

The three main causes of deterioration in concrete, freezing and thawing, reinforcement corrosion, and chemical attack by sea water, are discussed on the basis of the fundamental concept that these causes can be generally minimized if watertight concrete is attained.

CONCRETE IN MARINE ENVIRONMENTS

By Ivan L. Tyler

THERE ARE THREE MAIN CAUSES OF deterioration in concrete marine structures: (1) freezing and thawing attack on the concrete, (2) corrosion of reinforcing steel and disruption of concrete by the resulting expansion, and (3) chemical attack on concrete by sea water.

Deterioration from these main causes could be greatly minimized or completely eliminated if the concrete was watertight. Most other concrete structures would also last indefinitely if impermeability could be attained, but in marine construction it is of utmost importance. Most exposures have seasonal drying periods; tidal structures do not and so may become saturated after a few years of exposure.

It seems evident that watertightness should be the first concern of builders of marine structures. The extent

to which watertight construction can be achieved is probably the best measure of the structure's potential lasting qualities. However the hope of building completely impermeable structures is remote because of the nature of portland cement concrete.

Since permeability is of such overriding importance, an examination of permeability curves may be worthwhile. Fig. 1 shows curves of permeability for two concretes. The top curve has a water-cement ratio of 6.5 gal. per sack, the bottom curve has 4.5 per sack. Both concretes contain entrained air. The determinations were made on moist cured 6 x 12-in. cylinders by measuring the rate of penetration of tap water under different pressures.

Moist curing for 7 days did not produce anywhere near the potential of either mix so far as permeability is

concerned. It is worth noting that after 7 days of moist curing neither concrete mix is exactly watertight. Consider the right end of the lower curve showing permeability after 28 days of moist curing. It appears to be pretty good concrete so far as permeability is concerned. But is it?

According to the method of test used in establishing the curves shown in Fig. 1, it would take non-air-entrained concrete with a permeability coefficient of $K = 30 \times 10^{-12}$ about $\frac{3}{4}$ hr for a 1-in. penetration of water under 500 psi. With $K = 1 \times 10^{-12}$, represented by the better concrete of the lower curve, it would take 20 hr for 1-in. penetration.

Assuming an average hydraulic head of 1 ft against a concrete structure, it should take a little over a year for a 1-in. saturation of a well-cured concrete of the water-cement ratio indicated, and a little less than 5 years for a 2-in. saturation with the higher grade concrete represented by the lower

curve. With air-entrained concrete the rate of penetration would be somewhat slower with K remaining the same as for non-air-entrained concrete. Clearly, if the coefficients quoted could be depended on, these rates of penetration are close to the point of complete saturation at the reinforcement. A lower water-cement ratio is desirable to prevent salt water from getting at reinforcement having 2 in. of cover.

Many things affect the dependability of permeability coefficients found by experiment. These data are presented only to indicate that what is normally considered good concrete is still within the danger area if moisture penetration alone is taken as a criterion. As will be emphasized later, the presence of oxygen is necessary for corrosion to proceed. The foregoing data should be kept in mind for the more specific problems to be discussed: freezing and thawing attack, corrosion of reinforcing steel, and chemical attack on cement paste.

FREEZING AND THAWING

The severest natural exposures in the United States are along the New England seacoast where the principal deteriorating agent is frost. There is little air-entrained concrete with a sufficiently long service record to provide reliable data. Judging from the performance of 12 x 12-in. test piling, the life of well-made 7-sack air-entrained concrete should be more than 20 years for this size specimen. The useful life

of 5-sack concrete is much less. These results were from test piling of relatively small dimensions with reinforcement embedded only 1 to 1.5 in. It has been most difficult to distinguish the effects of freezing and thawing from those of steel corrosion in this exposure. However, the effects of mix proportions are entirely clear.

Lyse¹ explains at some length the problems faced in Norway in producing frost resistant concrete for marine structures. His recommendation, based on laboratory results, was to use a low water-cement ratio with what would normally be considered an excessive amount of entrained air (10 to 12 percent) in the concrete intended for exposure.

ACI member IVAN L. TYLER, research counselor, Research and Development Laboratories, Portland Cement Association, Skokie, Ill., has served as chairman of ACI Committee 207, Mass Concrete, and 221, Aggregates. Mr. Tyler, a recipient of the Henry L. Kennedy Award, is currently a member of ACI Committee 116, Nomenclature; 211, Mix Proportioning; and 311, Inspection.

However, there are questions about the validity of laboratory tests by freezing and thawing, particularly as applied to the performance of concrete in marine construction. Such tests, to be useful, must be accelerated with respect to time, and at least one of the factors involved, that of degree of saturation, does not lend itself well to controlled acceleration. As an example, drying the specimen before test generally increases its resistance to freezing and thawing during the relatively short period of an accelerated test but any effect of continual soaking, perhaps for 25 years or more, is lost. The proposal of Powers to measure the soaking time required before freezing produces dilation of the specimen, seemingly useful for other exposures, is hardly applicable, either, because the test cycle would have to be as long as the probable life of the structure.

Laboratory tests do not tell the complete story about the probable life expectancy of continuously wet concrete under freezing conditions. But one item is apparent—completely saturated cement paste (with air voids filled) will not stand freezing without disruption.

The aggregates must also be considered. Some may become susceptible to freezing damage when embedded in concrete before the cement paste becomes saturated, and in cases of frost-damaged concrete it is not always apparent which is at fault, the cement paste or the aggregate.

To summarize the state of knowledge on concrete for marine construction in severe climates, it seems best to first admit a considerable area of

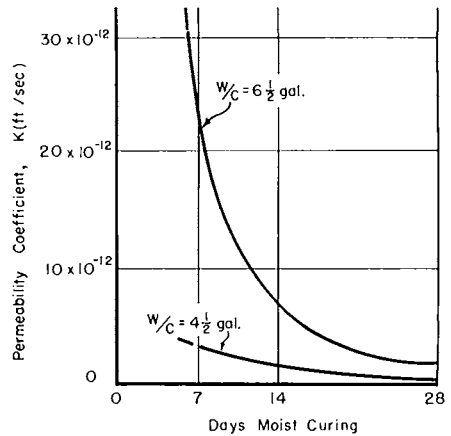


Fig. 1—Permeability of moist cured concrete

ignorance, particularly in the field of moisture content. A few items are of particular importance and should be emphasized in connection with frost resistance in sea water construction, the most severe exposure likely to be encountered by structural concrete.

1. It is assumed that all of the procedures of good workmanship, including mixing, placing, and curing, are being followed to the letter.

2. Aggregates must be "good." A good aggregate is probably best defined by its performance record in sea water construction but something of its suitability may be estimated from its physical properties. Ideally, it would be nonporous, but a porous material having a pore structure that would not become saturated might be just as satisfactory.

3. The concrete must be air-entrained.

4. The quality of the cement paste must be high, perhaps $4 \frac{1}{2}$ gal. of water or less per sack of cement to keep the rate of water penetration low.

CORROSION OF REINFORCING STEEL

Reinforcing steel rusts when it is exposed to moisture and oxygen. The presence of chloride speeds the reaction. In the process, the volume of

corrosion products formed is more than twice the original volume of the affected steel and produces expansive forces much higher than the tensile strength

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the best concrete can withstand. This somewhat simplified picture of reinforcing steel corrosion will do for most engineers concerned with the practical side of concrete performance.

Concrete with a pH of 12 to 13 normally provides a favorable environment for reinforcing steel. The presence of chloride alters this greatly by diminishing the passivity of the normal environment.

The main problem is to keep moisture, chloride, and oxygen away from the reinforcing steel. The first move is to avoid putting chloride into the concrete in the first place, as it will get there soon enough from the sea water. Extensive corrosion of reinforcement at some installations in the Pacific as a result of sea water penetration of concrete has been privately reported.

Earlier in this report, mention was made of the rate of penetration of water under pressure into concrete. An example cited 1 year to penetrate 1 in. of concrete at 1 ft head and somewhat less than 5 years to get 2 in. of penetration. Moisture may also move within the concrete by a diffusion process, and, by a similar mechanism, chloride may move from an area of

high concentration to one of low concentration. Cracks, bleeding channels, and other defects may affect moisture penetration. Laboratory experiments and field investigation show that moisture and chloride can penetrate rapidly enough to cause disruptive steel corrosion in a relatively short time—a few years, in some cases, in all but the best of concrete.

When moisture, chloride, and oxygen do get to the reinforcing steel, complex reactions can result. Corrosion cells are set up and the steel rusts and expands. Differences in salt concentration may cause conditions of severe corrosion and differences in temperature and in moisture content may contribute greatly to corrosion. One investigator in South Africa has gone so far as to recommend using salt in the concrete mix so that differences in salt concentration are less likely to occur after concrete has time to take up sea water. However, this recommendation seems highly questionable.

Out of this confusing picture of corrosion cells and factors that affect them, the builder of marine structures has one comforting fact to rely on: there is little to fear from corrosion if moisture, chloride, and oxygen are kept out.

CHEMICAL ATTACK ON CONCRETE BY SEA WATER

For the most severe exposures, deterioration of concrete by the forces of weathering is often so severe that chemical attack on cement paste by sea water is at least partly obscured. Furthermore, chemical attack is slowed considerably by low temperatures. It is in the warm climates that chemical attack assumes its greatest importance and it is here that chemical composition of cements has become of most concern. As with other problems of concrete durability, permeability of concrete is

of utmost importance. It should be stated that a great many of the reported cases of sea water attack could have been averted if a high quality concrete had been used in the construction.

Even so, there are well documented cases of sea water attack, some of this in concrete in the high quality bracket. In the author's experience, cements that are susceptible to sea water attack are usually vulnerable to sulfate exposure though the rates of attack may

not be the same. In all of the known cases, the vulnerable cements were those of high C_3A content but with only a general relationship between C_3A content and performance.

Not all concretes made with high C_3A cements have performed badly in sea water. Old structures in San Francisco Bay and others reported from South America have performed in an entirely satisfactory manner. In fact, much of the evidence against high C_3A cements rests on the performance of relatively small test specimens which cannot be expected to represent fairly the performance of full size structures.

It appears that chemical attack by sea water on concrete is a most complex subject. Much of the writing on the subject assumes that sea water attack is much the same as sulfate attack except that it is slower. It has been indicated that the presence of chloride in sea water should reduce the sulfate attack on C_3A to some

extent at least. Perhaps this explains why the more than 2000 parts per million of sulfate in sea water is not more destructive. There is some basis for a belief that sulfate may attack the calcium hydroxide released during the hardening of cement as well as tricalcium aluminate. This would support a view long held by several authorities who do not favor high lime cements for sulfate exposure. As of now it cannot be substantiated by field observations.

