joints dip downstream at an angle of 45°.

A short concrete-lined 39-ft-diameter supply tunnel conducts water from the intake to the manifold located in the hillside behind the power house. Six penstocks constructed as steel-lined pressure tunnels branch off from this manifold, horizontally and at right angles, to supply the 55,000-h.p. turbines. The gross head on this development is 145 ft, considerably less than at Bersimis No. 2.

The manifold is located approximately 150 ft back in the hillside behind the power house to avoid any danger of a landslide due to leakage from the concrete-lined section of the supply conduit. As an additional precaution, two rows of drain holes, at 20-ft centres, are drilled into the rock face at the back of the power house to relieve any seepage. The general layout of penstocks and drainage measures is shown in Fig. 7, with Fig. 11 giving a view of the six penstock tunnels at an early stage of construction.

The 120-ft-long steel linings of the 18-ft-diameter pressure tunnels form a watertight membrane which cuts off any possible seepage directly into the rock between the manifold and powerhouse.

The downstream 32 ft of the steel pressure-tunnel liners are designed for the full internal design pressure of 110 lb/in². Upstream from this section, the steel liner thickness is gradually diminished so that at its upstream end it can, without support from the rock, only take 50% of the internal pressure load at normal stresses (Fig. 12). The thickness of the liner, which is fabricated for steel conforming to ASTM Specification A201, Grade B, firebox quality, varies between $\frac{15}{16}$ in at the downstream end and $\frac{5}{8}$ in at the upstream end.

At the upstream end of the pressure tunnel, the ratio of liner diameter to thickness reaches a value of 346. At this value, Fig. 4 indicates that unless reinforcing is provided, buckling will occur at an external pressure of 45 lb/in². Grouting pressures less than 45 lb/in² are not effective in sealing up the gaps and hollow areas that may exist around the liner after concreting, and they do not induce sufficient prestress to counteract the effect of temperature differential during filling. To allow higher grouting pressures, therefore, the steel liner is anchored to the rock at eight points around the circumference, and rings of these anchors are spaced at 4-ft intervals.

The proximity of the concrete-lined manifold, and indeed the headwater of the Warsak development, to the penstock system suggests that high external pressures can also be exerted on penstock steel liners if the hydraulic system is dewatered.

A drainage system is, therefore, arranged close to the steel liners to relieve high external pressure. As described in an earlier section, and indicated in Fig. 7, a pipe located by brackets is set in the concrete surrounding the steel liners, and after completion of the grouting operation the pipe is pierced by drilling outwards from the liner through a temporary opening in it. The drilled hole from the liner then forms a ready path for the relief of high ground-water pressures around the liner.

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Butterfly Valves. The English Electric Co. Ltd. has sent us a two-colour booklet (HE/145) concerned with the butterfly valves manufactured at their Netherton works for hydro-electric and water supply schemes, oil pipe lines, and general hydraulic systems. The booklet contains a reference list, and the design of such features as dismantling joints, the main seal arrangement, and locking devices is illustrated.

Crofts (Engineers) Limited. This firm of mechanical power transmission specialists has sent us five booklets covering worm reduction gears, variable speed motor units, hydraulic couplings and drives, and shaft mounted gear units for use with rope drives, manufactured by them in their works at Bradford, Yorkshire. Dimensions and ratings for the full range of equipment dealt with, are given in these publications.