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 34 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.

Lyse explained 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.
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 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 notbecome saturated might be just as satisfactory.

3. The concrete must be air-entrained.

4. The quality of the cement paste must be high, perhaps 4½ 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.
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.
CONCRETE CONSTRUCTION IN AQUEOUS ENVIRONMENTS

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 C₃A 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.

REFERENCES


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:

*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 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 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 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 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.

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**CONSTRUCTION TECHNIQUE**

**Spacing of pipes**

One tremie pipe usually serves to place about 300 sq ft of concrete surface. 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...
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
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 value 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 dry docks, 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 resealing 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 sidewise, 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
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 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 scaling 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.
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.
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; (2) 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, interruptions
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-
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

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*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.

References
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-
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 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
JOINTS AND CRACKS

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 prevent 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 7 1/2 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>
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<td>Powerplants - expansion and contraction joints</td>
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<td>Federal Specifications—Sealer, hot poured type, for joints in concrete pavements, bridges and other structures (SS-S-164, 2-1-52)</td>
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<td>Cold-applied asphalt joint sealer</td>
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<td>Polysulfide-base joint sealer</td>
<td>Powerplants-vertical expansion joints and contraction joints</td>
<td>Federal Specifications—Sealing Compound, Rubber Base, two components, for calking, sealing, and glazing in building construction (TT-S-00227, 7-17-61)</td>
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<td>Rubber joint strip</td>
<td>Powerplants - floor expansion joints exterior joints below grade</td>
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<td>Coal-tar dampproof coating</td>
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<td>Vinyl resin dampproof coating</td>
<td>Coating for corkboard filler</td>
<td>USBR Specifications for vinyl resin paint (VR-6, 7-1-60)</td>
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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
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 were 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 accommodate 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 titantium 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 regrouted 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 pre-load 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. 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 to normal
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

![Diagram of powerplant construction](image)

**Plan view at turbine level**

![Diagram of powerplant construction](image)

**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 reliquefy 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.

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![Diagram of joints in powerplants](image)

**Fig. 7—Typical joints in powerplants**
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 damproofing 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 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.

Continuing inelastic deformation of a waterstop under uniform loading is undesirable since the material extending across the joint 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. 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 sections 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...
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 satisfactory 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](image)
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-

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**Fig. 13**—Labyrinth shape polyvinylchloride waterstop

**Fig. 14**—Failure of the concrete surrounding the waterstop may occur under large joint deformations
forcing steel or by the forming of contraction joints at proper intervals. Bureau policy does not favor the use of reinforcing steel for this purpose. Instead, grooves are made in the fresh concrete to a depth of about one-third of the lining thickness. When the grooves are of proper depth and spacing, cracking will occur at these planes of weakness.*

Longitudinal as well as transverse grooves are advisable in canals having lined perimeters of 30 ft or more. Curbs along the tops of canal linings offer resistance to contraction and sometimes induce longitudinal cracks 6 to 8 ft down the slope from the curb. For this reason, Bureau specifications may require the first longitudinal groove to be located about 8 ft down the slope, in contrast to a usual spacing of about 15 ft between other longitudinal grooves.

Recommended spacing of transverse grooves in unreinforced concrete linings varies from 6 to 15 ft, depending on the size of the canal, thickness of the lining, and characteristics of the subgrade. Some Bureau linings have been constructed with grooves 25 ft apart. Intermediate cracks have occurred between many of these grooves, indicating that a lesser spacing is preferable. This is particularly true when the subgrade offers considerable restraint to movement of the lining. A cushion of graded gravel should be used on rock subgrades to permit contraction of the lining without cracking.

**Sealants**

Grooves in concrete canal linings should be made straight, maintained to the required shape and size, and protected from dirt or other foreign substances until they are filled with a mastic sealant. The mastic should not be applied while there is free water in the grooves. To avoid delay it may be applied before the concrete curing compound; however, the concrete should be sufficiently stiff to prevent damage by the mastic applicator.

Most sealants dry out and harden to some extent when exposed to air and sunlight. Therefore, if a canal is not to be filled with water until long after construction is completed, it may

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*A groove in a canal lining is not considered to be a joint until the intended crack occurs.*
be desirable to delay filling the grooves until just prior to initial use of the canal.

The sealant materials should be resilient and adhesive, capable of sealing joints effectively against the infiltration of moisture throughout repeated cycles of expansion and contraction. The sealants are usually a ready-mixed, homogeneous blend of asphalt, virgin synthetic rubber, inert filler, and suitable solvents. Their consistency, without additional solvents, should permit ready placement at 70°F by extrusion from a calking gun or by troweling. After placement, the material should not flow from the grooves. The extensibility of mastics and their bond to concrete are evaluated by the mechanical devices shown in Fig. 16-17.

The Bureau is engaged in a continuing program to evaluate new techniques and materials used to fill and seal joints in canal linings. One phase of this investigation includes evaluation of a variety of joint compounds used in a test section of the New York Canal near Boise, Idaho. After 5 years' exposure, some of the compounds are showing considerable differences in performance. The materials, by general type, are listed below in decreasing order of performance.

1. Ready-mixed, rubber-asphalt mastic
2. Asphalt-rubber emulsion
3. Polysulfide rubber
4. Two-component mastic filler
5. Ready-mix asphalt mastic
6. Neoprene rubber
7. Conventional oil-based calking compound

Performance was based not only on whether the various compounds were keeping the joints sealed, but also on their ability to maintain resilience.

![Fig. 16—Extension machine with three specimens under test](image-url)
The relatively poor performance of some of the synthetic rubber sealers may have been influenced by the fact that they were applied during weather conditions which were less than satisfactory for this type of work.

Asphalt-rubber emulsion is generally performing well both as a joint and crack sealant. However, these materials are quite fluid and application on slopes is difficult. Good performance on both slopes and flat surfaces has been obtained with ready-mixed rubberized asphalt and polysulfide rubber mastics. For extensive applications, polysulfides are too expensive and performance under prolonged submergence has not been proven.

Bureau representatives have recently had occasion to witness field trials made to create and seal joints in concrete linings, utilizing a cross-shaped plastic insert. One arm of the insert is a smooth tapered member positioned in the plane of the joint to weaken the concrete and induce a crack. A corrugated cross member serves as a waterstop and seals the induced crack. The potential benefits of this technique have aroused the keen interest of highway engineers, as well as those responsible for irrigation projects.

During field trials, the inserts for longitudinal joints were machine-fed into the fresh concrete as it was placed. Those for transverse joints were more difficult to install since it was necessary to place the lining and then work the insert into its proper position. In the author's judgment, the advantages inherent in this technique have not been fully realized. The principal difficulties have been rotation of the insert and control of its depth below the concrete surface. Cores taken from test sections showed that complete consolidation of concrete around the seals was not always obtained. Unless the insert is properly oriented and aligned, the desired contraction crack is not achieved. Also, if the concrete is not fully consolidated around the insert, leakage will occur. Work is reportedly being done to overcome these objectionable features and to lower the present relatively high costs. It appears that techniques for jointing and sealing canals in one operation offer considerable promise for future irrigation projects.

**REPAIR OF CRACKS**

Before proceeding to make a watertight repair of a crack, it is necessary to decide whether or not it is apt to reopen after repairs have been completed. For example, if the crack occurs due to differential settlement and the structure foundation is then strengthened with piles, a rigid repair may suffice. Also, cracks produced solely by drying shrinkage would not be likely to reopen again after filling. Thermal cracks sometimes occur be-
fore backfilling a structure, which would not reopen if repaired after backfilling. On the other hand, if continued settlement or thermal volume change is likely to reopen a crack, then a flexible seal should be included in the repair. In rare cases, it may be necessary to remove enough concrete from each side of a crack to permit the cut surfaces to be prepared as construction joints and a contraction joint with a suitable waterstop installed midway between.

Crack filling methods

In all repair work, it is essential that the flow of water through a crack be eliminated. This may be done by dewatering procedures, grouting to divert flow of water, or calking the crack beyond the repair area with oakum or other suitable materials. Several different methods are used in Bureau work to mend concrete linings or structures damaged by cracking.

Mastic filling—Cracks in canal and reservoir linings or low-head hydraulic structures are simply routed out, cleaned by sandblast and air-water jet, and filled with joint compound as illustrated in Fig. 18-21. Usually, mastic joint compounds are applied on the edge of the crack nearest to the source of pressure so that the water tends to force the mastic into the crack. However, in some structures where this procedure is not practical, a concrete or metal retaining cap may be used to confine mastic. A simple retainer may be made by positioning a metal strip across the crack and fastening it to expandable anchors or grouted bolts installed in the concrete along one side of the crack. To maintain hydraulic efficiency in some structures it may be necessary to cut the concrete surface adjacent to the crack and place the retaining cap flush with the original flow lines (Fig. 22).

Portland cement grouting—Cracks in blocks of dams, thick concrete walls, or rock foundations for hydraulic structures may sometimes be sealed by pumping portland cement grout into them. Grout nipples and vent pipes are installed in holes drilled into the crack and the periphery sealed by calking to prevent loss of grout. Thorough washing with water or other cleansing agents prior to grouting assists in obtaining a better filling of the crack. For Bureau work, a Type II cement
is usually employed, of which 98 percent must pass a No. 200 sieve and 99.995 percent must pass a No. 100 sieve. Grout mixtures may vary in volumetric proportion from 1 part cement and 5 parts water to 1 part cement and 1 part water, depending on the width of the crack.

Dry packing—Narrow cracks in hydraulic structures may be repaired by filling the opening with dense mortar, provided recurrent cracking is not expected and strength restoration is not a factor. To prepare the crack for repair, it should be undercut slightly to a minimum depth of 1 in. The prepared surface should be kept damp for several hours, then dusted lightly with cement and packed with mortar. Mortar consisting of 1 part cement and 2 parts sand passing the No. 16 screen has proven satisfactory on many jobs. Only enough water should be used to produce a mortar that will stick together upon being molded into a ball by slight pressure of the hands. The material is then packed into the crack in 3/8 in. thick layers.

If the water pressure exerted on the crack is small and if the structure is not expected to undergo additional volume change or settlement which would reopen the crack, no further repair work is necessary. However, if watertightness is essential and movement may produce further cracking, a protective membrane should be provided over the mortar filling. On some jobs a membrane 2 to 3 ft wide constructed of alternate layers of hot-applied asphalt sealing compound and asphalt-impregnated canvas or burlap has been used. Each layer is cut 6 in. wider than the previous layer, producing a tapered edge.

Epoxy fillings—Epoxy cements have unusually high strength in tension, compression, and bond; negligible shrinkage; and relative insensitivity to moisture change. For these reasons, it is often possible to restore a cracked concrete structural section to its original strength and watertightness by filling the crack with epoxy cement. However, it should not be relied on to provide the flexibility necessary to accommodate additional opening of the crack. Two compounds, the basic epoxy resin and a catalytic hardener, are usually mixed together just prior to using, since the pot life after mixing may be as short as 20 min. Usually the hardener is incorporated with a polysulfide compound to obtain a less brittle material.

Fine cracks may be filled by injecting the liquid mixture into the
crack with a calking gun or grout pump. To prevent loss of epoxy, and contain the pressure during pumping operations, a putty-like epoxy containing inert fillers has been used to calk the edges of the crack prior to injecting the liquid. The calking material must be stiff enough to stay in vertical cracks without sagging and on hardening develop sufficient bond strength to withstand internal pressures up to 125 psi. Recent repair techniques of this type have also been developed in the Bureau laboratories wherein the edge of a crack is routed out to a depth of about 3/8 in. and the resulting groove filled with epoxy mortar. Holes for grout nipples are drilled deeper than the groove and the nipples are anchored with epoxy mortar. After the mortar has gained sufficient strength, filling of the crack with activated epoxy liquid is done by pressure grouting procedures.

Wide cracks and voids resulting from surface spalls may be repaired by filling them with epoxy mortar or epoxy concrete. The areas to be re-
paired should be cleaned, dried, and primed with the activated epoxy liquid. Next, epoxy mortar or concrete is made by mixing clean, dry, well-graded aggregates with the epoxy liquid. Mixes consisting of 1 part of epoxy liquid plus 5 parts of fine sand usually produce high strength mortars. Mixes with 1 part of epoxy liquid plus 3 parts sand and 4 parts of fine gravel usually produce high strength concrete. Overly rich mixes are sticky, hard to work, and no stronger than mixes that are just lean enough to provide optimum workability. The epoxy mortar or concrete is packed into cracks and consolidated in shallow depressions by troweling. Firm, slow strokes of the trowel are used to force the matrix into the primer coating.

To prevent epoxy materials from sagging in sloping or vertical cracks, they must be supported by forms or by sealing tape placed across the joint immediately following troweling. To avoid difficulty due to hardening, the materials should be used within about 40 min after they are mixed together. In cold weather, the concrete surrounding the repair should be warmed to 50°F.

When extensive repairs are required and the relatively high cost of epoxy materials cannot be justified, a portland cement mortar or concrete filling can be successfully used provided the area to be repaired is first primed with activated epoxy liquid. Usually these repairs are of relatively small volume and there is a tendency of the old concrete to absorb moisture from the new. Continuous water curing for a minimum of 36 hr is highly desirable and protection from freezing is essential. Immediately after water curing, while the new surface is still damp, it should be coated with membrane curing compound.

**CONCLUSIONS**

Properly selected and installed, waterstops and sealants presently available are capable of providing watertight joints in concrete water-holding structures. Proper installation is expensive, but maintenance cost resulting from defective materials and workmanship cannot be justified.

New waterstop shapes which conserve material and are easy to install should be developed. A major reason limiting the use of rubber and plastic waterstops is the necessity of constructing blockouts in forms and supporting the waterstops in correct position during concreting operations.

Development of new plastic materials with higher strength and elastic properties than now available would favorably affect the cost and quality of joint construction.