As in the case of frost attack, a considerable area of ignorance must again be admitted concerning chemical attack on marine structures. However, there are several statements that can be made with confidence.

1. Chemical attack is more severe in warm exposures than in cold exposures.

2. Some cements are more susceptible to sea water attack than others (cements of Type II composition supply ample resistance if used in good quality concrete).

3. Permeability of the concrete is probably the most important factor involved.

MISCELLANEOUS PROBLEMS

Besides the three major considerations that must be given to the performance of concrete in marine structures, there are some minor ones. Although not usually encountered, they appear with sufficient frequency to warrant mentioning here.

One of these would be acid contamination sometimes found in harbor areas. Unfortunately for portland cement concrete, there is no known remedy and the best that can be done is to make the concrete as watertight as possible to delay the acid attack. The use of limestone aggregate has been advocated in this connection.

The alkali aggregate reaction is

known to be aggravated by sea water exposure. Again, the use of highly impermeable concrete will greatly increase the time required for sodium from the salt to get at reactive particles.

What does prestressing do to the lasting properties of concrete in marine construction? At least it should prevent some of the structural cracking that may lead sea water directly to the reinforcement and to this extent it should be beneficial. On the other hand, it should have little or no effect on permeability, and the high tensile steel is usually more susceptible to corrosion than mild steel. Overall, any major effect seems doubtful.

SUMMARY AND CONCLUSIONS

In addition to the usual recommendations for producing high quality concrete, there are three items that should receive special consideration by the designer of marine structures, and a fourth that has been known to come into play on occasion:

1. Permeability is the property of concrete that should be of most concern. In freezing climates high impermeability delays saturation of the cement paste and to some extent it does the same for the aggregate and thus reduces damage due to frost. In mild climates it reduces the rate of chemical attack on cement paste and corrosion of reinforcing steel.

2. Air entrainment, a necessity in

freezing climate, is also an aid in mild climate by slowing the rate of sea water penetration.

3. Depth of cover over reinforcing steel, important in all climates, should never be less than 2 in. Cover of 3-in. is preferred.

4. To avoid the occasional highly susceptible cement of high C_3A content, it is recommended that the cement have moderate sulfate resistance equivalent to that of ASTM Type II.

Careful adherence to the generally accepted recommendations for high quality concrete, with particular consideration for the four admonitions given above will insure sound, long lasting marine structures.

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Presents some of the rules and lessons learned from experience, as well as analyzing a few of the difficulties and problems encountered, in the placement of high quality concrete under water by the tremie method.

PLACEMENT OF TREMIE CONCRETE

By Ben C. Gerwick, Jr.

TREMIE CONCRETE IS CONCRETE placed underwater through a tube called a tremie pipe.* The lower or discharge end of the tremie pipe is kept embedded in fresh concrete, so that washing and segregation are substantially prevented. With proper mixes and placement, extremely high quality concrete can be obtained.

Tremie concrete is used for the following purposes:

1. Cofferdam or caisson seal
2. Mass underwater concrete
3. Underwater structures (bridges piers, drydocks, etc.)
4. Repairs to underwater concrete
5. Joining tunnel sections

Tremie concrete for structural purposes frequently is reinforced. It may be used in conjunction with precast

concrete elements and with structural steel. It may be placed through any liquid lighter than fluid concrete, e.g., water or a bentonite suspension.

Principle

The aim is to introduce plastic concrete under the surface of the fresh concrete previously placed. Studies show that tremie concrete flows outward, pushing the existing surface outward and upward. As long as flow is smooth and the surface is not physically agitated, high quality concrete will result.

Mix

The proper mix is essential. The following are the author's recommendations:

Coarse aggregate—Use gravel, not crushed rock. For large masses, use 1½ in. maximum size. For smaller

*Placement of underwater concrete by buckets, which sometimes also is called tremie concrete, is not covered in this paper.

size, or where reinforcement or H-piles are to be embedded, use $\frac{3}{4}$ in. maximum size. For repairs, joints, etc., use pea gravel.

Fine aggregate—Use enough sand to insure workability; 42 to 45 percent, with 40 percent minimum.

Cement—A rich mix is essential; 7 sacks per cu yd for an average placement. Use an 8-sack mix for a small or complex placement; $6\frac{1}{2}$ sacks per cu yd minimum on large masses.

Slump—The recommended slump is 6 to 7 in. with a 5 in. minimum and an 8 in. maximum.

Admixture—In many specific cases, use of a retarding and plasticizing admixture has given excellent results with and without entrained air.

Equipment

The tremie pipes are generally eight times the size of the coarse aggregate. Pipes 10 to 12 in. in diameter are most common, but tremie grout has been placed through a $2\frac{1}{2}$ -in. hose under pressure.

A hopper is attached to the upper end of the pipe (Fig. 1). The entire assembly is lowered and raised during placement so means for this must be provided. For thin seals (3 ft), the pipe assembly may be set with a crane and supported on a frame or blocking under the hopper. For thicker

seals, the pipe assembly may be handled by a derrick or crane. A stand or frame with air hoists to raise the pipes gives the best control and eliminates jerks.

Engineers have invented, developed, and written about foot valves, cone valves, deflector valves, compressed-air rotary valves, and deflector plates at the bottom of the tremie pipe. These are neither necessary nor desirable. They generally cause plugs and laitance and have been abandoned in practice. The lower end of the pipe should be open. The object of these inventions was to prevent the sudden discharge of the concrete, but proper techniques give better control.

Tremie pipes on deep placements are long and, when raised, are hard to fill. As they are raised, sections are usually removed. The most common method is to use 10-ft sections which are unbolted and removed one by one as the pipe is raised. Good gaskets are essential. A quick, watertight coupler should be developed for this purpose. Telescoping tremie pipes and pipes with side gates also have been used with success.

The tremie pipe must be strong enough to withstand handling and lateral current pressures. Tremie pipes are usually made heavy enough to prevent floating even when empty.

CONSTRUCTION TECHNIQUE

Spacing of pipes

One tremie pipe usually serves to place about 300 sq ft of concrete sur-

face. The spacing is usually 15 ft on centers, but this varies depending on the thickness of the placement, the congestion from piles and reinforcement, and the configuration of the structure. One pipe in a 34 ft diameter caisson and two in a 35 x 90 ft cofferdam have proven entirely satisfactory but must be considered special cases. Recent trends have been to increase the spacing, and this is made more

BEN C. GERWICK, JR., president, Ben C. Gerwick, Inc., San Francisco, served on the ACI Board of Direction and as president of the Prestressed Concrete Institute. He is a member of ACI-ASCE Committee 423, Prestressed Concrete, and 343, Concrete Piles. He authored the cofferdam and caisson chapter of Handbook of Heavy Construction.

practicable by admixtures. When such a large area is to be covered that it is impracticable to set enough tremies for simultaneous concreting, pipes may be leap-frogged ahead into the advancing slope. The pipe should not be dragged through the concrete.

Starting the placement

Since the tremie pipe must be initially filled with concrete, the pipe must be sealed. The best method is to use a wood plug with rubber gasket fastened on with light wire. As the pipe is lowered to rest on the bottom, the water pressure seals the gasket and the pipe is kept dry. Concrete is now placed in the pipe. To start the concrete flow, the pipe is raised about 6 in. off the bottom. The weight of concrete pushes out the plug, and the concrete flows out to form a mound around the end of the pipe. More concrete is fed into the pipe and is maintained at a suitable height in the pipe to balance the rate of flow.

For structural concrete, grout of the same mix, less the coarse aggregate, often is used for the first few batches. This grout also lubricates the pipe.

For deep placements (over 70 ft) where the buoyancy of the empty pipe may be a problem, a go-devil may be used. The pipe is set on the bottom, with the bottom open and filled with water. The go-devil, which is a traveling plug, enters at the top, and is pushed down with first loads of concrete. It must not move too fast or the rush of water out the tube will scour the bottom or displace reinforcing steel or forms. An inside wire line has been used to control the fall; many experienced contractors, however, prefer to control the fall by having the go-devil fit tightly and adding concrete at the proper rate to control its descent. An inflated rubber ball makes an excellent go-devil and is recoverable.

The tremie pipe should be kept buried in the fresh concrete from 1½ to 5 ft depending on the rate of flow and the head of concrete in the tremie pipe. Deeper embedment gives a flatter slope provided initial set has not taken place. Here again, retarding admixtures help.

The depth of concrete in the pipe should be just enough to balance the water head and maintain flow. On deep placements, this means the concrete surface in the pipe will be deep, and continuous sounding will be necessary for control.

Batches of concrete should not be dumped suddenly into the hopper. Buckets should be opened gradually to provide a smooth, continuous flow. Stops of over 5 min are undesirable.

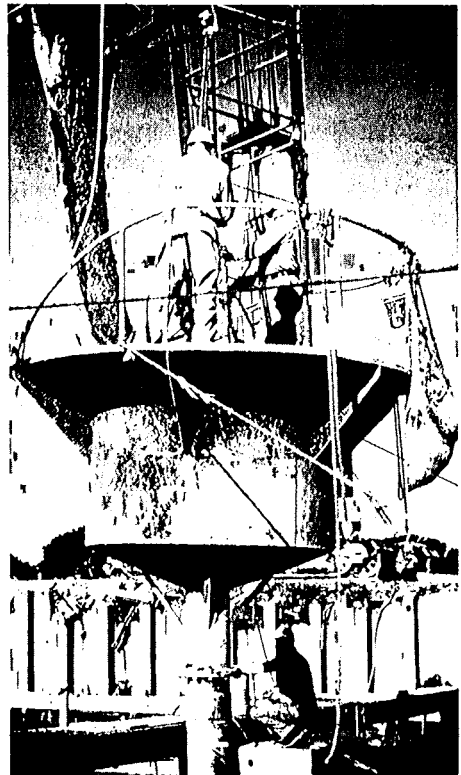


Fig. 1—Hopper attached to the upper end of the tremie pipe



Fig. 2—Placing of the seal and structural floors for a nuclear reactor

Pumped concrete has been tried, usually to the hopper. The results have been generally unsatisfactory. The proper mix for pumped concrete is not proper for tremie concrete.

Transit-mix trucks work well when discharging directly to a pipe and chutes work well in providing continuous flow.

The placement rates have varied from $1\frac{1}{2}$ to 10 ft of height per hr. Faster rates give flatter surface but pressure on the forms is higher.