Maintenance of joints in canal linings continues to be a source of major concern to irrigation districts. Fulfillment of the joint insert idea—to make and permanently seal contraction cracks—may in some regions make concrete linings practically maintenance-free. The design and construction of joint systems is just as important in assuring the integrity of concrete water-holding structures as are considerations of structural strength and durability. In both the laboratory and design office, development of better joint systems and materials must be emphasized if we are to fully utilize present structural knowledge. This is essential to meet the trend toward high, slender concrete structures designed to contain water without leakage.
REFERENCES


Discusses the design, construction, and maintenance of prestressed concrete tanks by East Bay Municipal Utility District, Calif., and describes improvements in design and construction procedures.

PRESTRESSED CONCRETE TANKS—
DESIGN, CONSTRUCTION, AND MAINTENANCE

By J. W. Trahern

The East Bay Municipal Utility District of Oakland, Calif., generally known as East Bay Water, is a publicly-owned water supply agency serving more than 1 million people in a 244-sq mile area on the east shore of San Francisco Bay. The rapidly expanding water consumption in the area is currently at an average rate of 160 million gal. per day.

East Bay Water imports water from its own source in the Sierra-Nevada Mountains 100 miles from the service area. The irregular topography of the service area and high demand at elevations higher than aqueduct entry into the area generated a $235 million complex of major water supply and distribution facilities.

Of the 124 filtered water reservoirs of 664 million gal. capacity distributed throughout East Bay Water's service area, 59 are prestressed concrete tanks with a total capacity of 140 million gal. These structures range in capacities from 0.2 to 12 million gal. Construction of these prestressed tanks has been spread over 27 years. The tanks have rendered excellent service. Maintenance costs have been low, practically insignificant. There has been little deterioration of the structures, and they have not required repairs of consequence.

All of East Bay Water's prestressed concrete tanks have been designed by its own staff. Some of the tanks have been built by its own forces, while others have been contracted to private firms.

Because concrete tanks are expected to be permanent structures, their tensioned steel elements are embedded
in concrete, for protection against corrosion, and are not accessible for adjustment of the stresses. Prestressed concrete tanks must be originally built so that subsequent changes will not necessitate adjustments of stress in the prestressing elements.

Vertical prestressing of the walls of concrete tanks is frequently used to prevent the development of cracks that might result from bending moments produced by horizontal prestressing and by liquid loading. Currently East Bay Water is providing vertical prestressing for the walls of all of its concrete tanks.

BAR-STRESSED TANKS

The design and construction of prestressed concrete tanks was first promoted in the United States by W. H. Hewett of Chicago in the early 1920’s. Tensioned circular steel bands were applied to the walls of concrete tanks in a quantity and distribution calculated to maintain circular compression in the walls under load conditions. The Hewett tank afforded some saving in both steel and concrete over the conventional reinforced concrete tank, and proved to be less susceptible to cracking of the walls and accompanying leakage.

The tensioned bands consisted of mild-steel bars threaded at the ends and coupled with turnbuckles. Tensioning was accomplished by running up the turnbuckles on the threaded bars. The tank walls were not prestressed vertically, but were equipped with a nominal amount of vertical reinforcement.

Hewett tanks were usually equipped with dome roofs supported by the tank walls. The design provided prestressing forces calculated to balance the horizontal thrust of the dome roof; to resist the hydrostatic pressure on the tank walls; and, at the same time, to retain a small amount of ring compressive stress in the walls at all elevations. Losses in stress resulting from creep in the steel and plastic flow of the concrete received no recognition. In today’s designs, such stress losses have been considered.

The first prestressed concrete tanks constructed by East Bay Water included mild-steel round bars and turnbuckles for the prestressing elements. No provision was made for loss of stress. These tanks, some over 27 years old, are giving excellent service. If ring tension has developed in the walls of the tanks as a result of stress losses, the quality of the structures has not been significantly impaired from the standpoint of the service they are rendering at present.

In the middle 1930’s, East Bay Water’s engineers economically modified the design by using higher strength square bars (85,000 psi ultimate strength) and providing coupling for the bars at special fabricated beams placed at intervals around the tanks. The advantage of the higher cost, high strength steel lay in the fact that the required prestressing force could be produced with less steel at a lower total cost.

The use of square instead of round bars, and of coupling beams instead of turnbuckles, resulted in a saving in the cost of installation because a great deal of effort was required to prevent the round bars from rolling.

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when the turnbuckles were tightened. In the square-bar and coupling-beam arrangement, the bars were run through properly-spaced holes in the beams. The beams were made continuous from the bottom to the top of the walls, and the bands were thereby securely supported in place. The bars were tensioned by running up nuts against the beams. The bearing of the square bars on the wall prevented them from turning when torque was applied to the nuts. More accurate prestressing with the square bars was possible because the torque applied at the nuts produced uniform tension.

East Bay Water has 20 tanks in service using the square prestressing elements. Vertical prestressing was not employed in these structures, but a nominal amount of vertical reinforcement was included in the tank walls. The tanks have consistently performed well.

**WIRE-STRESSED TANKS**

In the middle 1930's, the use of high-tensile-strength steel wire for prestressing concrete tanks was introduced. A machine capable of winding the wire helically on the tank walls under high tension at a fast rate was developed. Subsequently, a great number of wire-wound prestressed concrete tanks were constructed throughout the United States and abroad.

To maintain the walls of a tank in permanent compression, it is essential to include in the prestressing steel a substantial excess of stress over that required to meet design loads and to absorb subsequent stress losses. The surplus stress is almost constant and is correlated with the working stress of the concrete.

High strength steel rods having a minimum yield point of 70,000 psi can be safely stressed to about 50,000 psi. After deducting about 30,000 psi for excess stress, as is current practice for wire-stressed tanks, there remains for working purposes only about 20,000 psi. This means that, with high strength prestressing rods, only
about 40 percent of the safe working strength of the steel would be utilized to carry design loads.

In current practice, wire with an ultimate strength ranging from 200,000 to 250,000 psi is most used for prestressing tank walls, both vertically and horizontally. This wire may be safely stressed to seven-tenths of the ultimate strength, or 140,000 to 175,000 psi. Deducing 30,000 psi from these values for excess stress required, there remains 110,000 to 145,000 psi for working purposes. Consequently, with the high strength wire, approximately 80 percent of the safe working strength of the steel is utilized to carry design loads.

East Bay Water's first wire-wound tank was constructed in 1951. Since then it has had 35 more such tanks constructed. All are performing satisfactorily.

It appears that the wire-stressed tank should be somewhat superior to the bar-stressed type because positive provision is made for substantial losses of the prestress at small cost. However, experience with bar-stressed tanks has not revealed any serious defects in the structures due to stress losses.

The durability of the bar-stressed structures can probably be traced to three factors: (1) initial stresses are kept low—30,000 psi for steel bars of 85,000 psi ultimate strength and 500 psi for concrete of 3000 psi strength at 28 days; (2) normally, the tanks carry a water load resulting in small concrete stresses; and (3) creep of steel and plastic flow of concrete can be expected to be small, as shown by measurements of changes in stress. The stress changes in the bands of the bar-stressed tanks measured with an 18-in. extensometer are small enough to be within the range of accuracy of the instrument. One can, however, conceive of conditions under which stress losses in tanks could be large. Provision has been made in all East Bay Water's wire-stressed tanks for stress losses in the steel of the order of 15 percent. In these structures, measurements have indicated losses ranging up to 20,000 psi, and averaging about 10,000 psi.

**Other prestressing methods**

East Bay Water, in its plans and specifications for prestressed concrete tanks, has on occasion included alternative designs for horizontal prestressing arrangements. One of the designs is based on helical winding of high-tensile-strength steel wire, under tension, on the walls of the tanks to obtain the required prestressing load. After the wire has been installed, it is protected with shotcrete.

Another design is based on the use of tensioned wire cables to obtain the prestressing load. The cables, assembled in flexible metal sheaths, are cast within the tank walls, except at the ends, where they protrude at pilasters. They are tensioned by pulling the ends against the pilaster with hydraulic jacks. The ends are anchored at the pilasters, and the cable sheaths are filled with grout to protect the wires from corrosion.

**DESIGN AND CONSTRUCTION**

The principal structural elements of prestressed concrete tanks are foundation, floor, wall, and dome roof. In current practice, two of the elements, the wall and dome, are prestressed. The foundation and floor are generally constructed to more conventional standards. Although the prestressed wall and dome appear to attract the greater interest among engineers, substantial leakage from prestressed tanks is usually found to be the result of
inadequate floors and foundations. Leakage through the walls has seldom been severe.

Wall

Some of the principal elements requiring careful attention for proper design and successful construction of prestressed concrete tank walls are:

1. Dense concrete, without honeycomb or shrinkage cracks. A properly proportioned concrete mix, low water-cement ratio, careful placement, thorough vibration, and thorough curing of the concrete are essential.

2. Walls constructed of panels separated by vertical construction joints, each panel a continuous placement from bottom to top. The panels should be narrow enough to avoid the development of shrinkage cracks. Horizontal construction joints may result in differential shrinkage within the panels and locked-in stresses, rendering it impossible to achieve the anticipated distribution of prestress in the concrete.

3. High-tensile-strength steel for prestressing the walls. Cold-drawn wire, either stress relieved or not, is currently being used almost exclusively for this purpose.

4. Horizontal prestressing of the wall designed so that overstressing of the concrete will not occur. Some circumferential compression should be retained in the concrete after stress losses and with a full tank. In line with this consideration, an appropriate thickness of wall and a proper quantity and distribution of prestressing steel must be calculated for any tank, taking into account the elastic deformation of the structural elements. The arrangement of horizontal steel may be determined on the following basis:

(a) Original tensioning of sevenths of its ultimate strength.
(b) Stress losses of approximately 15 percent may take place in the steel wire in time. For lower strength steel, the loss may be greater.
(c) At any elevation on the tank
at each pilaster, the prestress in the concrete will be substantially that produced by the average stress in the cables.

5. Vertical prestressing of the walls with embedded cables in sheaths, similar to those used for horizontal prestressing, or with high strength steel rods (140,000 psi ultimate strength). The cables or rods are straight and are, therefore, not subject to serious friction forces and accompanying variation of stress. Sufficient vertical prestressing is provided in the tank walls to resist bending stresses produced by horizontal prestressing and water loads. As the horizontal prestressing proceeds, the walls slide radially inward on the footing. Friction between the footings and walls generates bending stresses in vertical wall elements. When the tanks are filled, the walls move radially outward, resulting in a reversal of the bending moment. The vertical steel is centered in the wall to provide for bending in two directions.

To minimize the friction force between the bottom of the tank wall and the footing, generated by radial movements of the wall, East Bay Water employs two sheets of graphite asbestos to separate the bottom of the wall from the top of the footing (Fig. 2-3). This serves to minimize the bending moments in the walls and the requirement for vertical prestressing. One patented system uses resilient rubber pads under the walls for this purpose and excellent results are reported.

6. Sufficient thickness of a tank wall at any elevation to support the initial tensioning of the prestressing steel without the development of excessive stresses in the concrete.

7. Radially oriented keys at the bottom of the walls, extending into the footings. Under this arrangement, wall panels may freely move radially on the footings but are prevented from
moving tangentially. Thus, horizontal loads on the tanks are supported by reactions at the shear keys directed tangentially to the circular walls at all locations.

Dome

The slab dome is a satisfactory roof for a circular tank. The material cost of such a roof is relatively low but the cost of forms is relatively high. Experience shows, however, that concrete dome roofs can be constructed on tanks at reasonable cost.

The stresses in a hemispherical slab dome are not significantly affected by the elastic property of the material of which the dome is constructed, and the character and magnitude of the stresses in such domes are easily computed. This is far from true of the normal segment dome. In the latter, the stresses caused by the elastic character of the abutment are many times as large as the stresses in the corresponding part of the hemispherical dome. A segmental dome may be equipped with an abutment providing the same reaction as furnished in a hemispherical dome to the corresponding segment by applying prestressing bands at the periphery.

The dome for the East Bay Water's Richmond Reservoir is 200 ft in diameter, rises 26 3/4 ft, is 4 1/2 in. thick over most of its area, and was constructed to a radius of 202 ft. Domes on smaller tanks are thinner. All domes have been constructed with a nominal amount of reinforcing steel for safety.

Currently, there appears to be a trend toward equipping prestressed concrete tanks with roofs composed of precast concrete elements. Perhaps some economic advantages will be realized from this approach.

Floor

As has been pointed out, most leakage from prestressed concrete tanks has been found to be through the floors. The essential features of the floor design currently used by East Bay Water are:

1. An underdrain system consisting of tile drains in graveled trenches (Fig. 3). The drain is extended to discharge at a convenient location where no damage will result and where the discharge of leakage water can be observed.

2. A reinforced concrete floor slab 6 in. thick supported on a prepared subgrade for the most part, but on the wall footing at the periphery. An arrangement of construction joints on approximately 12-ft centers is used. One system of these joints is located on radial lines, and a second system is located at right angles to the radial joints. Formed grooves at the construction joints are scaled with mortar at the bottom of the grooves and mastic in the top of the grooves. The mortar retains the mastic, preventing it from extruding through the joints by water pressure.

Foundation

Foundations for prestressed concrete tanks should preferably be nonyielding. All tank storage for East Bay Water is located in hilly areas. Bedrock is generally near the surface in those areas. It has been necessary, however, to avoid locations on the extensive fault zones in the area.

Sealing ring

Sealing of the joints between the floor slabs and walls of prestressed tanks has frequently presented problems. The general practice in the past was to use cement and sand mortar with an admixture of iron dust for the seal. This method was not always satisfactory. A rubber seal ring has done an excellent job. In this case, the walls and the
peripheries of the floor slabs rest on common circular concrete footing rings. An annular space, a few inches wide and 9 to 12 in. deep, between the edge of the floor slab and the wall, results in each structure. Grout is placed in the bottom portion of the space and an assembly consisting of a strip of rubber with steel half-rounds or ovals on the top and bottom is inserted.

**SAMPLE SPECIFICATIONS**

**Prestressing wire**

Class I wire

1. The prestressing wire shall be uncoated, hard drawn, stress relieved, and 0.156 in. nominal diameter.
2. The prestressing wire shall have a minimum ultimate strength of not less than 250,000 psi.
3. The stress at 0.007 in. per in. elongation shall not be less than 175,000 psi.
4. The elongation under load at fracture shall not be less than 0.04 in. per in. on a 10-in. gage length.

Class II wire

1. The prestressing wire shall be uncoated, hard drawn, and 0.196 in. nominal diameter.
2. The prestressing wire shall have a minimum ultimate strength of not less than 240,000 psi.
3. The stress at 0.007 in. per in. elongation shall not be less than 160,000 psi.
4. The elongation under load at fracture shall not be less than 0.04 in. per in. on a 10-in. gage length.

Class III wire

1. The prestressing wire shall be uncoated, hard drawn, stress relieved, and 0.250 in. nominal diameter.
2. The prestressing wire shall have a minimum ultimate strength of not less than 200,000 psi.
3. The stress at 0.007 in. per in. elongation shall not be less than 150,000 psi.
4. The elongation under load at fracture shall not be less than 0.025 in. per in. on a 10-in. gage length.

Class IV wire

1. The prestressing wire shall be uncoated, hard drawn, and 0.160 to 0.196 in. diameter.
2. The prestressing wire shall have a minimum ultimate strength of not less than 200,000 psi.
3. The stress at 0.007 in. per in. elongation shall not be less than 140,000 psi.
4. The elongation under load at fracture shall not be less than 0.025 in. per in. on a 10-in. gage length.

**Prestressing bars**

The physical properties of the prestressing bar steel, determined by static tensile tests, shall conform to the following:

- Minimum ultimate stress .............. 145,000 psi
- Minimum stress at 0.7 percent elongation .................. 130,000 psi
- Minimum stress at 0.3 percent elongation .................. 75,000 psi
- Elongation in 10 in.
  - For 145,000 psi ultimate stress...6½ percent, min.
  - For 130,000 psi ultimate stress...5½ percent, min.
- Modulus of elasticity...25 $\times 10^6$ to 27 $\times 10^6$ psi

**Horizontal prestressing**

1. Horizontal prestressing tendons may be Class I, II, III, or IV wire.
2. Class IV wire may be prestressed by pulling it through a die slightly smaller in dimension than the diameter of the wire. This does not apply to Class I, Class II, or Class III wire.
3. During the prestressing operations the temporary stress in any part of a prestressing tendon shall not exceed 80 percent of its minimum ultimate strength. Prestressing tendons at final placement shall have an initial stress not more than 70 percent nor less than 60 percent of their minimum ultimate strength.
4. The minimum center-line spacing in any layer of wires shall be 3 times the wire diameter after stressing.

**Vertical prestressing**

1. Vertical prestressing tendons may be high tensile strength steel bars or high tensile strength steel or high tensile strength steel wire. (See specifications, this sheet.)
2. During the prestressing operations the temporary stress in any part of a prestressing tendon shall not exceed 80 percent of its minimum ultimate strength. Prestressing tendons at final placement shall have an initial stress not more than 70 percent nor less than 60 percent of their minimum ultimate strength.
3. The tensile stress to which the tendons shall be pulled at the jacking ends to give the specified initial prestressing force shall take into account all friction at the anchorages, between the tendons and the sleeves, and at the jacks.
4. The vertical tendons shall be prestressed before wire winding the walls.
5. All prestressed tendons shall be bonded in the wall by cement grouting.
6. Anchor units at top and bottom of wall shall be of an approved design and of a type that has been generally accepted for construction projects.

**Concrete—reservoir wall**

1. Concrete, at time of prestressing, shall have a minimum of 5000 psi compressive strength.
2. Concrete shall have not less than 4500 psi compressive strength in 28 days.
3. Alternate panels shall be placed consecutively. Each panel shall be one continuous placement for full height of wall.

**General**

1. In the event the dome is placed before the walls are horizontally prestressed, the temporary erection rods shall be installed in conformance to the details shown on the plans and in advance of placing the dome.
2. In the event the dome is placed after the walls are horizontally prestressed, the installation of temporary erection rods will not be required.
3. A tensioning schedule and drawings showing the type, amount, and arrangement of prestressing tendons proposed shall be submitted for approval of the engineer.
4. Methods and details for anchoring the prestressing tendons (not shown) are subject to the approval of the engineer.
above the grout. The ovals and rubber are drilled for studs at short intervals. By drawing the steel pieces together with the studs, the rubber strip is spread to bear against the side boundaries of the annular space with sufficient pressure to prevent leakage at the joint. The point will remain tight under some movement of the wall.

At present, a joint seal far less expensive than the rubber one just described is being successfully used. This seal consists of a layer of grout with an asphalt and latex mixture overlay in the annular space between the floor and wall. Space is left in the top portion of the joint to install a rubber seal if later found necessary.

**CORROSION**

Corrosion of the prestressing wire of some prestressed concrete tanks has been reported. One tank in New York and one in Redwood City, Calif. collapsed. Two tanks in Menlo Park, Calif., were repaired when they appeared to be on the verge of collapsing as a result of extensive corrosion of the wire. Some corrosion of the wire of five tanks in Sacramento has been discovered. In this latter case, the extent of the corrosion is not so serious as to suggest immediate collapse of the tanks, but nevertheless, the installation of additional prestressing steel on the tanks, to insure against future collapse, was required. Of these nine tanks, eight have been used to store sludge and one was used to store cement slurry containing salt water.

The 12 million gal. Richmond Reservoir was constructed in 1954-55 by a private contractor. This tank has an inside diameter of 204 ft and the wall is 44 ft high. The prestressing was done with wire cables assembled in a flexible metal hose embedded in the walls. After tensioning the cables, grout was pumped into the metal hose so as to surround the cables and protect them from corrosion. For horizontal prestressing, there are 360 cables of 18 wires each, 166 ft long, a total of 204 miles of wire. For vertical prestressing, there are 320 cables of 18 wires each, 44 ft long, a total of 48 miles of wire.

The prestressing loads for this tank were controlled by measurements of the elastic elongation of the cables produced by tensioning. Tensioning of the cables was done in two stages: (1) something less than the required elongation was produced, and (2) the required elongation was accomplished after some creep of the steel and plastic flow of the concrete had occurred. With this two-stage process, the required elongation of the cables could be obtained with the production of a lower initial stress than would be required in single stage prestressing were used.

During the second stage of prestressing work at Richmond Reservoir, it was discovered that some of the wire had parted. It was necessary to remove 204 miles of wire from the metal hose that was embedded in the walls of the tank and to replace it with new wire. The reservoir has now been in operation for over 6 years with no signs of distress being observed.

The original wire placed in the walls of the reservoir was under stress and exposed to air and moisture for about 3 months between the time of the first stage and that of the second-stage prestressing operation when the ruptured wires were discovered.

On another occasion the author observed failure of cold-drawn wire after a rather short exposure to at-
mosphere and tensile stress. In this case, at small prepared openings in the shotcrete protective coating, the wire was left exposed to provide for periodic measurements of the tension in the wire. The exposed wires began to rupture within a short time. They were subsequently covered with cement mortar for protection.

Reports of corrosion problems with some prestressed tanks have prompted East Bay Water to examine its own tanks. In 1956, extensive effort was expended in a survey of a large number of the tanks. A large amount of prestressing wire was exposed and observed on this occasion. Nothing alarming was found. Again in 1961 a similar survey of the tanks was made. Some superficial corrosion of the prestressing steel was found on a couple of tanks, nothing that appeared to be serious at this point. Metal loss observed did not appear to be significant.

Recently, East Bay Water has been experimenting with copper sulfate half-cell potential readings on the outside surface of the wire-wrapped tanks. By statistical analysis of the data, an indication of corrosive action taking place on the embedded wire is obtained. It has been found that as corrosive products are formed and other conditions change, the indicated anodes migrate. For this reason the surface potential method does not appear to be a practical technique for locating the most corroded spots on a tank.

Cathodic protection for the prestressing wire of one tank is now being installed. Preliminary investigations have indicated that a high-voltage low-current system will be required. A 200-v, 15-amp rectifier will be used on this experimental installation. It is expected that the effectiveness of cathodic protection will be determined when this installation is put into operation and tested.

Following are some interesting points regarding corrosion of steel reinforcement in concrete structures that have been advanced in several articles on the subject:

1. The presence of three elements simultaneously are necessary to support corrosion of steel: oxygen, moisture, and ions. In a great many cases of corrosion, chlorides have provided the ions. The use of calcium chloride in reinforced concrete structures should be avoided.

2. The rate of corrosion of reinforcement decreases with an increase of cement content; decreases with an increase of concrete cover over the steel; increases with an increase of water in the mix; and increases with an increase of the concentration of chlorides present.

3. Some agencies, including East Bay Water, have adopted the practice of specifying a protective coating over the prestressing steel of tanks and of pipe consisting of a flush coat of neat cement slurry followed immediately by a coat of cement and sand mortar. It is expected that inclusion of the slurry in the protective coating produces improved protection for the reinforcement by providing additional hydroxides and by minimizing voids in the coating in the vicinity of the steel.

A great many prestressed concrete tanks are in service in the United States and abroad. Only a few scattered cases of serious corrosion of the prestressing steel have been reported. These reported cases of corrosion have not been substantially correlated with any of the elements of environments to which the steel has been subjected. At this point, total field experience with prestressed concrete tanks indicates that there is some risk of corrosion failure connected with the decision to use these structures for storing liquids.
East Bay Water's tanks have been designed by people experienced in the design of such structures. Construction of the tanks has been subject to rigid inspection by experienced personnel. The tanks have been constructed by contractors experienced in the techniques involved. Hopefully, conformance with this procedure will result in a continued record of good experience. East Bay Water's 27-year record is satisfactory.
The behavior of prestressing wire in both sewage and water storage tanks is described. The conditions which led to the failure of a sludge digestion tank and the opinions of the cause of failure are discussed, and methods of avoiding problems suggested.

PRESTRESSED CONCRETE TANK PERFORMANCE

By Morris Schupack

The construction of prestressed concrete circular tanks developed primarily through the interest of a single company with a proprietary method of prestressing circular structures. In the United States, most tanks have been constructed by the wire winding method. The first prestressed concrete tanks were an attempt to prestress reinforced concrete tanks with rods and turnbuckles. It became apparent, generally, with the problem of placing stressing rods on a wall and the rather inefficient use of steel, that another method of prestressing tanks were necessary.

The wire wrapping method, developed by Morris Crom, was introduced in 1941. This method draws high strength carbon steel wire through a die and the energy required to reduce the wire prestresses the wall.

It should also be noted that the method of wrapping concrete pressure pipes with high strength wire has been in use for over 20 years, however, Freyssinet constructed the first prestressed pipe in 1936. Prestressing of pipe by wire wrapping is an excellent use of high strength wire and has created a rather large pipe industry throughout the world.

Other methods of prestressing tanks have been used with varying degrees of success. Particularly in Europe, individual prestressing tendons stressed against buttresses has been a competitive means of performing circular prestressing. The use of individual tendons stressed against occasional buttresses has various mechanical problems which have been somewhat more successfully solved in Europe than in the United States. At present, the individual tendon system is more competitive in Europe, than in the United States.
TANK DESIGN

The design history of prestressed concrete tanks has been a matter of using basic shell principles considering the various phases of construction and loading. In general, the construction techniques have evolved from a trial and error basis, as have many other techniques in the construction industry.

The problem of designing and constructing prestressed concrete tanks were generally simple, as long as small tank diameters were involved. However, many items which were ignored in smaller tanks soon became sources of major leakage problems in larger tanks. It became apparent that careful analysis of all factors that may affect the structure had to be considered in the design in order to construct a tank which would perform satisfactorily. In early prestressed concrete tanks (small tanks of 500,000 to 1,000,000 gal.), fixed bases and fixed dome roofs were generally used, and shotcrete (pneumatic mortar) construction preferred.

A fixed tank wall base or a fixed dome has the tank sidewalls monolithic with the foundation and dome ring. This offered radial resistant against inward and outward movements of the wall and the dome. These movements are caused by circumferential prestress, prestresses losses, shrinkage and shrinkage recovery, thermal effects, and water pressure.

Reinforcement

It was thought somewhat erroneously that normal reinforcing steel for the core walls and domes caused excessive shrinkage stresses and should be minimized or avoided. Some well-known trade publications still indicate that this is a proper approach. It was found that by minimizing the vertical reinforcement and by cutting off dowels from the dome or the fixed footing, horizontal cracks generally occurred at the vicinity of the ends of these cut off bars as shown in Fig. 1. These cracks seemed to occur with time even with substantial vertical prestressing.

Since the small tanks performed rather well, if they were constructed properly, it was thought that there was no reason not to step out into the larger tanks. Larger tanks generally made shotcrete uneconomical, and therefore concrete core walls were considered. Problems, such as cold joints, construction joints, and honeycombing, occurred in the initial cast-in-place concrete construction which were not as easily controlled as a good shotcrete application. Today, cast-in-place concrete construction is most used.

Wall bending stress

The first prestressed concrete tanks were generally designed with fixed bases and domes. However, as soon as the tanks became larger, it was found that if fixed conditions were used, high vertical bending stresses occurred in the walls that could not be economically or satisfactorily reinforced. In order to minimize the vertical bending stresses, the wall base was allowed to slide during the prestressing procedure.

Sliding bases were of many types. Concrete footings were treated to permit the radial motion of the wall as the prestressing forces were introduced. However, it was frequently found that friction relieving devices were not dependable. Horizontal cracking of the
walls occurred during or after prestress, unless a conservative design assumption was made for determination of vertical reinforcement. The early procedure was to release the tank base only for the prestressing operation. After the prestressing operation, the tank base was generally hinged. Based on experience with the smaller tanks, it was thought that the same general design assumptions could be used for larger tanks.