Finishing the placement

In many structural installations, it is possible to overflow the containing form and thus waste any laitance and scum that may have formed. This is not possible in a cofferdam seal. Here, if felt necessary, an air-lift pump may be operated in a low corner. The placement can be started at one end and

thus any laitance and scum will flow to the corner where the pump is located. In most cases, however, this removal during placement is not employed for the average cofferdam seal.

A few hours after the placement is finished, when the concrete has set, or at least the next morning, a diver should jet the surface clean. This removes laitance before it has hardened and saves jackhammer work later. It also may be possible at this time to break up or wash down the hump that is left at the location of the tremie pipes.

Sometimes tremie concrete is used to form an underwater seat or support. In this case, it is often possible to overflow it and have a diver screed it off. This, later followed by the jet, will give a reasonably good surface, free from laitance, true, but somewhat rough.

Results

Good tremie concrete should give 28-day strengths of 4000 to 8000 psi. There is perfect curing and no drying shrinkage. The fact that it is placed under pressure, and that it is intruded, may account for its uniformity, general freedom from voids and honeycomb, and its high density which has been measured at 152 to 155 lb per cu ft.

Tests show excellent bond to steel, rock, timber, and concrete. A bond test of a steel H-pile to tremie concrete purposely placed in muddy water gave no failure at 80 psi. A design valve of 30 psi proved entirely safe. The pile itself must be clean of mud and algae.

A test on a peeled timber pile gave a value of 80 psi in combined bond and taper effect and 30 psi was safely used in design.

Several tests in which tremie concrete was placed in precast concrete boxes showed excellent bond. Cores cut down the dividing line did not reveal a seam between the tremie concrete and the box. Recrystallization may have taken place.

Tremie concrete walls placed against steel forms are equal to those cast dry. The veneer facing of dry concrete, formerly specified on naval drydocks, is unnecessary, if the forms are rigid and accurately placed.

Slopes of the surface of tremie concrete vary from 1:3 to 1:12. The average of reported results are 1:9.

Large masses and high placement rates give the best results. Slow placement may cause crusts to form which may subsequently rupture with excessive formation of laitance.

Tremie concrete has been placed successfully in water with a surface temperature of 35 F.

Form pressures

Tremie concrete generally attains its initial set in 1 to 2 hr. The water

temperature has some effect but in mass concrete the heat of hydration is the controlling factor. Retarding admixtures will produce substantial increases in form pressures. Measured pressures reach a maximum value at about 10 ft (at 3 ft per hr placement rate), or the height obtained after 2½ to 4 hr of placement. Above this level, initial set will have taken place in the bottom concrete to reduce pressure.

Measured form pressures run from 300 to 1000 lb per sq ft. With a 5 ft per hr placement rate, an 800 lb per sq ft form pressure was measured. With a 4 ft per hr rate, 600 lb per sq ft proved safe and with a 2 ft per hr rate, 500 lb per sq ft proved safe. Pressures of about 300 lb per sq ft were measured on a placement made at 1½ ft per hr.

When placing a high wall in lifts, anchors or ties must be provided to prevent the forms or sheet piles from deflecting away from a previous lift, tremie grout running down the crack, and excessive fluid head developing.

In narrow walls, there is an excessive impact head developed adjacent to the tremie pipe. This head reached 1000 lb per sq ft at a placement rate of 4 ft per hr.

Plugs, loss of seal, and resealing

Many of the difficulties in placement of tremie concrete arise from plugs, followed by loss of seal. Plugs are caused by arching, delays (over about 10 min), unworkable mix (too dry), segregation, poor aggregate gradation, and leaks in the joints of tremie pipes, thus leaching out the cement. A plug can best be freed by quickly raising the tremie pipes a few inches at a time.

When the seal is lost, the concrete runs out and water rushes back into the tremie pipe. Loss of seal should be avoided because the resealing process

always forms laitance. The best rescaling method is the use of a go-devil (inflated ball). The pipe is raised clear of the concrete, the ball pushed down almost to the bottom by concrete, the pipe reset in fresh concrete, and the placement continued. Care must be exercised not to push the go-devil down too rapidly with the concrete charge, as this would force the water out as a jet and badly wash the placed concrete.

Admixtures

Retarding and plasticizing admixtures have proven beneficial in improving workability and preventing segregation. These, often with the addition of 4 percent entrained air, have improved flowability, given a more level surface, lowered the heat of hydration, given less laitance and greater uniformity. However, they should not be used indiscriminately. Air entrainment by itself appears of no value.

With a thin seal over a larger area, the use of a retarding admixture produced a level surface (1:15) and minimum laitance. The laitance itself was soft and easily removed. Tests on a retarding admixture showed a reduction in internal heat from 150 to 120 F.

In jointing underwater vehicular tubes, a plasticizing admixture improved the flowability down, under, and back up the joint seal. A plasticizing admixture improved the workability of tremie grout used to bond steel H-piles to precast concrete.

The use of admixtures permits wider spacing of tremie pipes, because of the greater flowability and flatter slopes. The increase in form pressures on high placements must be carefully considered.

Laitance

Laitance forms when cement is washed out of the concrete by water.

Under normal conditions, a thin film will form and will be floated to the surface. Obviously, it is desirable to keep the quantity of laitance to a minimum and to avoid laitance pockets. Excessive formation of laitance often also means that strata or seams of partially cemented mortar exist in the concrete.

Laitance usually is formed at the beginning of the placement, as the concrete flows outward. Thus, it is generally proportional to the area to be covered.

The most common cause of excessive laitance is loss of seal, usually due to lack of control in raising the tremie pipe. Violent attempts to clear plugs, and accidental or intentional jerking of the pipe cause laitance and gravel seams.

When the tremie mass is flowing sideways, if it encounters a drop, laitance will gather in the low spot and be trapped. Closely spaced reinforcing bars or piles often will screen the flow, resulting in a drop. The reinforcing steel should be designed to minimize this screening effect. The largest possible bars should be used; spaced, if possible three or four times the coarse aggregate size, with a similar space between reinforcement and the form. Thus, #14 bars with a 6 in. spacing are preferable to #8 bars with only 3 in. spacing. Particular care is needed at points of lap splice of vertical bars.

Openings or "windows" sometimes have been left in dividing forms to provide keys. These are bad, since the concrete cannot be kept at exactly the same level on both sides; thus, concrete flows through and spills down, forming laitance.

Stream and tidal flow and wave action will cause water agitation and leaching of cement through comparatively small cracks in forms. The velocity of water in which tremie concrete

is to be placed should be minimized by forms or baffles, but tremie concrete has been placed in velocities as high as 10 ft per min.

Divers walking on fresh concrete, attempts to use vibrators, attempts to control and direct the flow with bottom deflectors or valves, are all causes of excessive laitance and should be avoided.

Form details

To provide keys with precast concrete walls, corrugations with large smooth undulations are much preferable to rectangular keys. There is some doubt in the author's mind that keys, even corrugations, are necessary as bond may well be more than adequate; however, further tests are necessary.

Horizontal shelves in the form of keys or structural design trap the rising laitance and water, and thus often defeat their purpose. A slope of 1:1 (or steeper) transition is much better. Where precast units sit on top of one another any necessary seats should

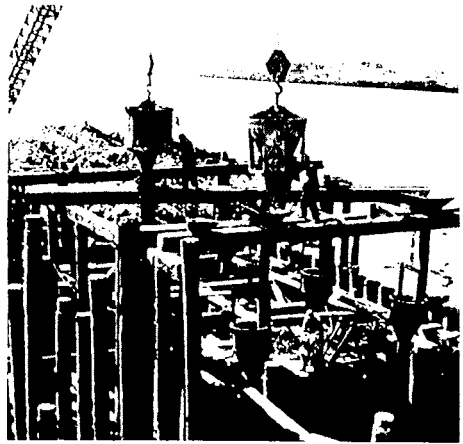


Fig. 3—Typical placement of small cofferdam seal

be external, and the inside should have this sloping transition detail, not a horizontal one.

The mortar from tremie concrete will run through small openings. In a test it ran 8 in. into a horizontal 1 x 6-in. slot. This property is useful in tremie repairs, etc., but is a problem in sealing forms. Weighted canvas will impede the flow enough to prevent loss of mortar through such joints.

COFFERDAM AND CAISSON SEALS

Cofferdam and caisson seals usually function in four ways: (1) resist hydrostatic uplift, (2) make the bottom watertight, (2) provide rigid lateral support or strutting at the base, and (4) act as a distribution block to transfer load to the piles.

With modern techniques available for placing of structural reinforced tremie concrete, it is desirable to consider constructing footing blocks as part of the tremie placement. This would reduce the dewatered depth and head on the cofferdam walls, reduce the necessary thickness of the seal for resisting uplift, and reduce the dead weight on the foundation. The placement would be completed rapidly and

economically. Much chipping of high spots and laitance, which consumes a great deal of time in the average cofferdam, could be avoided. To accomplish this, the reinforcing steel must be made up as a rigid (welded) structural frame and accurately set and held securely. The largest possible bars and spacings should be employed.

Cofferdams and caissons can generally be pumped in 3 to 6 days depending on stresses involved. Tremie concrete will generally develop 2500 to 3000 psi in 7 days.

Anchorage to resist uplift can be provided by piling or by drilling and grouted anchors. The piling must be clean of mud and algae.

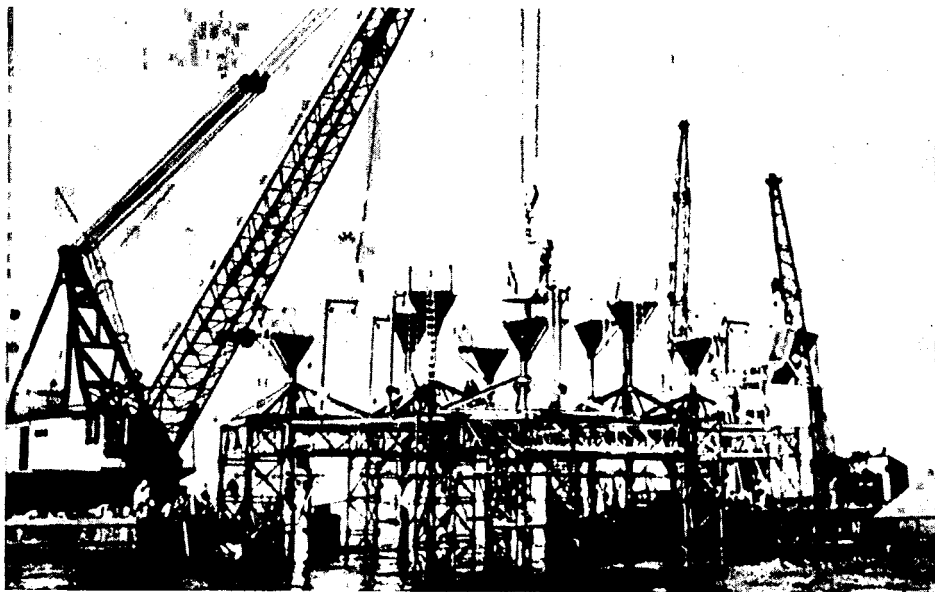


Fig. 4—Placing one of the main piers of the Richmond-San Rafael Bridge by tremie

Sometimes a low spot is left or a sump is formed in one corner to aid dewatering. A weighted box or form can be set. Since the seal is reduced at this point, it must be checked structurally. Sufficient thickness must be left and details made properly to insure good tremie concrete and prevent accumulation of laitance around the sump. However, many experienced contractors prefer to chip the sump with air tools after dewatering.

On thin seals, it is desirable to start, 6 in. low. Where a reinforced footing block is to be concreted later in the dry on top of the seal, the tremie placement should be stopped 6 in. below elevation. This will minimize chipping of high spots to clear the reinforcing, and the deficiency in height will be made up in the dry placement.

Controlled relief holes (valved pipes) formed in the tremie seal will often be useful in reducing the uplift pressure after dewatering while allowing the inflow to be controlled.

Cleaning before placement

Where sheet piling and bearing or uplift piling are driven before excavation, soil will be wedged in arches and between flanges. These should be jetted clean to insure bond and continuity of the seal, and thus prevent a blow. Algae often form rapidly in the quiet waters of a cofferdam and must be cleaned off to insure bond.

For structural concrete, where a firm bond of the new placement to rock or concrete is desired, it may be necessary to pre-clean the surface. Loose gravel in small quantities is not objectionable, as it will become embedded in the concrete. Normal muddy water is not detrimental to a good tremie concrete. Silt and mud in large quantities should be jetted and removed by air-lift. In smaller quantities it can be stirred up thoroughly by a jet. The silt will be in suspension in the water and rise on top of the concrete surface. Sand and clayey sand should be removed by air-lift; otherwise, it will tend to be trapped.

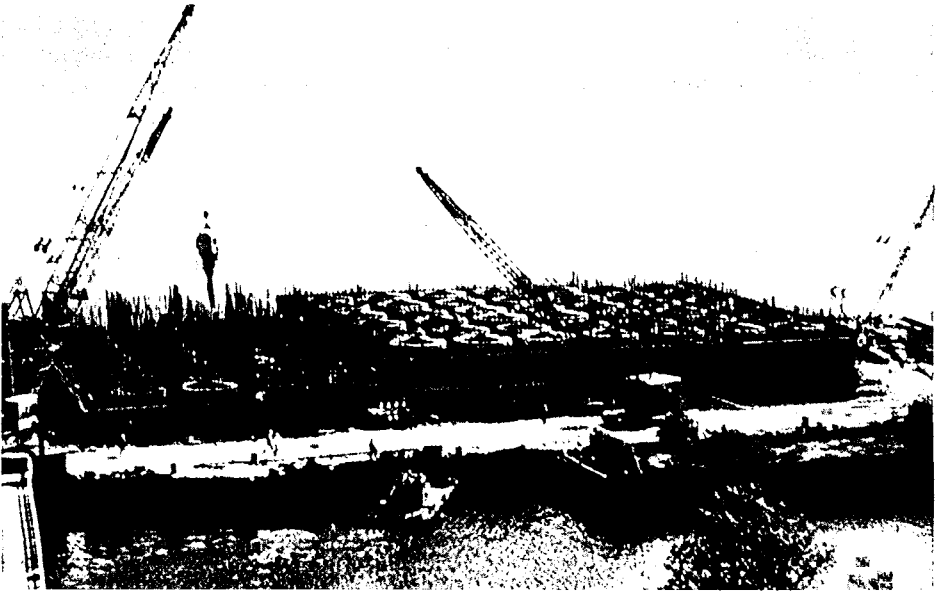


Fig. 5—Caisson of Verrazano-Narrows Bridge

CONSTRUCTION JOINTS

Vertical construction joints can be formed with steel forms, or corrugated sheets or sheet piles which are later removed. A precast concrete divider wall or a well-anchored steel form may be left in place.

Satisfactory horizontal construction joints require: (1) the lower placement to be as level as possible; (2) the surface to be jetted clean and laitance removed several hours after placement; (3) excessive silt and sand be removed before the next placement; and (4) the placement to be started with a rich tremie grout.

Where there is a curtain of reinforcing steel around the perimeter, the first placement will generally have a step down of about 1 ft outside the bars. Particular care should be taken to jet and clean the laitance from these pockets. (There may be similar pockets behind H-piles.) At the start of the second placement, tremie grout should be placed to fill this outside

pocket up above the level of the remaining concrete. This usually requires a diver. If this is not done, the laitance from the new placement will flow into and be trapped in this structurally important area.

To prevent excessive screening by reinforcement near a construction joint, lap splices in the vertical bars should be located so the end of the upper bars are 1 to 2 ft above the construction joints.

Logistics

Tremie concrete volume may be 5000 cu yd or more on large projects. Placing rates on such major projects often run 100 cu yd per hr, with an extra hour or two at the beginning, and also at the end for topping off. The logistics of maintaining such continuous placement is complicated and requires careful planning.

Where transit-mixed concrete is used, availability of trucks,

from train crossings and drawbridges, and alternate sources in event of plant breakdown must be considered. A sufficient quantity of aggregate must be available at the plant and cement must be stockpiled or deliveries arranged to insure continuous production.

For site batching, adequate supplies of sand, gravel, and cement must be on hand and delivered during the placement. Frequently, a fresh water supply barge is needed. Fog, tides, and rough weather must be anticipated. On one 5000-cu yd placement, over 30 individual barge movements had to

be scheduled to an exacting timetable. Large projects require the on-site availability of complete standby equipment for every phase of batching, mixing, delivery, and placing.

Notable tremie concrete projects

A great many interesting and important smaller projects for special structural purposes and for repairs have been carried out successfully with tremie concrete. Underwater walls, joining seals of underwater tubes and pipelines, intake structures, pumphouse floors, are some examples.

Some of the larger tremie concrete projects include: (1) Detroit River tunnel, 1906; (2) Pearl Harbor drydock 1909 - 1913; (3) New York drydocks (450,000 cu yd of reinforced structural concrete in steel forms), 1944. (4) San Francisco drydocks (reinforced structural concrete in precast concrete forms), 1945; (5) Potomac River bridge, 1940; (6) Chesapeake Bay Bridge, 1950; (7) San Mateo transmission tower, 1951; (8) Richmond-San Rafael bridge (structural tremie concrete working in composite action with reinforced structural shells), 1953; (9) Delaware Memorial bridge (with a single seal of 26,670 cu yd), 1955; and (10) Verrazano-Narrows bridge, 1962.

SPECIAL CASES

Cyclopean concrete walls have been constructed by tremie concrete. Approximately 3 ft thick layers of 12-in. rock are placed and the voids filled by tremie concrete. The front face of such a wall is usually formed by precast slabs or steel sheet piles. The back face takes a natural slope. Obviously, a smooth flow is not deposited under the surface, and considerable laitance is formed. However, this flows down the back side and is wasted. The tremie concrete does a good job of fill-

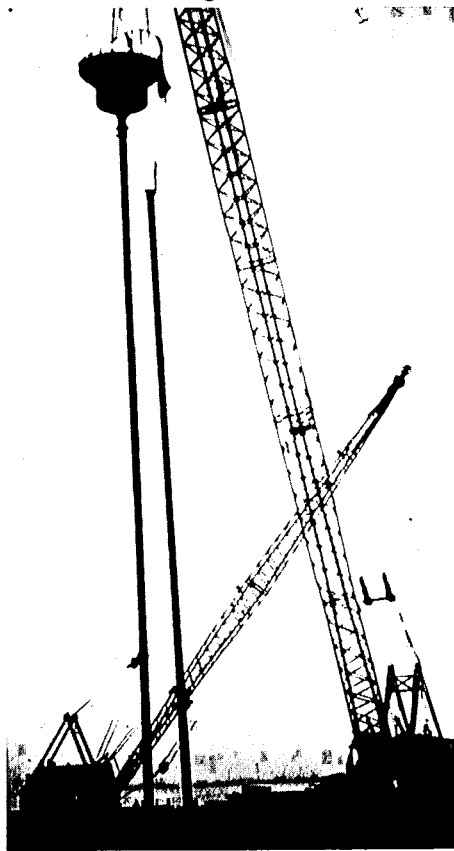


Fig. 6—Tremie concrete for the Verrazano-Narrows Bridge was placed at a depth of 170 ft. requiring the pipe to be set in sections

ing the voids and bonding with the rock. It is important that concentrations of fines in the rock be avoided. The tremie method is believed superior to the alternate method of bottom-dump buckets because the flow is smoother and more continuous.

Several unreinforced mass concrete cut-off walls and bulkheads have been built by placing tremie concrete through bentonite. The bentonite held the earth during excavation and is largely recoverable and reusable as the concrete wall advances.

In Milan in 1961-1962, this same process was used to construct the reinforced structural walls of the subway. The wall was constructed in sections, with a formed vertical joint. Inspection of these walls showed surprisingly uniform, good concrete. There was slight weeping at the vertical construction joint under hydrostatic head, but this would undoubtedly have occurred with concrete placed in the dry. At several places the author dug into the concrete to inspect bond with the deformed reinforcing bars. A trace of bentonite was trapped under the deformations, but otherwise the surface between bar and concrete was clean.

Problems

Improperly placed tremie concrete can lead to serious difficulties. On one project of several thousand yards of tremie concrete, the night superintendent was rushing to set a record. Buckets were opened quickly and pipes were raised rapidly to get the concrete to flow faster. When plugs formed, the pipes were raised and dropped with a jerk to try and free them. A record was indeed set, but subsequent diver investigation showed seams of gravel and trapped pockets of laitance, all buried under sound structural concrete. This probably would not have been too serious as a mass concrete

seal, but this was a main bridge pier. The gravel seams and laitance pockets had to be chipped out and refilled with tremie grout, all work being done underwater with divers. This was a costly and time-consuming mistake.