It was found that unless the walls were heavily reinforced vertically, and rather thick, cracking of the nature shown in Fig. 1 often occurred. This cracking has occurred as much as 3 to 5 years after construction. Apparently the volumetric changes caused by temperature, creep, shrinkage and shrinkage recovery, differential temperature, and shrinkage effects, caused moments which were beyond the capacity of the nominal vertical reinforcement and vertical prestressing used. It was found that a predictable base condition could be established by utilizing elastomeric pads which had a predictable radial restraint to the wall and/or dome motion. This resulted in a design criteria compatible with the actual performance of the tank.

The actual design of a prestressed tank would be rather simple if one only had to be concerned with the ring stresses. However, the vertical moments which cause horizontal stresses, and eventually cracking, must be investigated. This takes a rather careful analysis utilizing all the known characteristics of concrete under various temperature differentials, and other volumetric change conditions. The application of these methods is beyond the scope of this paper.

**Concrete**

Whether regular concrete or shotcrete is used in constructing a prestressed concrete tank, careful planning must be assured in the construction phases to avoid cold joints. It has been found time and again that where cold joints exist in concrete tanks, they will probably leak. A dense concrete which is homogeneous is more important than high concrete strength. Using a dry mix which would result in honeycombing and possible cold joints is not a good procedure for prestressed concrete tank walls. It is necessary to insure as much workability as possible to avoid cold joints and honeycombing.

**PRESTRESSING STEEL FOR WATER TANKS**

The item which seems to cause the greatest amount of concern, and has had the greatest publicity in recent years, is the problem of protecting the prestressing steel. There have been various incidents of corrosion problems, both in prestressed concrete tanks and in pipe.*

For wire wrapped tanks, the meth-

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od of protecting the wire has generally been the use of shotcrete. It is necessary to completely encase prestressing wire with a rich dense mortar which is bonded to the previous layers in order to insure corrosion protection. If the wire is not completely protected it is likely that corrosion will occur.

Corrosion of properly protected wires will generally not occur, even if a hairline crack exists through the shotcrete, and water from the inside of the tank leaks through (Fig. 2). Proper protection of the wire seems to require a minimum dense mortar cover of $\frac{1}{2}$ in.

On one particular water tank inspected recently, it was found that where the shotcrete had spalled off, the wires were corroded on the outside face. By chipping into adjacent shotcrete it was found that wherever the shotcrete was less than $\frac{1}{2}$ in. thick, corrosion existed. Where the shotcrete was more than $\frac{1}{2}$ in. thick, the wires were bright and clean (Fig. 3). This particular tank was a slipform tank which had suffered every catastrophic in the construction phases that can happen during a slipforming process. The tank is full of cold joints, and has been leaking for over 11 years. In areas where water seeps through the wall almost constantly, openings were made to expose the wire. It was found that where shotcrete was $\frac{1}{2}$ in. thick or greater, and bonded to the underlayers, the wires were bright and clean. This finding was similar to that found on other tanks examined.

In the various tanks examined by the author, no sign of stress corrosion in water tanks has been found. This, however, does not preclude the possibility of stress corrosion occurring if wires which are improperly protected.
were subject to an environment conducive to stress corrosion. This could be an environment of a corrosive atmosphere, direct contact of the wires with detrimental chemicals, or a condition which may cause electrolysis.

The only means of protection, in a corrosive environment, is to assure that a substantial complete protective coating exists, and that detrimental cracking of the tank walls is avoided by a knowledgeable design approach.

**PRESTRESSING STEEL FOR SEWAGE TANKS**

The problems which have occurred in sewage treatment tanks have been disconcerting to owners and engineers. The failure of a 2 million gal. sludge tank was one which certainly received much publicity. (It is difficult to see how 2 million gal. of sludge not properly contained could escape publicity.)

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**Fig. 4—Roof and wall details of failed sludge tank**
The conditions that existed at this particular structure resulted from a series of unfortunate circumstances. A basic problem which occurred seemed to be an over-all corrosion of the prestressing wire which manifested itself in the forms of ordinary corrosion and stress corrosion. In the one tank that failed out of eight digestors, the corrosion was extremely widespread. It appears from personal limited examination of the other tanks that their prestressing steel is also corroded to varying degrees.

A series of details, which on the surface seemed unimportant, led to this major corrosion problem. As found repeatedly, the points which make problems in structures, are details and not over-all strength. The unbonded construction joint of the roof parapet detail shown in Fig. 4 permitted water to pass through the parapet joint and then find its way between the brick and into the galvanized dovetail. Fig. 5 shows how the parapet has moved on its support pushing the brick and how the water found its way between the previously bonded brick coarses. Most brick in areas of this type of distress were highly deteriorated probably from freeze-thaw action. Note also on Fig. 5, the vertical dovetails which were generally highly corroded.

The prestressing steel wire was placed almost directly against the galvanized dovetails (Fig. 6). Present day good practice would require a minimum of \( \frac{3}{8} \) in. mortar between dovetail and prestressing wire.

The extent of corrosion on this tank was such that the galvanized dovetails were completely disintegrated in many cases (Fig. 7). Since these galvanized dovetails were not filled with mortar (which is usual practice), they formed...
vertical channels for the roof drainage. As a result, the dovetails eventually corroded even though they were galvanized. The drainage water then found its way to the prestressing steel. In some areas the prestressing wires were not spaced sufficiently to insure complete filling of the spaces between the wires. Consequently the water apparently had horizontal passages between dovetail channels. The alternate wetting and drying which occurred on these wires was extremely conducive to corrosive conditions. This is unlike the leakage in the water tank where there is not as much alternate wetting and drying and change of air. It can be seen in Fig. 8 how the corrosion in the vicinity of the dovetails was quite marked whereas the corrosion away from the dovetails diminished.

The failure may not have occurred if the drainage hadn’t been subjected to contamination by sewage gas. It is likely that the sewage gas, which was under a pressure of about 13 in. of water, escaped through the concrete roof and mixed with the rain water. The detail of the roof did not include waterproofing (Fig. 4). It was insulated with foam glass and covered with a mastic. This foam glass with a mastic coating did not afford a water or vapor barrier. Therefore, the water probably mixed with the sewage gas and formed a severe corrosive medium.
It is well known that steel under stress in the presence of hydrogen sulfide will suffer stress corrosion. Also, $\text{H}_2\text{S}$ mixed with water will give either $\text{H}_2\text{SO}_3$ or $\text{H}_2\text{SO}_4$, which are extremely corrosive to steel. These unfortunate circumstances grouped together resulted in a condition of over-all corrosion, and the consequent failure shown in Fig. 9.

It has been stated that the failure was caused by a basic characteristic of the as-drawn wire: susceptibility to stress corrosion. The point remains, however, that there are many miles of prestressed pipe, and several thousand prestressed concrete tanks which have not shown this stress corrosion phenomenon.

It is necessary to point out that in a good number of tanks which have been conceived on an inadequate design philosophy have leaked for over 10 years. In these water tanks, generalized corrosion does not exist if proper shotcrete protection has been applied. Some wire corrosion exists, but this generally can be traced to electrolytic corrosion because of aluminum torpedo splices or the use of other dissimilar metals, or where pneumatic mortar cover coat was improperly placed or not bonded. Stress corrosion, as found in the failed sewage tank, has not existed, under these conditions. This again does not imply that stress corrosion cannot occur, but it does indicate that it takes a particular environment to cause stress corrosion. It is felt by the author that a condition of protection against corrosion can be created with the proper construction techniques and materials.

It is also worth mentioning that in another sewage digester which suffered a failure, but not of a catastrophic type, the corrosion of the wires were generally limited to those areas where the sewage had access to the wire. Fig. 10 shows a crack pattern in a small digester that manifested this type of failure after about 11 years of use. Note that the failure lines follow approximately the yield lines of a plate.
ported on four sides. The first indication of leakage this tank indicated by the vertical cracks. But during inspection, the yield line cracks of the core were traced.

An initial crack in this tank was used by the type of vertical prestressing unit used. The vertical unit placed on the inside face of the wall and quite often it cracked the wall from top to bottom. Generally for circumferential prestressing, these cracks were closed up so that they could be found. Apparently, the failure crack did not close up completely and sewage found its way into the crack and apparently, with time, corroded wire in the immediate vicinity of a vertical crack.

It was most interesting to note that when one removed the mortar on one side of the crack, the wires were sight. In this particular instance, the color of the corrosion was black. This apparently indicated that a ferric sulfide was produced. The corewall crack itself also indicated that it must have been acted on by some rather strong acid. The cracked area indicated deterioration of the surface of the concrete. This was not so with the inside of the tank wall.

Fig. 10—Sewage tank failure

CAUSES OF CIRCUMFERENTIAL PRESTRESSING STEEL PROBLEMS

As a guide to the designer and developer and as a construction check list, the following is a listing of the probable potential causes of prestressing steel corrosion or reduced ultimate strength in circularly prestressed structures:

1) Poor design and construction of hole or pipe openings
2) Voids in mortar between adjacent wires due to poor wire spacing
3) Bunching of wires sprung around windings resulting in poor mortar coverage of wire
4) Poor bond of cover coat to corel or previously placed shotcrete
5) Inadequate thickness of mortar cover (minimum specified cover should be 3/4 in.)
6) Improper mix of shotcrete
7) Stressing methods which cause nicks and notches in wire
8) Splices of dissimilar material
9) Additives such as calcium chloride in corewall concrete or shotcrete
10) Corrosive environment caused by leakage through cracks in the corewall
11) Excessively delayed cover coating
12) Inadequate access facilities for shotcrete application
Careless welding in vicinity of wire  
Using wire as a ground for welding  
Permanent overstressing of wire  
Inadequate winter protection, high winds during shooting, and other improper procedures for shotcrete  
Inserts of dissimilar metal in contact with prestressing steel  
Electrical contact of prestressing steel with pipes or inserts that may carry currents  
Horizontal cracks or control joints which move and inadvertently dislodge the mortar protection  
Localized restraint of walls to stress and volumetric changes by improperly attached walls, pipes, or other appurtenances  
No curing or improper curing of cover coat

PROTECTION OF PRESTRESSING STEEL

There is apparently a much greater need for a fool proof protection of the prestressing steel on sewage tanks than on water tanks. It is not felt that galvanized wire under a detrimental environment would necessarily give an appreciable extended life to the prestressing steel.

It is the opinion of the author that prestressing steel for sewage tanks should be placed only under the most careful supervision. The wires should be accurately spaced and the protective covering should be greater than that required for water tanks. It is also necessary, evidently, to insure that the wall design takes into account all the environmental problems which may effect the cracking performance of the wall.

CONCLUSIONS

In the various tanks inspected, and from the particular experience available in this field, it is evident that prestressed concrete tanks can be designed to perform satisfactorily under many conditions and environments. It is necessary to fully consider all design problems that may not be necessary for the more usual structure. Since there are few other types of structures which are subjected to full load for most of their life, and since any performance characteristic which will create cracks produce an immediately tell-tale failure, it is necessary to fully understand the performance of these structures and to provide details which will assure the assumed structural conditions.

The protection of the wire is most important and it is necessary to assure that complete and sufficient cover be used on the wire. If special requirements such as manhole openings or pipe openings may cause bunching of wires, the details should be conceived so bunching will be avoided. It is also necessary to prevent stray electrical currents from being carried through the tanks into the prestressing wire.

Another item in design which must be considered during the initial stages and then in the operational stages is attachment. No attachments should be made to the tank wall which will change its basic structural action. If the tank is not designed for these attachments, cracking may occur which will be detrimental to the performance of the tank.

In general, it is necessary that prestressed concrete tanks be treated similarly to other structures, with periodic inspections to insure continued
proper performance. Because of the belief that concrete is maintenance free, large apparent distress in tanks have been ignored, and consequently, with time, rather major corrective measures were necessary. If these problems were detected at an earlier date, minor provisions could have been made to avoid major problems.

Observations reported here are based on personal field inspection by the author prior to September 1962. Other reported incidents in prestressed concrete tanks have not been described.
Prestressed concrete cylinder piles are especially suitable for substructures of viaducts, grade separation structures, and bridges because they function simultaneously as foundation piles and structural members. Their design, fabrication, and installation is described.

PRESTRESSED CONCRETE CYLINDER PILES

By W. T. Robertson

The successful adaptation of prestressed concrete cylinder piles as piers for viaduct, grade separation, and bridge structures has stimulated interest in research and development in the design, fabrication, and erection of these structural members. Among the major projects in which large diameter piles have been utilized are: the Maracaibo Bridge, northwest Venezuela; Lake Pontchartrain Bridge, New Orleans; off-shore oil structures, Gulf of Mexico; Illinois Toll Highway grade separation structures; the Puyallup River Bridge (Fig. 1), Tacoma; and the Portage Bay Bridge, Arboretum Interchange, Second Lake Washington Bridge approaches, and Galer-Lakeview viaduct, Seattle.

During the past few years the state of Washington has constructed a total of seven structures using prestressed concrete cylinder piles. The first of these involved the driving of 36-in. piles with 5-in. shells in the east approach to the Eleventh Street Bridge in Tacoma. These piles were driven to a design load-bearing capacity of 106 tons. The Ravenna Boulevard viaduct in Seattle used 48-in. piles of the same type driven to a design load capacity of 260 tons. These two structures used post-tensioned strands.

The successful application of cylinder piles to the Eleventh Street and Ravenna Boulevard structures encouraged the Washington Department of Highways to explore the possibility of incorporating them in the design of statically indeterminate structures in which the piles not only serve as pier shafts but become integral parts of the structural frame, thereby providing increased rigidity.
Prestressed concrete cylinder piles are economical and cause a minimum of interference with vehicular or boat traffic during construction by the elimination of footings, seals, and cofferdams. While they do not provide a panacea for all foundation problems, they have opened a new field of economical bridge construction where suitable foundation conditions exist. When properly designed, they also present an aesthetically pleasing structure for either viaduct or bridge construction.