Holes, or openings sometimes are accidentally left under, around, or through divider walls. Tremie concrete will flow and spill through these and, even though much of it turns to laitance, much good concrete will be placed erroneously in the wrong spot and may require laborious and expensive removal later. Thorough checks should



Fig. 7—Core through tremie concrete into rock, showing bond.

be made of keyways, handling holes through sheet piles, etc., and the space under forms should be sealed with canvas, sandbags, or gravel.

A caisson seal was placed properly and subsequently pumped out under a true effective head of 80 ft and appeared at first to be a perfect seal. After 2 weeks, considerable leakage developed in an oblique seam about 1/16 in. wide. This seam was probably aggravated by heat of hydration and drying shrinkage, and certainly could have been expected in concrete

placed in the dry. However, its origin probably was due to the use of several tremie pipes and the seam represents the junction line of two masses. This would indicate the desirability of having one pipe lead the others by a considerable height, to make sure the concrete from the secondary pipes is intruded under the other. In this case, the seam was sealed completely by pressure grouting, but only after considerable experimentation in getting a grout mix and technique that would hold long enough to set.

CONCLUSIONS, ACKNOWLEDGMENTS, AND REFERENCES

Tremie concrete, therefore, is an excellent means of placing high quality concrete underwater but requires especially careful control. It is an art, and the individual workmen have a great influence on its success. They must be trained, and even rehearsed, and above all, properly supervised.

The author is indebted to Ben C. Gerwick, Sr.; to William J. Talbot, J. H. Pomeroy & Co., Inc.; to Gene Rau, J. Rich Steers, Inc.; and to J. Wayman Williams, Jr., of Sika Chemical Corp., for information and data appearing in this paper.

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Principal joint treatments are discussed, particularly the rigid and flexible pre-formed types and field applied sealers. A review is also made of construction joint requirements, and methods and materials used in repair of cracks.

JOINTS AND CRACKS IN CONCRETE WATER-HOLDING STRUCTURES

By George B. Wallace

GOOD QUALITY CONCRETE MADE according to ACI standards is for most engineering uses practically impermeable. Therefore, the watertightness of structurally sound barriers and containers made of such concrete and exposed to hydrostatic pressures depends upon the watertightness of the

joints and their effectiveness in preventing cracks. Even with extensive knowledge of the properties and uses of concrete, it is seemingly impossible to prevent cracks in concrete structures. When cracks occur, effective repairs are required to maintain the integrity of water-holding structures.

JOINT FUNCTIONS AND TYPES

When an unrestrained concrete element dries or cools uniformly, it contracts, producing strain without stress. Similarly, chemical and physical changes occur as concrete ages, causing autogenous shrinkage. On the other hand, concrete cast against rock or older concrete, which has essentially completed shrinking, is restrained from

contracting by its bond to the fixed surfaces. The bonded surfaces provide a restraint against which contractive strains cannot occur without causing proportional stresses.

Contraction joint

When such stresses exceed the low tensile strength of new concrete, crack-

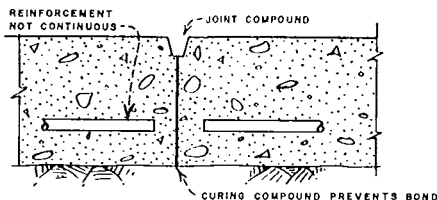


Fig. 1—Contraction joint

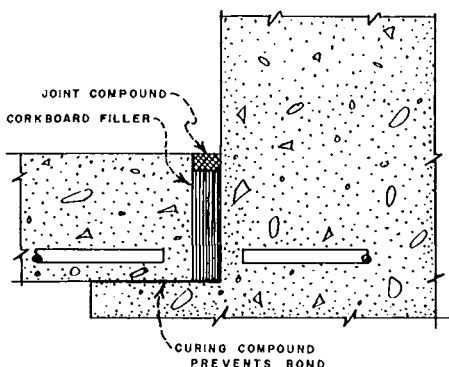


Fig. 2—Expansion joint

ing will occur. Well-spaced contraction joints divide restraining forces so that contraction results in opening of the joints rather than random cracking of the concrete. A simple contraction joint is shown in Fig. 1. Many contraction joints are more complex but all have two common properties: there is no bond between the joint faces, and reinforcement is not continuous across the joint.

Expansion joint

When unrestrained concrete is wetted or heated, it expands. Expansion may also occur during the aging process, due to chemical reactions between cement and certain aggregates. These

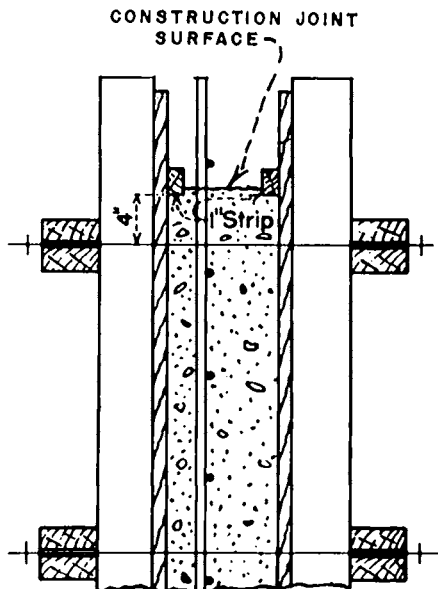


Fig. 3—Construction joint

expansions usually develop compressive stress which good quality concrete can safely withstand. However, when a relatively light concrete member adjoins a massive or fixed member or when a long thin slab is subject to buckling, an expansion joint of compressible material should be provided. A simple expansion joint is shown in Fig. 2. Expansion joints also serve as contraction joints, therefore, tension steel is not placed across these joints. Unbonded dowels can be installed across expansion or contraction joints to carry shear forces which might otherwise cause displacement of the structure.

Construction joint

Cracks due to vertical shrinkage are virtually eliminated, without using horizontal contraction joints, by placing concrete in horizontal lifts of limited height. Between horizontal lifts, properly bonded construction joints are sufficient to seal out water and

ACI member GEORGE B. WALLACE, head, Concrete Materials and Structures Section, Division of Research, Bureau of Reclamation, Denver, is chairman of ACI Committee 504, Joint Sealants. He supervises field and laboratory tests of rock foundations for Bureau dams and various structural tests of metals, masonry, rubber, and plastic materials.

transmit induced stresses between the lifts. Both horizontal and vertical construction joints are sometimes required to facilitate construction. However, bonding of the vertical joint surfaces is more difficult and cannot so readily be relied upon for watertightness. A simple forming technique to achieve a straight, neat appearing line at construction joints is shown in Fig. 3. Note that in contrast to the contraction and expansion joints, reinforcing steel may be continuous across the joint.

Methods and materials used to pre-

vent flow of water through various types of joints may be required to resist many physical and chemical forces, including hydrostatic pressure, tension induced by contraction, shear caused by foundation deformation, and deterioration by severe weathering, intense sunlight, extreme heat or cold, and aggressive chemicals. Joint materials subject to frequent reversal of movement or vibration must also be capable of resisting fatigue. Table 1 lists information on joint materials used by the Bureau of Reclamation.

JOINTS IN DAMS

Placing of concrete in large masses at the rapid rate now commonly practiced in construction of large dams tends to develop unfavorable stresses during the cooling period. To prevent serious cracking, a system of contraction joints capable of relieving the stresses must be provided. Cracking is controlled in many dams by use of transverse joints, normal to the axis of the dam (Fig. 4). However, for the larger dams such as Hoover, Grand Coulee, and Glen Canyon, longitudinal joints are also employed. These joints are parallel to the axis of the dam and are offset at the transverse joints to avoid a continuous longitudinal joint plane through the structure.

To hasten cooling and shrinkage of concrete blocks formed by the contraction joints, refrigerated or cold river water is often circulated through metal coils embedded in the concrete at each horizontal construction joint. The construction joints divide the blocks into lifts which are about 5 to 7½ ft in height. Each lift is constructed by uninterrupted placement of 16- to 20-in. layers of concrete, spread and vibrated in such a manner that

the layers are knitted together without the formation of cold joints.

Each block is mechanically interlocked with adjacent blocks by vertical keys along the transverse joints. Although the keys restrain large upstream and downstream movements, each block is free to contract without significant restraint from adjacent blocks. After cooling is completed and the joints are opened, portland cement grout is pumped under pressure into the joint openings to assure a monolithic structure capable of transferring operating stresses across the joints.

Joint spacing

Spacing of contraction joints is a most important consideration. If joints are too far apart, excessive shrinkage will produce cracks in the blocks. Conversely, if the joints are too close together, shrinkage may be so slight that the joints will not open enough to permit complete grouting. For average conditions, a transverse joint spacing of 50 to 60 ft has proved to be satisfactory. Upstream-downstream length of blocks is normally limited to about 180 ft. For dams of thicker

TABLE I—REFERENCE SPECIFICATIONS FOR JOINT MATERIALS

Material	Bureau of Reclamation uses	Specifications title and number
Rubber waterstop	Powerplants - expansion and contraction joints Canal structures - expansion and contraction joints	USBR Guide specifications (tested in accordance with FTM standards 601)
PVC waterstop	Canal structures - expansion and contraction joints	USBR Guide specifications (tested in accordance with FTM standards 601)
Rubber gasket	Concrete pipe joints	USBR Guide specifications (tested in accordance with FIM standards 601)
Metal seal	Dams - contraction joints Powerplants-contraction joints	Federal Specifications—Steel plates, sheets, and strip; corrosion resisting (QQ-S-766c)
Corkboard joint filler	Powerplants - expansion joints	Federal Specifications—Filler expansion—joint performed, nonextruding, and resilient types, for concrete (HH-F-341a, Type 1, Class A, cork, 11-1-61)
Sponge rubber joint	Powerplants - expansion joints Canal structures - expansion joints	Federal Specifications—Filler expansion—joint performer, nonextruding, and resilient types, for concrete (HH-F-341a, Type 1, Class A, 11-1-61)
Hot-poured asphalt joint sealer	Dams - roadway joints Powerplants - roofs, horizontal joints	Federal Specifications—Sealer, hot poured type, for joints in concrete pavements, bridges and other structures (SS-S-164, 2-1-52)
Cold-applied asphalt joint sealer	Canal lining - contraction grooves expansion joints	Sealing compound, rubberized, cold application, ready mixed for joints in concrete canal lining (USBR, 2-25-60)
Polysulfide-base joint sealer	Powerplants-vertical expansion joints and contraction joints	Federal Specifications—Sealing Compound, Rubber Base, two components, for calking, sealing, and glazing in building construction (TT-S-00227, 7-17-61)
Rubber joint strip	Powerplants - floor expansion joints exterior joints below grade	USBR Guide Specifications (tested in accordance with FTM Standards 601)
Portland-cement mortar	Concrete pipe joints	USBR standard mortar requirements
Coal-tar dampproof coating	Coating for corkboard filler	USBR Guide Specifications, (tested in accordance with appropriate ASTM standards)
Vinyl resin damp-proof coating	Coating for corkboard filler	USBR Specifications for vinyl resin paint (VR-6, 7-1-60)

section, longitudinal joints are usually provided.

In general, horizontal construction joints do not require a seal if the hardened concrete face has been properly cleaned and roughened to insure tight bonding of the subsequently-placed concrete. Good workmanship is a primary requirement for effective preparation of the bonding surface. Wet sandblasting, as shown in Fig. 5, is done just prior to the next concrete replacement. This procedure is favored by the Bureau of Reclamation over the commonly used air-water jet "green cutting" method. Applied too soon, a jet will erode too much concrete and loosen aggregate particles at the surface; if applied too late, it

will be ineffective in removing laitance. However, the use of low pressure (about 40 psi) water jets is sometimes permitted in locations difficult to sandblast, provided the fresh concrete surface has been sprayed with a concentrated retarder. This treatment retards setting of the surface concrete to a depth of about 3/16 in. Therefore, the application of the water jet may be delayed until the untreated concrete immediately below the surface has gained sufficient strength to resist erosion.

Present Bureau specifications require spreading of a layer of mortar over the old concrete surface immediately prior to the next concrete placement. However, laboratory tests and field experience show that equally good

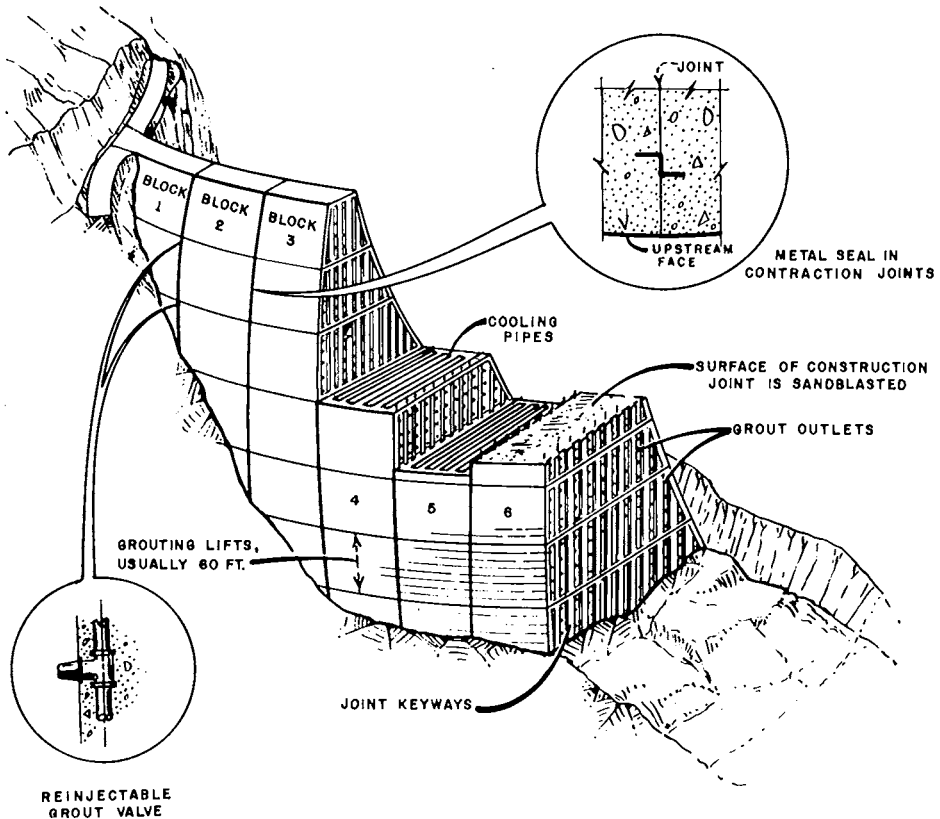


Fig. 4—Joint systems of a concrete dam showing use of seals and grout valves.



Fig. 5—Preparation of construction joint by wet sandblasting; cooling pipe in place, and vertical joint keys in background

bond can be obtained without a mortar bedding and consideration is being given to modification of this requirement in future specifications.

Contraction of the blocks, accompanied by opening of the joints, creates passages through which water from the reservoir could flow. To prevent this, metal seals are installed in the contraction joints adjacent to the upstream and downstream faces, as illustrated in Fig. 4. These seals serve as both water and grout stops. Current practice requires two such seals, about 5 ft apart at the upstream face. They are also placed horizontally across the joints to define upper and lower extremities of grouting lifts. Two standard metal seals, shaped like the letter Z and the letter M, are used in Bureau structures. The Z-type seal is easily installed, but is limited to locations where little joint movement is expected. This seal is well adapted to contraction joints which are to be grouted, since grouting tends to restrict joint movement. The M-type seal has an expandable shape which can accom-

modate greater movement at a joint and better withstand repeated deformations.

Current specifications for metal seals in dams require corrosion-resistant stainless steels annealed to maintain flexibility. The material must be of low carbon content or stabilized with columbium or titanium to facilitate welding and to retain resistance to corrosion after welding. Stainless steel seals will withstand grouting pressures as well as impact from concrete placement, hydrostatic pressure, and corrosion.

Grout valve

In contrast to Bureau practice, several European dams have been constructed without artificial cooling. European engineers have developed a reinjectable grout valve which permits joints to be regouted several times as they continue to open during the lengthy air cooling period. One type of reinjectable grout valve consists of a pipe T and a deformable rubber cap illustrated in Fig. 4. Grout forced into the valve compresses the rubber cap, permitting the grout to flow into the contraction joint. Upon release of pressure, the cap reseats on the T, thereby completing the initial joint sealing operation. Water is then circulated at a low pressure to flush the grout from the valve and connecting pipes. The system is then ready for another injection if such is required at a later date.

The reinjectable grout valve is being used at Glen Canyon Dam for quite a different reason. Here, the sandstone abutment rock deforms under load in general accordance with Young's modulus but exhibits a considerable amount of permanent set when load is relieved. Thus, when the reservoir is lowered following initial filling and the arch returns to its un-

loaded position, an opening may form near the abutment. Installation of the reinjectable valves in these locations will enable such an opening to be refilled. Actually, it is planned to preload the abutment blocks by applying a uniform pressure up to 200 psi in the end contraction joints. It is expected that this will induce most of the permanent set prior to initial grouting of the abutment blocks. However, if such is not the case the reinjectable grout valves may be used to fill subsequent openings.

Joint systems should be designed for adequate protection and good workmanship must be used in constructing joints. Care must be taken

to insure that joint seals do not become wrinkled or torn and that concrete adjacent to the seals is thoroughly consolidated. Even a small seep through a joint, in conjunction with freezing and thawing, erosion, and other forms of deterioration, may cause damage to a dam. Continued deterioration over a long period could cause a high dam to become unstable. Thus, even small defects in joints can result in expensive repairs, requiring drainage of the reservoir and consequent loss of power and water revenues. Inspection and detection to prevent use of substandard materials and methods are basic safeguards against leakage.

JOINTS IN POWERPLANTS

Layouts for multi-unit surface powerplants are governed by their waterways (penstocks, turbines, and draft tubes), generators, transformers, and overhead cranes. Elevations are usually set so that access to the plant at the generator floor level will be slightly above the tailwater surface.¹ Below this level, these structures may properly be classified as water-holding structures as indicated in Fig. 6. In addition to the usual joints to facilitate construction and prevent unsightly cracks, joint designs are required to protect machinery against misalignment due to building distortions. Compressible joint materials between generator units also dampen transmission of vibratory stresses. Certain joints below grade are designed to permit refilling with hot asphalt should the original asphalt be displaced.

The exact position of a joint is governed by structural, architectural, and construction requirement. However, joints in multi-unit plants are usually provided in the following general areas: (1) between groups of two

or more power units, (2) between units of a building having wide variations in cross section, (3) between portions built on foundations having different bearing values and (4) where buildings are weakened by openings. For smaller plants of one or two units, major expansion joints are not usually required.

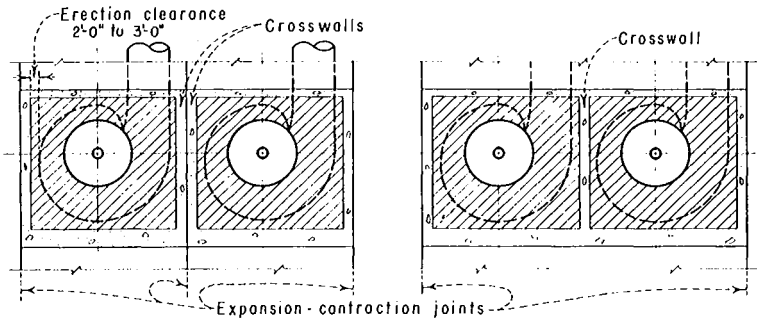
Two-unit bays may often be combined into one monolith. This technique eliminates one cross wall, as shown in Fig. 6, with a saving in building length of 3 to 5 ft for each pair of units so combined. This may be especially advantageous in a narrow canyon, but it requires simultaneous construction of the two-unit bays. This increases the cost of formwork and decreases the flexibility of the contractor's operations. For these reasons, monoliths of more than two-unit bays may be less economical than a structure divided into independent parts by expansion joints running completely through the building from footing to roof deck. Each part should be capable of moving due

foundation settlements, without damage to adjacent parts.