**DESIGN**

With few exceptions, design procedures for pile-supported structures follow those of cast-in-place construction. However, driving stresses and the lateral load resistance of the foundation material assume added importance in this type of construction; therefore, thorough investigation of dynamic effects of the pile hammer and of the soil resistance to shear and deformation are essential.

In general, design specifications for prestressed piles follow the recommendations of *AASHO Standard Specifications for Highway Bridges* for prestressed concrete members.

Currently, all cylinder pile designs are based on 6 x 12 in. cylinder strengths of 6000 psi and 270,000 psi ultimate strength 7/8 in. 7-wire strands. Concrete is required to have a strength of 5000 psi at the time of release.

**Seismic effects**

Seismic effects frequently control the design of the pile section. The recommendations of the Joint Committee of the San Francisco Section, ASCE, and the Structural Engineers Association of Northern California have been followed. The coefficient $C$, defined as the ratio of the horizontal seismic effect to the vertical load applied at the upper end of the pile, may be readily computed by the equation:

$$ C = \frac{0.025}{T'} $$

where $T'$ = natural period of vibration of the structure in seconds or $1.11 \sqrt{\delta}$; and $\delta$ = statical deflection, in feet, due to a horizontal force equal in magnitude to the vertical load on the upper end of the pile. The assumed vertical load consists of the dead load from the superstructure and pile cap, one-fourth of the weight of the pile above ground line, and one-half of the design live load without impact. Bridges in the state of Washington are designed for values of $C$ between an upper limit of 0.10 and a minimum of 0.03.

Numerous proposals have been made for determining the character and magnitude of the dynamic effects of driving. However, past designs have been on a semiempirical basis. For instance, 48-in. piles with 5-in. shells prestressed to 1100 psi have shown horizontal cracks when subjected to the blows of a 42,000 ft-lb single-acting hammer. Assuming a concrete strength of 7000 psi with a tensile strength of $7\sqrt{7000} = 586$ psi, the tensile stress reflected from the tip must have been in excess of 1686 psi. Generally, the compressive stress directly under the pile cap and cushion will be about twice this amount, viz., 3372 psi. These stresses usually occur when the pile tip en-
counters difficult penetration near the ground surface so that little energy of the hammer is expended in overcoming skin friction on the periphery of the pile.

**Wave equation**

Pile driving is fundamentally a problem in the transmission of longitudinal waves in prismatic bars. In 1931, Isaacs, of Australia, observed that wave action was present during the driving of piles. In 1934, the basic wave equation, well known to mathematicians, was derived by Timoshenko. It takes the form of a second-order partial differential equation:

\[
\frac{\delta^2 u}{\delta t^2} = C^2 \frac{\delta^2 u}{\delta x^2}
\]

where \( u \) = longitudinal displacement of a given cross section; \( x \) = distance from driven end of pile to cross section being considered; \( t \) = time interval between waves; and \( C \) = velocity of wave propagation in the \( x \) direction = \( \sqrt{E/\rho} \), where \( E \) = modulus of elasticity of the concrete and \( \rho \) = mass per unit volume of concrete. All units must be consistent.

This equation may be solved for simple cases by calculus, but it is extremely difficult when applied to practical problems. Its solution as applied to pile driving was first published in 1938 by Fox but the effects of the hammer ram, cap block, driving cap, cushion, pile, and foundation materials introduced formidable complications and simplifying assumptions were necessary. However, computers have made it possible to solve the problem by numerical methods, and developments in this direction appear to be most promising.
Smith outlined the results of studies of the use of computers and a numerical method suitable for computer programming. By this method all of the aforementioned variables can be introduced into the problem without seriously complicating the solution. In the author's opinion, this solution is the most promising approach to pile problems and further research in this direction will materially advance the art and science of foundation engineering.

Soil properties

Another challenging phase of pile design pertains to the resisting properties of soils. While internal friction and cohesion are important factors in all foundation problems, resistance to lateral deformation is equally important in the design of laterally loaded piles. It is well known that the soil modulus (resistance per unit of deformation) is a variable quantity in a given soil and also varies widely for different strata and degrees of compaction. A considerable amount of experimental research has been completed in this field but much remains to be done. The expression for soil modulus is

\[ E_s = \frac{k f y}{r} \]

where \( E_s \) = soil modulus (psi); \( k \) = constant which varies with the type and condition of soil and the method of installing the pile; \( y \) = lateral deflection of pile (in.); \( r \) = radius of pile (in.); and \( f \) = soil stress (psi) taken at a point on the stress-strain curve which has a strain equal to \( y \). The stress-strain curve is obtained by laboratory tests on soil samples.

Values of \( k \) ranging from 7 to 11 have been obtained from experimental data but continued research will eventually enable us to obtain more exact values for a given set of conditions. The Washington Department of Highways is presently conducting lateral load tests to establish the values of \( k \), as well as the point of maximum moment in the pile (Fig. 2-3).

Preliminary estimates of the required embedment of pile may be made by using the well-known pole formula, provided due consideration is given to the permissible lateral movement at the ground surface.

Maximum stress

Piles are designed as beam-columns subject to lateral and vertical loads at the ends. The maximum stress will occur at the lower end of the member, and for this reason maximum allowable stresses for the design of beams are permitted. At points in the upper one-half length of free-ended piles and the center one-half of fixed-
end piles, the unit stress must satisfy the equation:

\[ \frac{f_a}{F_a} + \frac{f_b}{F_b} = 1 \]

where \( f_a \) = computed axial stress; \( f_b \) = computed bending stress; \( F_a \) = allowable axial stress; and \( F_b \) = allowable bending stress.

The axial load multiplied by a suitable factor of safety must not exceed the Euler critical column load.

Washington designs require that each pile be cast as a unit.
Although a 28-day cylinder strength of 6000 psi is usually specified, 7500 psi or better is normally obtained. A strength of 5000 psi at the time of strand release is required.

Spiral reinforcement consists of ASTM A 82 cold-drawn No. 3 wire. At each end there are five turns at 3/4 in. spacing and 60 turns at 2 in. In the remainder of the pile, a maximum spacing of 4 in. is specified. A 2 x 1/4 in. steel ring is placed flush with the outside surface at each end of the pile to assist in resisting the bursting pressure under the driving cap.

**Splices**

Splices and connections to pile caps are made by placing a solid concrete plug to a depth equal to the outside diameter of the pile on each side of the splice point or below the cap (Fig. 4). The plug is reinforced to resist the external forces for which the pile is designed. To secure adequate bond between the plug and the pile, the inside surface of the pile is roughened with epoxy grout or the plug is placed against tacky epoxy. To reduce shrinkage, 2 oz of a reducing densifier and 1 2/3 oz of aluminum powder per sack of cement are added to the plug concrete.

Pick-up points for piles are carefully selected so as to avoid tension in the concrete shells. For stress analysis, an allowance for an impact force equal to the weight of the pile is added to the dead load. Care is exercised in detailing the shell at points of pick-up to provide for stress concentration. Often the holes provided for lifting are later utilized as vents to relieve water and air pressure built up within the pile during driving. These pressures could become sufficient to crack the spirally reinforced shell.

![Moment](https://placekitchen.com/180/180?text=Fig. 3b—Computed moment in pile)
PRESTRESSED CYLINDER PILES

Shear in kips

Load in kips per Foot

Fig. 3c—Computed shear in pile

Fig. 3d—Computed loading of pile
Three manufacturers in the Puget Sound area have fabricated prestressed concrete cylinder piles for the Department of Highways. Two of these fabricators are now forming the inner surface of pile shells by pulling a steel mandrel through the stationary steel outer form (Fig. 5-6).

The mandrel is moved through the full length of the pile at a rate of 6 to 12 in. per min. The rate of movement depends on the concrete mix and the temperature of the concrete. During casting operations, the mandrels are buoyed by the concrete in the bottom, so it is necessary to hold them down with pins inserted through the outer steel forms. Mandrels vary in length from 64 to 80 ft and each is provided with a tip shaped in a manner to facilitate movement. Each is equipped with form vibrators to assist in keeping the concrete in a semi-quick condition while the mandrel is in motion. Additional vibrators may be used on the outside form and in the concrete as it is placed through a horizontal slot running the full length of the top of the outer form. Both fabricators utilizing the mandrel method of forming encountered many problems and a few disappointments before
PRESTRESSED CYLINDER PILES

perfecting the method but, when properly controlled, the equipment is capable of producing excellent results.

High strength concrete for prestressed piles is readily obtained by local manufacturers due to the abundance of excellent aggregate and cement in the Puget Sound area. The cement conforms to ASTM C 150, Type II or III, and is usually used at the rate of about 8 sacks per cu yd to produce slumps of from 0 to 2 1/2 in. The slumps will vary from 0 to 1 1/2 in. without admixture, and from 1 to 2 1/2 in. with a set-retarding admixture but without air entrainment. In no case is use of calcium chloride permitted.

Both steam and moist hot air curing have been used in the manufacture of cylinder piles. Curing is continued until a 6 x 12 in. cylinder strength of 5000 psi has been attained.

Present equipment will permit the casting and handling of 54 in. outside diameter cylinder piles in lengths up to 175 ft.

Stress in the strands is measured by the jack pressure gage and is checked by the amount of elongation. The required tension is maintained by mechanical locks or anchors until time of release.

Experience to date indicates that the spiral reinforcement should be thoroughly tied to four or more #4 bars or welded to No. 3 gage longitudinal wires spaced about 15-in. apart.

INSTALLATION

Although several methods of preparing cohesive soils to receive large diameter cylinder piles have been studied, the most satisfactory method in aqueous environments is to bore holes in the foundation approximately 1 in. smaller in diameter than the outside of the piles. The piles are then dropped through the water, peat, or other material which has negligible resistance to penetration, and are than driven to bearing (Fig. 7). The boring and driving may be done with the aid of a template to a tolerance of 3 in. in any direction. Leads are not required for pile driving; however, the maximum permissible horizontal offset between the top and bottom of the pile must not exceed 0.5 percent of the pile length.

In granular soils, jetting is frequently required to sink the piles prior to final seating with the hammer.

When the point of the pile is passing through soft soil where there is little or no resistance to penetration at the tip, there is a possibility that longitudinal tensile stresses will be set up in the pile by elastic shock waves induced by driving. For this condition the stroke of the hammer should be reduced or additional cushioning provided in the helmet. When the pile is being driven into firm ground, the full stroke of the hammer should be used to develop the final driving resistance.
Soft soils and water may rise inside the pile to levels considerably above the original ground line. In this event, high internal pressures may be created due to confinement of materials beneath the driving cap. When this condition develops, the water and soil should be removed to avoid excessive hoop stresses.

Excessive manipulation of piles to force them into proper position should not be tolerated.

Pile cutoffs should be made with pneumatic tools, sawing, or other approved methods, but in no case should explosives be used.

A test pile is usually driven in accordance with the ENR formula and then load-tested to determine the correction for the formula before other piles are driven. This process may be repeated several times until a final corrected formula indicates the load-bearing capacity for a maximum settlement of $\frac{1}{4}$ in. in 48 hr. A factor of safety of 1.5 or 2 is generally specified. These test piles are usually of the same design and are driven at the same locations as permanent piles. If not damaged during testing they may be used as part of the permanent structure.

**SUMMARY**

Although the bearing capacities must depend upon the foundation materials encountered, the structural and
geometric properties of piles, and applied loading, Table 1 lists the nominal load and hammer capacities assumed for preliminary design.

Current contract prices in the Puget Sound area for structures supported on cylinder pile piers average about $11 per sq ft, while similar structures supported on conventional footings and piles cost approximately $15 to $18 per sq ft. The cost of furnishing 54-in. piles delivered at the site is about $23 per ft. Driving costs will vary, but they have averaged about $1500 per pile. For stream and lake crossings, estimates indicate that the difference in cost between hollow pile supported bridges and those of more conventional design is equal to or exceeds the cost of footings, seals, and cofferdams. In some instances, construction utilizing conventional footings and cofferdams would not be economically feasible and a relocation of alignment would be mandatory.

Prestressed cylinder piles have made a great impact on bridge construction and, as more is learned of their real potential, new and more economical construction will ensue.

**REFERENCES**


6. Patterson, Donald, Pole Type Buildings, American Wood Preservers Institute, Chicago, 1957, 63 pp.
Describes the features of 20 in. square hollow piles for Portland Harbor's Pier 4. The mysterious break of the first two driven under test is described and the corrections applied both to manufacturing and driving techniques are explained.

HANDLING AND DRIVING
PRESTRESSED CONCRETE PILES

By Thane E. Brown

The new Pier 4 in Portland, Ore., for bulk discharge is the first concrete dock in that harbor. This is especially unusual there in the heart of the Douglas fir region where treated wood piling and timbers, with a life expectancy of 40-50 years in that fresh water, would be the most universal method of construction. However, the 900-ton-per-hr bulk unloader to be used at this facility had such heavy wheel loads, that careful preliminary studies revealed the greatest efficiency and economy to be in a concrete structure. The final design called for all prestressed concrete piling and standard reinforced concrete caps, beams, and deck.

The preliminary design of the 110 x 760 ft open pile structure called for the use of 20 in. square prestressed piles with hollow cores for the outer half of the dock where railroad and crane loads occurred, and 16-in. octagonal, solid, prestressed concrete piles for the inner portion of the pier (to be used for stockpile of bulk materials). Pile bent spacing was 30 ft on the outer section where 20-in. piles were used, and 15 ft on the inboard section where 16-in. piles were used.

The final design called for 5000 psi concrete, 3500 psi before release of the prestressing strands, and an initial prestress calculated at 745 psi. Lengths were varied from 80 to 120 ft.
Following the soil borings, a test pile program was immediately pursued. Fifteen of the 20-in. piles were cast, driven on location, and left in place so that they could be incorporated into the final structure. Preliminary design called for a five pile cluster under each crane rail at each bent. One test cluster was driven near the inner end of the dock where the soil was a fine silty sand, and another five pile group was driven near the outer end of the dock where a medium to dense sand was encountered. Five other individual piles were scattered throughout the site.