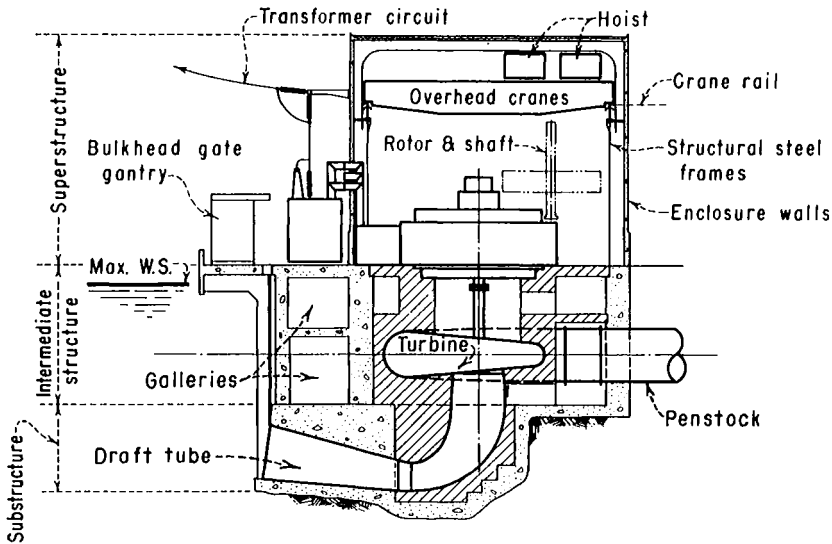
Fig. 7 shows basic joint details for powerplant construction. The substructure shown in the lower part of Fig. 7 consists of massive concrete footings and walls which absorb vibrations and carry the heavy machinery loads to the rock foundation. Considerable external water pressure from the tailrace

may be exerted on the joints. Therefore, multiple seals, capable of preventing passage of water into the plant even though foundation displacements occur, are often used in these joints. Combinations of rubber and asphalt seals have performed well under such conditions, in both expansion and contraction joints.

Asphalt seals are constructed by



Plan view at turbine level



Vertical section through powerplant

Fig. 6—Principal powerplant joints are located to facilitate construction and minimize building length

forming an opening of square cross section along the joint, as shown in Fig. 7. This formed opening is then filled with hot asphalt. Steam pipes are embedded in the seal to relieve the asphalt when opening of the joint

may require additional filling. Primary protection is provided by the use of rubber waterstops placed across the joint, one on each side of the seal. These will also serve to contain any asphalt that seeps from the seal.

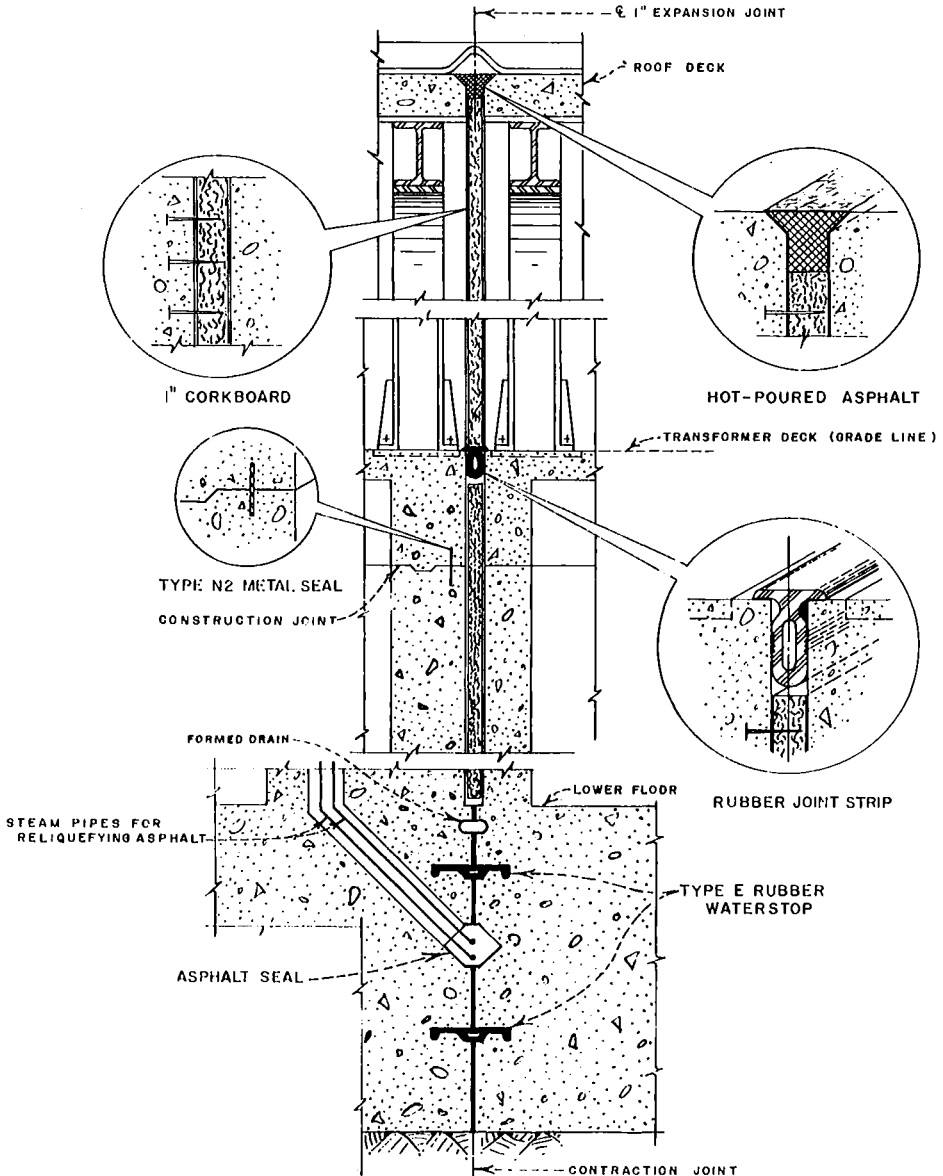


Fig. 7—Typical joints in powerplants

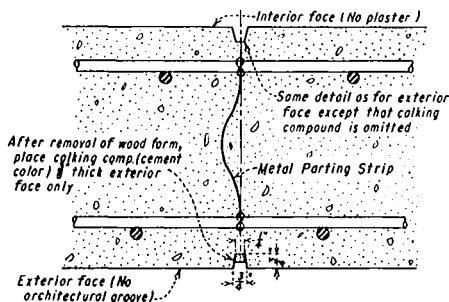


Fig. 8—Control joints relieve stress concentrations near openings in building

Multiple seals have also proved useful in dams where considerable movement is expected. In this case, metal waterstops are usually substituted for the rubber waterstops adjacent to the asphalt seals. Similarly, asphalt seals containing resistance heaters have been used in hollow-gravity Bissina Dam in Italy. These accommodate displacements caused by temperature, water loads, and differential settlement of the buttresses.

As work on a powerplant progresses, horizontal and vertical construction joints are required in order to divide the structure for concrete placements of a convenient size. In walls crowded with reinforcing steel, preparation of joint surfaces is difficult. To insure watertightness, the keyways and metal seals shown in the middle portion of Fig. 7 are installed in the horizontal construction joints below tailrace water surface. These seals, usually of stainless steel, are placed parallel to the face of the concrete at right angles to the joint. The lengths are welded together to form a continuous protective diaphragm.

Above grade, structural steel columns and beams have often proved economical when used in conjunction with curtain walls and roof decks. Details of a typical expansion joint through a curtain wall and roof are

shown in the upper portion of Fig. 7. Filler materials for such joints are pre-molded corkboard or sponge rubber. The sponge rubber is especially suitable where it is desirable to minimize load transfer through an expansion joint. In vertical joints, the filler material may be secured to the concrete surface by nails precast in the first-placed concrete or by a waterproof cement.

Surface seals

To seal the exposed surfaces of joints, a polysulfide-base rubber compound is used except where it would come in contact with bituminous dampproofing material, which is often used as a coating on corkboard filler. Deterioration of polysulfides by vapors from bituminous coatings has recently been experienced. Current research by the Bureau indicates this deterioration may be prevented by coating corkboard with an innocuous vinyl resin material in lieu of reactive bitumens. Hot-poured asphalt is presently being used in roof joints where the sealant is in contact with bituminous roofing materials.

A protective rubber strip has been developed for use in floor joints. (Fig. 7). This strip is hammered or pressed into place and stays securely locked in horizontal joints. It is also desirable for use in exterior walls below ground where backfill will retain the strip as well as below water surface if secured by metal strips. The rubber provides effective protection against spalling or breaking of the concrete edges.

Control joints

Metal strips cast in concrete walls adjacent to doors and windows induce planned cracks referred to as control joints. Such joints are often more effective than additional reinforcing steel in

keeping building walls free from random cracking near openings. Control joints, illustrated in Fig. 8, utilize parting strips of sheet steel waxed on one side and wired to the reinforcement. They are installed at regular intervals along the building walls. Narrow grooves are formed on both exterior and interior faces in line with the parting strip to further weaken the concrete and assure a neat, inconspicuous crack. Where grooves are exposed, they may be filled with a joint compound of a color to match the concrete.

On occasion, it is desirable to provide a waterstop which can be serviced or replaced after installation. The arrangement shown in Fig. 9 has proven

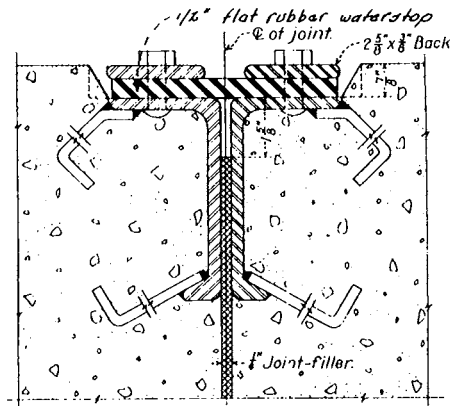


Fig. 9—Exposed serviceable or replaceable rubber waterstops

useful in powerplants or dams where joints intersect inspection galleries.

JOINTS IN CANAL STRUCTURES AND PIPELINES

Rubber waterstops have a long history of effectively sealing joints in canal structures and pipelines. They were first used by the Bureau in structures for the All American Canal in 1935. Since that time, there has been a great deal of development in both rubber and polyvinyl chloride (pvc) waterstops. Natural rubber generally has some advantage in recovering from larger deformations. Pvc, on the other hand, is not as vulnerable to deterioration from sunlight and oxidation as are the rubber products. It has been reported that when pvc is immersed in an alkaline solution comparable in concentration to concrete laitance water it is subject to extraction of plasticizing agents. In this case, the amount of extraction under prolonged immersion was relatively small and the consequent hardening and reduction in tensile strength and elongation were not considered objectionable for most purposes.²

Continuing inelastic deformation of a waterstop under uniform loading is

undesirable since the material extending across the point opening may be damaged on subsequent closing of the joint. Tests in the Bureau laboratories are in progress to determine the creep properties of both pvc and rubber waterstop materials. At present, the objectionable features of both rubber and pvc waterstop do not preclude their use under ordinary joint conditions. The Bureau has specified plastic waterstop as an alternate for rubber waterstops, except for pressure conduits.