Since there was no pile driver in Portland rigged to handle 120-ft. piles of this type, and since the test program did not warrant building a special driver for this job, a practical compromise was reached with a barge mounted crane. Driving accuracy was controlled by a lower set of guides fixed to the deck and an upper set of swinging leads carrying a 30,225 ft-lb hammer.

The first test pile, a hollow core pile, broke under the hammer after 14 blows while moving 1/8 in. per blow. It had penetrated 72 ft into soft ground. A second test pile was set up and was driven 74 ft into the ground, where it too, mysteriously broke under the hammer while moving at the rate of 11 blows per ft. A third test pile was driven successfully.

A diver was sent down to cut the strands and retrieve the top sections of the first two broken piles for inspection. The 12 in. core form was out of position as much as 1 1/2 in. in these sections. Insufficient holddowns in the forms, spaced about 20 ft on center, had allowed the form to shift while the concrete was being vibrated. The form displaced the strands resulting in a weak eccentric pile. The remaining 12 test piles then became suspect. The problem was to try to determine the position of the forms in these piles.

An investigation was performed to determine the core location by the radiographic method using Iridium 192 for the radiation source. By checking instrument readings against physical measurements on the open end of a broken stub, the accuracy of the radiographic equipment was established as being between 1/2 and 1 in. of actual location. The specified tolerance for position of the form was 1/4 in., and since acceptance or rejection of an expensive piling was at stake, it was thought best to use a more positive method of measurement than the radiographic system. Three holes, 1/4 in. in diameter, were drilled in each pile at a specified location in the "upper face," and the distance to the core measured. As a result of these measurements, all undriven piles were rejected as not meeting specifications with respect to accuracy of workmanship, and 12 new piles were ordered to be cast. Core form holddowns were revised to provide control on 3 ft centers and probe tests were made with a wire immediately after casting before the concrete had set (Fig. 3). No floating of the core forms was experienced.

During the driving of the first three test piles, a number of circumferential cracks developed under the hammer blows. It was noted that these occurred...
Fig. 1—Plan of Pier 4, Portland, Ore.
at the locations opposite joints in the steel forms or about every 20 ft. These rough spots in casting of future piles were practically removed by carefully taping the joints in the steel forms, thus eliminating a potential plane of weakness.

It was felt that part of the trouble in the breaking of the first two piles might also be attributed to the loose manner in which the pile hammer fitted on the pile head. The pile head was square, as was the follower on the steam hammer, but scar marks on the side of the pile head showed that the hammer had rubbed as it struck. If the hammer was not square and concentric with the pile head, then it would strike an oblique blow and could impart bending and torsion in addition to axial compression. The third test pile had the square corners of its head rounded off before driving so that the hammer could not impart a torsional load. The follower on the hammer, and its lower guide were modified into a circle. All remaining recast piles had the top 12 in. cast into a 19\(\frac{1}{2}\) in. diameter circle (Fig. 4). This feature was also adopted in the final design of the permanent dock.

**Driving**

Whenever the soil offered little resistance to driving, a single blow of the hammer would often send the pile down a foot or more. The shock wave from the hammer in this case would not be transmitted to and absorbed by the ground beneath, but would return up the pile in a recoil action as a tension wave. This tended to open cracks or to develop new cracks that were not previously discernable by eye. To relieve this situation, it was decided to half-stroke the hammer at the start of driving and to change over to full stroke only when the pile had penetrated into the ground 25 ft or more, or until driving resistance
was encountered such as 25 to 30 blows or more per foot. While these limits were arbitrary and set empirically, the problem of pile cracking was practically nonexistent after this change in driving technique. This policy of half-stroke on the hammer in soft driving was also followed on the main pier construction to follow.

A cushion of 6 or 7 layers of 1 in. Douglas fir boards was placed directly on the top of the pile. This worked satisfactorily as noted in the excellent condition of the pile heads after driving. Piles driven later during construction of the pier had a cushion consisting of four laminations of 1-in. fir.

**Load testing**

The remaining 12 test piles were driven without incident. Accurate driving records were kept throughout. The load test program was then started. A five pile cluster, directly under a crane rail was selected. A steel jacking frame spanned across the four outside piles and supported a steel beam that crossed over the top of the center pile. Four 100-ton jacks were calibrated and used to load the center pile. Strand vises connected to the exposed prestressing cables of the outside piles provided the uplift resistance for the center jacks.

This first pile was loaded to 250 tons with a total net settlement of just over \( \frac{1}{2} \) in. It was driven 66 ft into the ground at the rate of 84 blows for the last foot. Other piles in this five pile cluster were also driven to elevation minus 90, but were experiencing soft driving, such as 30 blows per last foot. As a result of this first successful load test in a location showing poor soil it was felt that a design load of 125 tons per pile was tentatively justified. However, other tests were to follow. The second set of five piles to be tested were driven under a future crane rail. Here the borings showed better soil and the driving was harder. Blows for the last foot were mostly 90 to 120. For fear of breaking the piling, driving was stopped after a penetration of only 27 ft and the top 20 ft of the piles were cut off. The load test equipment was set up and the
The pile driver used is of interest. There was no driver in the Portland Harbor large enough for the job, so a special one had to be built (Fig. 6). It was powered by two 70 hp boilers, and mounted on a new steel barge.
34 ft wide by 109 ft long. It was equipped with 120 ft steel fixed leads. The hammer was a special Raymond 00 type. Two 5 in. jet pumps provided water for the twin jets on either side of the leads. The leads were braced 70 ft above the deck and connected at the bottom to a sliding "spotter," a steel frame that shuttled in or out 20 ft to permit accurate plumbing and spotting of the pile. The pile was held in position at the bottom by the pile guide which had a hinged gate. This gate swung open to receive the pile and then closed to hold the pile as a bottom guide. The floating driver had the usual deck winches and rigging so that with the help of anchors and deadmen, it could be maneuvered from pile to pile.

Alignment

The soft driving encountered in much of the job site caused some problems in pile alignment. The specifications called for final position of the pile head to be within 6 in. of specified position, with a widening of the cap required for any pile over 3 in. out of position. While excellent alignment on the finished job was actually accomplished, in some cases the pile would spring well out of tolerance position when the hammer was removed. Temporary timber stays were permitted to hold the piles in the position they were driven. After a few days, when the stays were removed, the ground was found to have "frozen" around the pile and the desired position was maintained.

As a result of test pile data, the minimum penetration of piling that would be required to obtain sufficient load capacity of each pile was reasonably certain. Therefore, the length of every pile was carefully established in advance, so that the costly expense of cutting off a prestressed concrete pile after driving would be avoided. As a result, an unusual tolerance specification was included whereby the pile was to be driven until its head was at proper cutoff elevation, with a vertical tolerance of plus or minus 1/2 in. It was further stated that if driving conditions developed that made this impractical, the pile would then be cut off at the contractor's expense, and if overdriven, the cap soffit could be lowered, or the pile built up at the contractor's expense.

This specification on vertical tolerance was quite satisfactory and caused no hardship on the contractor. None of the piling had to be cut off because of inability to drive down, and only five were accidentally driven below cutoff.

Realigning piles

When setting cap forms, it was often necessary or desirable to pull a pile under the cap, in order to better fit the form. This procedure of pulling a pile into alignment was permitted
by the terms of the specifications provided it was accomplished "with a force not to exceed 1000 lb." The purpose of this specification was to avoid creating excessive bending stresses in the long piles. This was theoretically sound but when the job was under-way the problem arose of how to enforce it.

Tension gages inserted in a line would work, but they are costly and it was questionable whether the contractor could justify the investment, and there was a question of the life expectancy of this delicate equipment. Finally, a simple inexpensive method was devised that worked to everyone's satisfaction. A seven strand, 1/16-in. diameter wire rope (airplane control cable), was tested to failure in the laboratory. The breaking strength was just over 500 lb. Double strands were made up into assemblies 17 in. long, with shackles at each end for convenience in inserting in direct line of pull of the contractor's come-along rigging. Inserted between shackles and parallel to the light wire strand was a loose safety chain. If the maximum load of 1000 lb was exceeded and the strand broke, the safety chain would prevent injury to workmen or loss of equipment.

An average of seven piles per day was maintained as a driving schedule, exclusive of shut downs.

The main dock has 343 concrete prestressed piles of the 20 in. square type totalling 36,600 lineal ft, and 352 of the 16 in. octagonal prestressed concrete piles, totalling 34,400 lineal ft. There were 2200 cu yd of precast beams, and 5100 cu yd of cast-in-place concrete caps, deck, and utility tunnel.

**PERFORMANCE**

Frequent check levels are run on the pier because of the soft driving of the piling that was encountered. The extreme northeast corner of the pier settled slightly when the backfill was placed with compactors, but no

![Fig. 7—Pier 4 shortly after completion](image-url)
structural distress was noted, and the settlement was not considered serious.

The 110 x 760 ft concrete dock was designed as a continuous span structure with no expansion joints. Having been through an annual temperature range of 100°F, its behavior has been as anticipated. Some small cracks developed during construction in several pile caps and in pile heads where buildups were installed. No advance in these cracks because of temperature, or deflection of the structure from unloader, rail trains, or ship impact has been detected.

The test piles were manufactured by Empire Prestress Co. and driven by General Construction Co. Pier 4 structure was built by C. M. Corkum Co., with the prestressed piles made by Ross Island Sand and Gravel Co., and driven by Raymond Concrete Pile Co. All engineering design and supervision was performed by the staff of the Portland Public Docks under direction of the author.
Reports the design of the apron for Port of Seattle's Pier 28. Included are the development of design criteria, economic considerations, and details of the prestressed concrete pile substructure and precast, prestressed concrete apron superstructure.

APRON DESIGN FOR PORT OF SEATTLE'S PIER 28

By Wheeler H. Rucker, Jr.

Development of a marine terminal facility such as Port of Seattle's Pier 28 requires, as one of the initial steps, the establishment of economic and structural design criteria. Such criteria may not be obtained from codes or other publications as it is dependent on the specific usage of the completed facility and must be developed exclusively for each installation.

Pier 28 is intended for primary use as a general cargo terminal with most shipments made up from a number of small consignments. The terminal must also have a secondary capability of accommodating the increasingly popular use of large containers for shipment of all types of cargo.

Structural design criteria

Small consignments of general cargo are usually handled on pallets by 5 to 7-ton fork lift trucks. Typical commodities weigh 40 lb per cu ft and may be stacked as high as 15 ft. From this is obtained a requirement for a uniform load capacity of 600 lb per sq ft.

The use of containers for the shipment of general cargo has been rapidly gaining in popularity. Vans, the latest development of the container principle, are made to ride a freight car or fit a truck body. They are usually 8 ft wide to match legal truck widths, 8 ft high to come just under highway clearance heights, and up to 25 ft long. Their maximum gross weight, which is limited by highway load restrictions, is 30 tons. Vans are transported to and from shipside along the apron structure by fork lift trucks. Transfer between the apron and the ship is done with gantry cranes, which may also be used to handle other
heavy lifts weighing as much as 50 tons. Use of this equipment dictates that the apron be designed to support the concentrated loads from a 30-ton fork lift truck anywhere on the apron or a 50-ton traveling gantry crane under the crane rails.

To provide complete flexibility of operation, the structure is also designed to support a 30-ton truck crane at any location and the standard Cooper’s E-50 railroad loading on each of the apron tracks.

In addition to the gravity loads, the apron structure is designed to withstand the kinetic energy of a 25,000-ton ship with a velocity of 2 ft per sec striking the bullrail at an angle of 10 deg. The seismic load used in design is equal to 6 percent of the total of the dead load plus one-fourth of the uniform live load acting in any direction in the plane of the deck.

Economic considerations

Study of existing ocean terminal facilities in the Pacific Northwest discloses an almost universal previous use of creosoted timber because of its availability and low cost. These facilities are not, however, designed to support the heavy concentrated loads required to support container-handling equipment.

For Pier 28, alternate designs in timber or concrete construction were prepared, using the same load criteria for each. The cost estimate for the concrete apron, including dredging, rip-rap, bulkhead, trackage, asphaltic concrete surfacing, fender system, and utilities was $12.20 per sq ft. For a creosoted timber apron, on the same basis as the concrete one, the estimated cost was $11.20 per sq ft.

Initial cost does not, however, present the total picture as to cost of a facility. Not included are such considerations as life of the facility, maintenance costs, and interest on the investment. The Port of Seattle is primarily concerned with capitalized cost which may be defined as an amount of money sufficient to construct a facility and to provide a reserve which, when invested at a fixed rate of interest, will pay all maintenance expenses during its life with enough left over to replace the initial amount at the end of the life of the facility. This concept of capitalized cost is used only for the purpose of comparison of various types of construction and is not, for several reasons, followed in the budgetary procedure of Port of Seattle.

Table 1 compares the alternate timber or concrete aprons for Pier 28 on a capitalized cost basis. Cost of future removal of the facility prior to reconstruction was not considered in the data of Table 1.

It was on the basis of this type of capitalized cost analysis that the decision was made to prepare final contract documents for a prestressed concrete apron structure.

Fig. 1 shows a typical section through the apron of Pier 28. Prestressed concrete piling are driven on 10-ft centers in bents spaced on 20-ft centers. Conventionally reinforced cast-in-place subpile caps are then constructed; precast, prestressed concrete deck panels are positioned; and closure sections placed. To provide a maximum of flexibility for future operations, the entire area under railroad and crane rails has been depressed so that rails and timber ties may be installed in the conventional manner in rock ballast.

ACI member WHEELER H. RUCKER, JR., supervising senior engineer, Port of Seattle, is responsible for all design and specifications for the Port. A registered professional engineer, he joined the Port of Seattle engineering staff in 1959.
TABLE 1—ALTERNATE DESIGNS COMPARED BY CAPITALIZED COST

<table>
<thead>
<tr>
<th>Item</th>
<th>Timber ($/ft²)</th>
<th>Concrete ($/ft²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial cost per sq ft, C</td>
<td>11.20</td>
<td>12.20</td>
</tr>
<tr>
<td>Life span in years, n</td>
<td>35</td>
<td>50</td>
</tr>
<tr>
<td>Annual depreciation, D</td>
<td>0.152</td>
<td>0.080</td>
</tr>
<tr>
<td>$D = \frac{Ci}{(1+i)^n - 1}$</td>
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<td></td>
</tr>
<tr>
<td>Interest rate of invested capital, i, percent</td>
<td>4.0</td>
<td>4.0</td>
</tr>
<tr>
<td>Annual maintenance cost, O</td>
<td>0.05</td>
<td>0.02</td>
</tr>
<tr>
<td>Capitalized cost, $S$</td>
<td>$16.25$</td>
<td>$14.70$</td>
</tr>
<tr>
<td>$S = C + \frac{O + D}{i}$</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

APRON SUBSTRUCTURE

**Prestressed concrete pile design**

The solid prestressed concrete piling in general use in the United States are usually octagonal or square in cross section and range in least dimension, face to face, from 12 to 20 in. Piling with a least dimension greater than this are usually designed with a hollow core to decrease the pile weight for ease of handling.

Only limitations of transportation and handling govern the maximum allowable length of prestressed concrete piling. It is felt that for marine construction in the Puget Sound area, piles 130 ft long are feasible provided they are driven in 30 to 40 ft of water. For greater depths, it is conceivable that even longer piles could be successfully used.

Prestressed concrete piling used in Pier 28 were designed on the basis of ultimate strength theories to have a minimum safety factor of 4 for direct load and 2 for bending or buckling.

**Determination of required prestress force**

Perhaps the most controversial of considerations relative to the design of prestressed concrete piles is the determination of the amount of prestress required. To determine this required amount of prestress, it is first necessary to understand the loading to which the prestressed pile will be subjected. These loadings may be classified principally in three groupings: (1) loads due to transportation and handling, (2) loads due to driving, and (3) in-service loads.

During transportation and handling of the prestressed pile, it is loaded with its own dead load as a beam. In order not to overstress the pile in bending due to this loading, the designer should establish the location of the pickup points to be used.

During driving of a pile, the impact of the driving hammer on the head of the pile causes a compression
shock wave which travels from the head end to the tip end and is then reflected from the tip, returning as a tensile shock wave to the pile head. The intensity of this tensile shock wave is related to the velocity of the driving hammer, the relative weights of hammer and pile, the nature of the material into which the pile is driven, and the nature of the cushion between the pile head and the driving hammer. Horizontal cracks will appear in the pile during driving if the maximum intensity of the tensile shock wave is greater than the total of the effective prestress in the pile and the modulus of rupture of the concrete.

The amount of prestress usually used to resist driving stresses encountered in marine construction ranges from 700 to 1200 psi. Under average driving conditions and by exercising care in driving, the lower of these values has proved to be satisfactory in many cases. However, there are also many instances on record of severe cracking or complete failure of the pile during driving, particularly when the effective prestress is in the range of from 700 to 850 psi. Likewise, there are instances of tensile cracks appearing during driving of concrete piles stressed to 1200 psi.

In-service loads may be either direct load from the superimposed weight of the superstructure and the live load, or a combination of direct load and bending due to the superimposed load plus either a lateral seismic load or the load caused by floating objects beating against the piles. Another factor which may affect the in-service longevity of prestressed concrete piles is the possibility of damage to the prestress strands due to corrosion. The nature of prestressing tends to close up any cracks which may have occurred. However, floating objects beating against a pile could conceivably either severely crack or spall the concrete and subsequent repetitions of this load throughout the years could result in eventual exposure of the stressing steel.
As a result of careful consideration of each of the aforementioned factors, it was decided that all prestressed concrete piling used in Pier 28 should have a minimum residual prestress, after losses, of 1100 psi.

**Selection of pile size**

Selection of the most economical size of prestressed concrete piling for Pier 28 was made on the basis of least cost per square foot of deck for the supporting piling. For various distances from the bottom of the pile cap to the assumed point of fixity 15 ft below the ground surface, pile capacities were computed for a range of pile sizes. This computed capacity was the lesser of the short column capacity (based on a safety factor of 4) and the capacity as governed by buckling (based on Euler's formula with one end fixed, one end half-fixed, and a safety factor of 2).

Knowing the structural load capacity of a pile, it is a relatively simple matter to compute the penetration required to develop the pile through friction, the cost of the pile, the tributary area of deck which may be supported by one pile, and the pile cost per square foot of deck area. Next, for any given distance from the bottom of the pile cap to the point of fixity of the pile in the ground, a curve may be plotted showing the relationship of pile size to cost per square foot of deck area. Such a curve is shown in Fig. 2.

It can be seen from Fig. 2 that the most economical pile size is one where the allowable load capacity as governed by buckling is equal to the allowable capacity for direct load.

This type of analysis was followed at Pier 28 and resulted in the use of 16½ in. octagonal piles under the depressed deck section and 14 in. square piles for the remainder of the apron. The design load used for the 16½ in. octagonal piles was 100 tons and for the 14 in. square piles was 85 tons.
The apron superstructure is designed to support the loads previously listed and to transmit these loads to the supporting substructure. In addition, it is designed for low maintenance costs and to provide adaptability for possible future modifications which may be desirable to meet the requirements of new developments in cargo handling procedures.

To resist damage from heavy concentrated loads being dropped on the deck and to provide sufficient mass to dissipate lateral ship impact loads, a minimum deck panel thickness of 10 in. was used. A bent spacing of 20 ft was selected as being consistent with this thickness of prestressed concrete slab and the applied loads. This spacing also proved to be economical from the standpoint of pile spacing and bay width of the transit shed structure which is partially supported on a portion of the apron.

**Details of construction**

Much economy can be realized in construction by the simplification of details; standardization of components and the use of construction tolerances as unrestricted as possible, consistent with the intended usage of the completed facility. At Pier 28, the pile caps are cast-in-place using standard form units for all bents. Any deviation in pile location is quite easily accommodated by this procedure. As shown in Fig. 3, the pile caps are cast in two stages with the first stage placement up to the soffit of the precast deck panels.

Next, the 6 ft wide precast, prestressed deck panels, nominally 18 ft 3 in. long, are set in place. The panels are pretensioned for positive moment and conventionally reinforced for negative moment and to provide continuity over the pile caps. Negative reinforcement is spliced by welding opposing bars inside the legs of splice angles at each bent.

After the precast panels have been set and the splices made in the negative reinforcement, the upper portion of the pile cap is cast. Both stages of the pile cap are designed to ac
APRON DESIGN FOR PIER 28

compositely with the deck panels for maximum economy.

Inasmuch as it is virtually impossible to accurately control the camber of the prestressed deck panel units, an asphaltic concrete leveling and wearing course is provided.

The total contract cost for this apron construction was $12.80 per sq ft of which $8.88 was for the apron structure itself and the remainder was for dredging, rip-rap, utilities, trackage, bulkhead, fender system, and miscellaneous appurtenances.

SUMMARY

The economic design of any structural system is dependent on a thorough understanding of the loads to which the system will be subjected and of the characteristics and limitations of the various components which make up the system. At Pier 28, the apron is composed of precast concrete piling and deck panels connected together with cast-in-place pile caps. The efficacy of the system depends on the satisfactory performance of each of these components as well as complementary interaction.

Use of ultimate strength formulas throughout the design results in a well-balanced structure and much economy, particularly in the case of the piling. Selection of the proper pile size will also result in economy. The use of a pile larger than the most economical size will not appreciably increase the total cost of piling, but it will add considerably to the cost of the deck through use of longer spans. On the other hand, use of piles that are too small will result in excessive pile costs and less stability with little savings in deck cost as compensation.

In design of the deck, it appears that the most economy is obtained when the moment requirements for the design uniform loads and concentrated loads are nearly the same. It is also desirable to have a section of sufficient mass to resist ship impact without sacrificing economy of design to obtain that mass.

Economical limitations

There are several factors which could seriously affect the financial feasibility of construction of an apron structure similar to the one at Pier 28. For example, private enterprise operating on risk capital could probably not afford the capitalized cost basis of initial project comparison, and would select that type of construction which could be built for the least initial cost. Tidal range and water depth requirements would affect the pile costs significantly. The span length for the type of deck construction used is also quite critical, having an economic limitation to a span between 18 and 24 ft.

Of course a change in loading criteria or size of the total apron structure included in one contract would be of basic concern. In terms of economic feasibility, it is felt that this type of construction could not be justified for projects of less than 50,000 sq ft unless forms for the precast elements are readily available near the project site.

For the particular operational requirements and physical situation of the Port of Seattle at Pier 28, it is felt that the precast, prestressed method of apron construction employed has resulted in a facility having maximum economy, flexibility, and ease of maintenance which will provide good service for many years beyond the 50 on which its feasibility is based.
CONSTRUCTION AND PERFORMANCE
OF HOOD CANAL FLOATING BRIDGE

By C. C. Nichols

The Hood Canal is a westerly arm of Washington State's Puget Sound; about 55 miles long and from 1½ to 2 miles wide. This canal has presented a major water barrier to travel between the Seattle metropolitan area and the Olympic Peninsula to the west. With the desire to improve trans-Puget Sound transportation, the Washington Toll Bridge Authority authorized the construction of the Hood Canal Bridge. The bridge consists of four basic units: (1) floating structure, (2) fixed structure approaches to each end of the floating structure, (3) west road approach and (4) east road approach.

Of primary interest is the 6470-ft floating structure. A floating structure was selected for the major portion of the crossing because of the great depth of water to be found in Hood Canal. At the bridge site, water depths vary from 70 ft at the shoreward ends of the floating structure to a maximum of 340 ft at the draw spans. In addition, these depths exceed 180 ft for about 4000 ft along the axis of the bridge. Such depths of water preclude the economical construction of a structure supported on piers, including structures of the suspension type. Since the bridge is situated on tidal waters provisions had to be made for a tidal range of 18 ft.

To accommodate large vessels, a
draw span was incorporated in the structure which, when in its open position (Fig. 1), provides a channel 600 ft wide. Secondary navigation channels 200 ft wide are provided at each end of the floating structure by the transition truss spans connecting the fixed structure to the pontoons.

The central draw unit consists of flanking or guide pontoons and two draw or movable pontoons. The flanking pontoons are U-shaped structures with two separate hulls each 23 ft wide with a 56-ft spacing giving an over-all width of 102 ft. In its normal closed position, each draw or movable pontoon is positioned in this lagoon with 302.5 ft of its length extending beyond the flanking pontoon and 168 ft engaged by the flanking pontoon hulls. The draw pontoons are held in alignment at the center of the channel with a mechanical shear key and locking devices. Close vertical and horizontal alignment between the flanking and draw pontoons is maintained by guide rollers positioned near the ends of the normal engagement length of 168 ft.

When necessary to open the unit, the draw pontoon is retracted 303 into the lagoon between the flanking pontoon hulls at each side of the channel. Power is provided by four 40-hp d-c motors for each pontoon. The power is transmitted through gear reducers and pinions on the flanking pontoons geared to racks running full length on each side of the draw pontoons.

To prevent salt spray from interfering with vehicular traffic, the roadway is elevated a minimum of 14 ft 5 in. above the pontoon deck. The roadway deck increases in height at each end of the floating structure to provide the required navigation clearances under the transition truss spans. Maximum height of the roadway above the pontoon deck is 53 ft 7 in.

**PONTOON CONCRETING**

A balance must be achieved between strength requirements, flotation characteristics, and economy in the design of any pontoon bridge. For any given strength requirement, the requirements of flotation and the interests of economy are best served by a construction using walls and slabs of minimum thickness consistent with proper and efficient placement of concrete.

In the Hood Canal Bridge, a 9-in. thickness was selected for the exterior walls and bottom slabs and a 6-in. thickness selected for the interior dia-

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CHARLES C. NICHOLS, engineer of toll facilities, Washington State Highway Commission, Olympia, Wash., was in charge of completion of construction and the strengthening and modification program of the Hood Canal Bridge.

phram walls. Top slabs of the pontoons were made 7 in. thick. A typical standard pontoon is 50 ft wide, 1-ft deep, and 360 ft long. The pontoon hull section (top slab excluded) was post-tensioned with 43 tendons with each tendon composed of eight 3/8 in. diameter strands enclosed in a 15/32 in. outside diameter conduit. Mild steel was used as local reinforcement in the pontoon hull section while mild steel comprised both principal and local reinforcement in the top slab of the pontoon.

The pontoons and anchors of the Hood Canal Bridge were built in a construction area along the Duwamish River in Seattle in a graving dock, 12 ft wide, 408 ft long, and 18 ft deep. The graving dock was of sufficient size to permit two standard size ponton
toons to be constructed simultaneously. Included in the work area facilities, were the customary framing yard, concrete batch plant, traveling cranes, shops, warehouses, and job offices. An outfitting dock fronted along the river.

Steel forms were used to fabricate the pontoons. Special sections, not readily adaptable to the use of steel forms were framed in wood. Before placing any forms, however, the timber floor of the graving dock was given one or two coats of a wax compound to help break the bond between the pontoon concrete and the timber floor. Side and end forms were then placed and reinforcing steel and prestress tendons installed according to plan.

At the completion of a graving dock construction cycle, the gates were opened allowing the dock to flood. The pontoons were then towed out of the dock and moored at the outfitting dock completion of the top slab where necessary and erection of the superstructure.

Mix proportions

Specifications for the project called for continuous placement of concrete in the pontoon hull with no cold joints. All concrete used in the pontoon hulls was designed to have a compressive strength of 3000 psi at 10 days. The concrete was proportioned as follows:

Cement .................. 5.75 sacks per cu yd
Coarse aggregate, dry ...... 814 lb per cu yd
Intermediate aggregate,
dry ...................... 1172 lb per cu yd
Sand, dry .................. 1270 lb per cu yd
Maximum total water ...... 5.0 gal. per sack

A water-reducing admixture and an air-entraining admixture were added to the concrete to improve workability and control shrinkage and setting time.