Waterstops

Waterstops of the Type A and B shapes shown in Fig. 10 are widely used in Bureau canal structures.³ These shapes are specified for either rubber or plastic materials. Center bulb, 9 in. rubber waterstops have been tested as shown in Fig. 11 and elongated as much as 5 in. without failure. Most waterstops are employed by embedding one-half of the seal on each side of the joint. In the center bulb type, the enlarged section at each end

provide an anchor against displacement of the waterstop, and the hollow center bulb allows transverse movement without damage to the concrete. Rubber waterstops with this shape are specified for large cast-in-place concrete siphons such as those shown in Fig. 12, and in large horse-shoe shaped conduits frequently used to convey normal river

flow and irrigation releases under earth dams.

Other shapes such as the labyrinth shown in Fig. 13 are easier to install and are not so apt to become displaced during concrete placing operations as the dumbbell and center bulb types. However, the rigidity of the labyrinth seals may cause failure of the concrete surrounding the waterstop if large joint deformations occur (Fig. 14). Less rigid, corrugated waterstops offer considerable promise for sealing joints in irrigation structures, but so far have not been used by the Bureau.

In addition to containing an approved waterstop, expansion joints in canal structures are filled with sponge

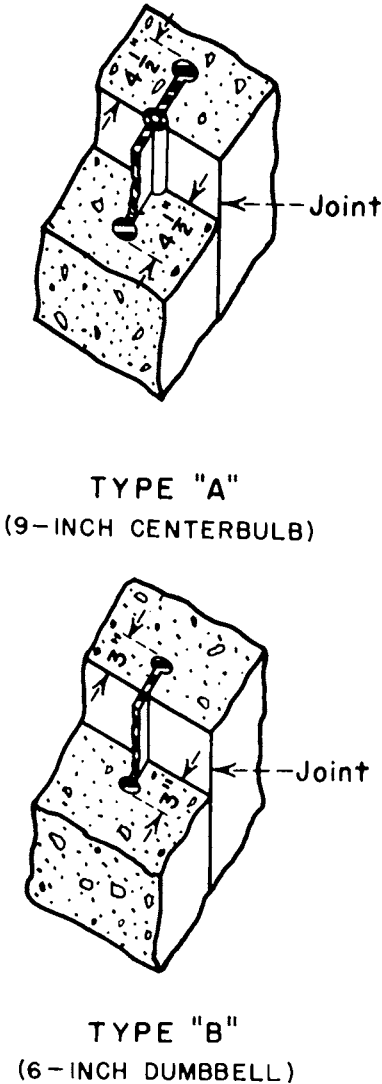


Fig 10—Waterstop for canal structures

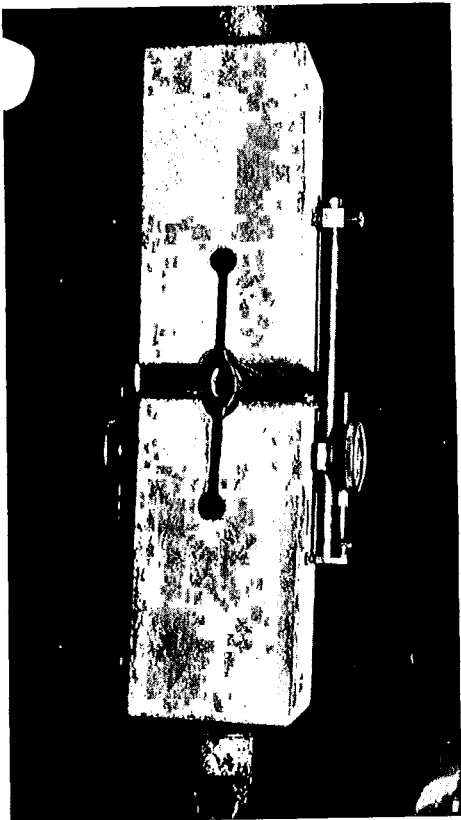


Fig. 11—Test of centerbulb type water-

rubber ranging in thickness from $\frac{1}{2}$ to 1 in. The sponge rubber filler is cut and placed flush with the joint periphery and secured to the concrete with copper nails.

Pipe joints

Due to the increased value of land and the necessity of maintaining freedom of movement for mobile farm equipment, irrigation distribution systems are being designed with fewer surface canals. This has resulted in a large increase in the use of concrete pipe for underground conveyance of water. The transition from surface laterals lined with concrete to subsurface concrete pipelines would not have been possible without the development of economical watertight joints.

Intensive development in the last decade has provided a series of satis-

factory joints for both high and low pressure pipelines. Fig. 15 shows joints now being used by the Bureau for both precast and monolithic concrete pipe. The Type-B mortar band joint was widely used in years past, but its use is now restricted to free flow or low pressure lines. Even when mortar joints are permitted by the specifications, some contractors have preferred to use a rubber gasket joint. In such cases the Bureau permits the use of serrated or tapered rubber gaskets conforming to ASTM Designation C-443. In Type R-1 and R-2 joints, a seal is effected between steel plates and rubber O-ring gaskets; whereas, in Type R-3 and R-4 joints a seal is achieved by an O-ring rubber gasket directly against concrete surfaces between the bell and spigot ends of the pipe. All of these rubber gasket joints

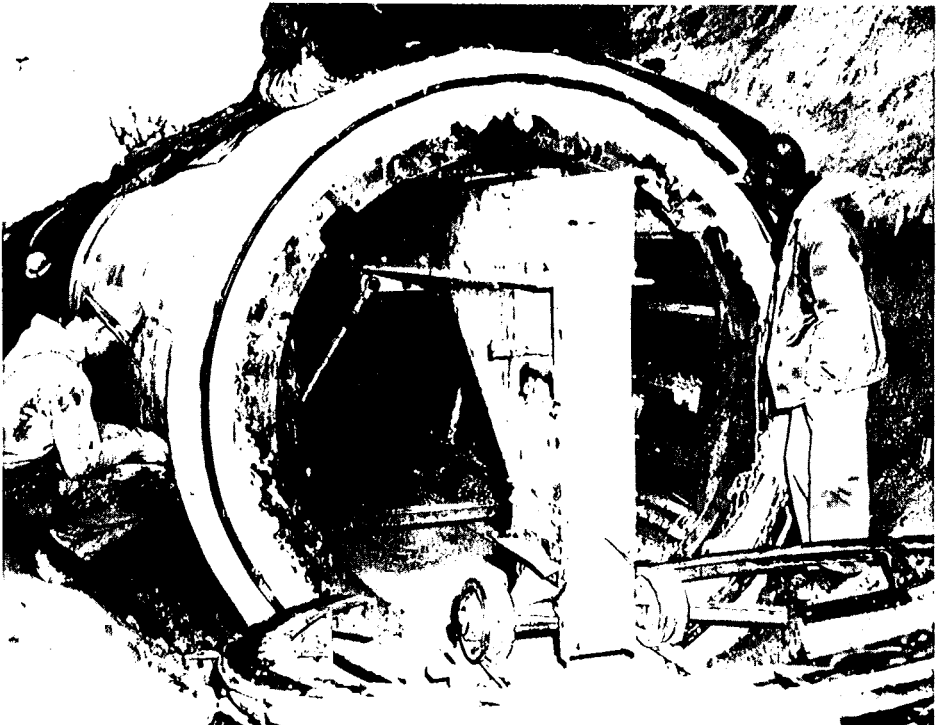


Fig. 12—Rubber waterstops used in siphon barrels



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may be used in pipelines under pressures up to 125 ft of head. From 125 to 600 ft of head, Type R-2 joints are used with concrete pipe reinforced with a steel cylinder. Pipe lengths and consequently the joint spacings for

pipelines vary from 6 to 24 ft, depending on pipe diameter and facilities for making and handling pipe. Rubber waterstops 9 in. wide, spaced about 25 ft apart, are used with monolithic pipe as shown in Fig. 15.

JOINTS IN CANAL AND RESERVOIR LININGS

Much of the water diverted and stored for irrigation is lost by seepage from unlined canals and reservoirs before it reaches the farmers' fields. Concrete linings are effective in preventing losses through previous soils provided the linings include impervious joint systems designed and constructed to prevent uncontrolled cracking.

Concrete slabs of a canal lining are subject to potentially serious tensile stresses induced by temperature and moisture changes, but compressive stresses accompanying these changes are usually of little concern for two reasons. First, concrete can usually resist compressive stresses induced by environmental temperature increases, provided irregularities of the subgrade

and lining do not induce buckling. Second, the expansion of concrete, even when completely saturated, is never as great as the contraction resulting from initial drying. Thus, expansion joints are ordinarily not required except where structures intersect the canal lining or where a rapid change in alignment occurs.

Contraction joints

Contraction cracks which result from tensile stresses produced by temperature and moisture decreases are of primary concern in design of concrete linings. Linings for irrigation purposes cannot economically be designed to overcome cracking, but some control of cracking can be accomplished by use of rein-

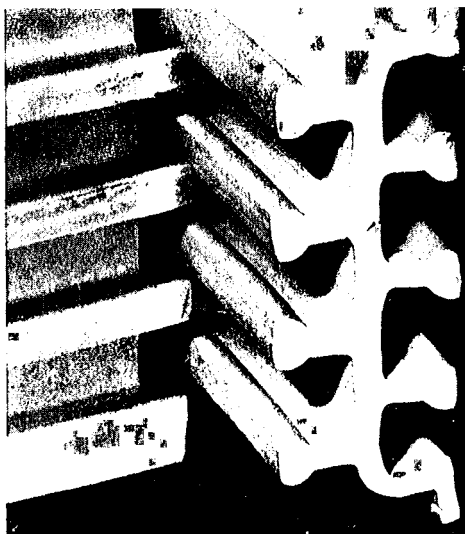


Fig. 13—Labyrinth shape polyvinylchloride waterstop

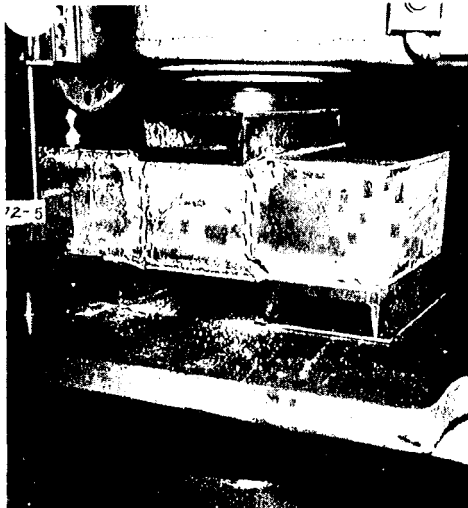


Fig. 14—Failure of the concrete surrounding the waterstop may occur under large joint deformations