All concrete was batched immediately adjacent to the graving dock where the pontoons were constructed. The contractor's batch plant was automatic; the two mixers had a capacity of 2 cu yd. Provisions were made for automatic dispensing of admixtures.
Fig. 2—Cut-away drawing of a standard pontoon

Placement

Concrete was discharged into 2-cu yd buckets, then by flatbed truck and crane to a hopper at the end of one of the two placing bridges spanning the dock. Each bridge was equipped fore and aft with numerous metal chutes discharging into rubber elephant trunks extending to the top of the forms. In the forms, sheet metal chutes were placed between the form and the reinforcing steel to confine the flow of concrete and limit segregation.

Concreting commenced with the first bridge which would start at one end filling the transverse and longitudinal walls to a height of about 5 ft letting the concrete spill out into the bottom slab area. The second bridge was usually from 1 to 2 hr behind the first bridge and filled the bottom slabs and completed placement in the walls. Care was taken to place the concrete as close to its final position as possible to avoid excessive drifting. When concrete from the second bridge was placed, the first lift of concrete was revibrated by allowing the vibrators to sink of their own weight and seek their own level in the partially stiffened mass. Care was also taken to insure adequate but not excessive vibration of the concrete at all locations.

A standard pontoon hull consisted of 1320 cu yd of concrete and averaged 33 hr for placement for an average rate of 40 cu yd per hr. Certain special pontoons required continuous hull placements lasting up to 70 hr.

After stripping the forms, small honeycombs or rock pockets were occasionally found but these were of a minor nature and easily repaired. The concrete was cured by continuous application of water until the pontoons were launched.

Typical average concrete strengths based on cylinder tests were as follows: 4400 psi at 10 days, 6800 psi at 28 days, and 9000 psi at 1 year.

The pontoon hulls were post-tensioned after cylinder tests indicated strength of 3000 psi. The prestressed elements of the original design im
an average prestress in the concrete of 450 psi in the bottom slab varying to 220 psi in the top of the walls. Subsequently, the pontoons were strengthened by the addition of 24 prestressing tendons each composed of 40 high strength wires of \( \frac{1}{4} \) in. diameter. This prestress force was applied symmetrically and resulted in an added concrete prestress of 430 psi, giving a maximum prestress in the bottom slab of 880 psi.
ANCHORS

Anchorage of the floating structure posed many unusual and unprecedented problems. Both lateral and longitudinal anchorage systems were necessary which would assure stability of the structure under the combined efforts of wind, wave, and tidal forces yet be flexible enough to accommodate the 18-ft maximum change in the tidal level. The only feasible method of providing lateral anchorage was by means of cables connecting the pontoons to concrete anchors on the bottom of the canal (Fig. 5). The concrete anchors were constructed in the graving dock in much the same way as the pontoons and with the same concrete mix specified for the pontoons.

The anchors are 19 ft wide, 40 ft long, and 15 3/4 ft high and are of cellular construction. The anchors were towed near the bridge site in groups of six and temporarily stored on the beach. To lower an anchor weighing 530 tons required an elaborate plant. Two barges, each measuring 34 by 200 ft, were situated 23 ft apart with two 80-ft girders spanning between and across the barges. Head blocks and floating blocks were mounted on the girders and reeved with 11 parts of 1 1/8 in. diameter wire rope from two 2-drum hoists rated at 20 tons. The anchor was floated into position between the barges and floating blocks attached with 6-in. pins to each of the four lift columns embedded in the anchor. The anchor was then filled with concrete and lowered to the canal bottom.

Anchor cables

The anchor cables are designed to have equal stress at a mean water elevation with no external forces acting on the bridge. Only under this ideal situation will the bridge be on true center line. Whenever wind, wave,
or tidal forces are acting on the structure, it will move off center line until the load in the windward cables increases to a value equal to the external forces acting on the bridge.

The length of the cables was selected such that at high tide with maximum wind, wave, and tide forces acting on the bridge, the load in any cable would not exceed the allowable working stress. A large percentage of the maximum stress in any cable is due to the change in length of a cable from mean water to extreme high water. The closer an anchor is to the bridge, the greater this increment of load becomes as the tide rises. Consequently, the cables are relatively long. Distances from the pontoons to the anchors varying from 610 ft in 58 ft of water to 1220 ft in 340 ft of water. The anchor lines are continuous 1¾ in. two part bridge strand cables.

After intensive study of various methods of combating the corrosive effects of salt water on the cables, it was decided that cathodic protection would give the best results. Graphite anodes were suspended from the pontoons at each pair of anchor cables. Electric current impressed through these anodes travels to the anchor cables and thence to a resistance box in the pontoons.

No anchor cables were utilized for the longitudinal anchorage of the bridge. Instead, the floating structure is anchored to the channelward piers on each side of the canal with a steel strut connecting the top of the pier with the top of a steel tower on the end or cross-pontoons. This strut is provided with universal joints at each end to accommodate tidal changes and the small lateral movements of the floating structure due to wind, wave, and tide forces.

**TRANSITION SPAN AND PONTOON INSTALLATION**

Erection of the transition trusses posed interested problems. Rather than erect the spans in place of falsework piling, the contractor elected to float the spans into position and transfer the weight of truss from the barge to the pier and pontoon tower by using the changing tide level and ballast water. The barge used to float the truss into position was a 210-ft pontoon which now is part of one of the movable pontoons.

Timber falsework was erected to the correct height on the pontoon. The truss was then erected on the falsework. The unit was then towed to the bridge site and positioned between the pier and tower on the floating structure at high tide. Water was pumped into the pontoon supporting the tower to lower it approximately 2 ft. As the tide dropped, water was pumped out of the pontoon until the wind shoe connections at each end of the truss could be bolted securely. At this time, virtually all of the weight of the truss was still on the falsework. As the tide continued to drop, all of the remaining water in the pontoon was pumped out and the truss was transferred gradually to its permanent supports.

**Ponoon installation**

Pontoons and anchors were towed to bridge site by ocean-going tugs (Fig. 6). Generally, two tugs were used with one pulling the pontoon and the other ranging alongside in reserve. The distance from the construction area in Seattle to the bridge site is about 35 nautical miles and the tows averaged approximately 12 hr each.

Pontoons were first moored near the bridge site, where they were bolted
together in units of two or three pontoons. Abutting ends of the pontoons were brought together and shear keys engaged. High strength bolts were then inserted through pipe sleeves and as they were tightened, the rubber seal on the exterior face of the bulkhead compressed to the point where a watertight seal was attained. At this point there was a gap of approximately 1½ in. between the abutting pontoons. This space was filled with grout. After the grout hardened, the bolts were tightened to their design load.

**PONTOON MODIFICATION**

During the winter of 1959-1960, severe storms caused damage to the pontoons of the partially assembled bridge on the west side of the canal. The damage was particularly noted in the bolted joints connecting the pontoons. As a result of this development, consulting engineers were retained to review the circumstances and analyze the structure. It was their recommendation that it would be prudent to strengthen the pontoons and their connections.

The pontoons were modified by the addition of post-tensioning cables installed within the pontoons and use of an epoxy-resin grout in the connections between the pontoons. In the standard pontoons, cables were installed immediately below the top slab and just above the bottom slab with 12 cables in each location. This required about 10,000 holes to be cored through the transverse walls.

After cables were tensioned to their design load, all holes in alternate bulkheads were thoroughly sealed to provide watertight bulkheads. The cables were tensioned in lengths up to 1050 ft in one operation with hydraulic jacks at each end. After pontoon connections had been made at the bridge site, additional cables were installed across the joints to further strengthen the joint and provide continuous prestressing throughout length of bridge.

**Grout**

Use of the epoxy-resin grout in the pontoon connections involved special problems. To insure the bond between the grout and concrete, it was essential
that the concrete surfaces be as clean as possible. To clean the abutting surfaces of the pontoons, a steel cofferdam was installed which provided a dry work area approximately 6 ft wide between the adjacent pontoons. The exterior surfaces of the pontoon bulkheads were then sandblasted to remove all foreign matter. The cofferdam was then removed and the shear keys and bolts engaged to provide a water-tight seal between the pontoons. Fresh water was then used to flush the joint surfaces to eliminate salt. To provide better workability of the epoxy grout and to shorten the curing time, heat was applied to pontoon bulkheads and to the sand and epoxy. Heating units were installed in the small opening between the pontoons and the concrete heated to a temperature of approximately 130°F. When the desired temperature had been reached, the heated sand and epoxy were mixed in a ratio of two parts sand to one part epoxy and the mixture was placed in the joint. Two stiffening girders were used for on-site connections to limit movement in the joint while the connection was being made.

**PERFORMANCE**

The bridge was opened to traffic on Aug. 12, 1961. During the first year, the bridge was subjected to winter storms of near record intensity and has satisfactorily withstood the forces engendered by these storms and has demonstrated adequate structural stability.

The accentuating factor contributing to vulnerability of exposed areas is the close proximity to the water level of most equipment contained on the bridge. During periods of heavy storms it is not uncommon for seas to break over the lower deck parapet and flood the entire surface of the top slab of the pontoons. As can be readily seen, this condition gives extreme exposure to all areas of concrete and metal.

Due to this marine exposure it has been found necessary to take special precautions in the protection of the electrical components of the drawspan system to insure against moisture inter-
preheating pontoon connecting joints prior to placing epoxy grout

Fig. 9—Preheating pontoon connecting joints prior to placing epoxy grout

ferences. Constant routine attendance has been given to the equipment's electrical heaters; the condition of the limit switches, relays, and rectifiers; and the continual checking of the movable span control system. It has been found that a normal three coat lead paint system on steel has definite limitations to this type of exposure and this method of metal preservation is giving way to epoxy coatings, and more protective methods of encasement in concrete or other materials where conditions will permit.

The concrete surfaces thus far [1962] have shown no ill effects from marine exposure and no special protective treatment has been necessary. Encouragingly enough, marine growth on the immersed areas of the pontoons has not been the source of any concern, and based on the amount of growth on some pontoons which have been afloat for over 3 years, it seems unlikely that marine growth allowances will ever be exceeded.

Periodic checks of the tension in the anchor cables are made to insure proper bridge alignment and also serve as a check against theoretical values of stress under various conditions of weather, tide, and current. These tension checks are made by 100-ton hydraulic jacks installed at each of the anchor cable cross heads.

The first year of operation clearly demonstrated that bridge structures of the floating type are feasible and practical on inland marine waters. With out doubt the Hood Canal Bridge is the forerunner of other even larger floating bridges destined to be constructed in the Puget Sound region.
CONCRETE PONTOONS FOR MARINAS

By A. Morgan Noble

Concrete has proved to be an excellent material for marina pontoons. The two most important design criteria are stability and durability; the pontoon must be stable in rough water and it must be durable and require a minimum of maintenance. It must also be designed for handling during the construction and assembly of the marina. In addition, a good marina pontoon, under the expected loading, will provide a desirable 15 to 20 in. boat-slip freeboard. The structural design must consider the compressive forces in the pontoon developed by the applied load and the resisting buoyant forces. Tensile forces must be minimized or properly resisted when the deck and pontoons are to be monolithic. Lightweight concrete pontoons with, timber stringers, deck and facing are shown in Fig. 1 and pontoons monolithic with the deck structure are shown in Fig. 2.

TYPES OF CONSTRUCTION

Normal weight concrete

A good product can be produced with normal weight concrete (145 lb per cu ft) if a dense and workable mix is placed in strong, tight forms. However, greater buoyancy is required to support the dead weight than in the case of a lightweight concrete design, larger handling equipment is required, and handling stresses affect the pon-
Lightweight concrete

There are many aggregates used for lightweight concrete (less than 110 lb per cu ft). Earlier designs, first appearing on the Pacific Coast 15 years ago, used perlite aggregates which resulted in concrete weighing less than water, and consequently the pontoon would float even if ruptured. Although the compressive strength of this concrete was less than 1000 psi, the design is still successful, providing it is used correctly. However, after 3 or 4 years of service, it was discovered that sea urchins could wear holes through the sides of the pontoons.

Later designs have used an expanded shale aggregate to produce concrete weighing about 100 lb per cu ft with strength greater than 3000 psi. Such a design was used in Shilshole Basin by the Port of Seattle.

Lightweight concrete pontoon construction requires a contractor experienced in mixing and handling this type of concrete. In areas where experienced lightweight concrete manufacturers are located, its use is prevalent and acceptable. Where no such experience is available, other materials are more in demand.

Concrete coated pontoons

Recently, an expanded polystyrene pontoon coated with 3/8 in. of concrete trowled in place has come into use. The coating adds to the dead weight, but the concrete protects the polystyrene from petroleum products floating in the water. This type pontoon has the appearance of a concrete product, but floats much higher. Concrete pontoons have also been built with shotcrete.
CONCRETE MARINA PONTOONS

COMPARISON OF TYPES

Stability

The heavier the pontoon the more stable it will be, providing the center of gravity is kept low. However, much of the stability of the boat slip can be derived from the deck structure. Construction of a diaphragm with a continuous sheet deck or use of a diagonal wood deck can greatly help stability. One patented pontoon uses a concrete monolithic deck and buoyancy box which results in a stable platform (Fig. 2). However, this introduces stresses in the concrete from wave action which requires special design attention to transfer these stresses without cracking the concrete.

Durability

Concrete borers of the pholad family, if present in marina waters, can damage concrete pontoons unless the concrete contains a hard-shelled, well-dispersed aggregate. Ordinary rock or expanded shale aggregate has resisted attack, but perlite aggregate, shotcrete without a coarse aggregate, and plaster concrete coated pontoons can be bored by the pholad. It has usually taken 3 to 4 years for this to occur. In the colder waters north of Monterey Bay, this concrete borer has not been found to be active.

Handling

Structural design for handling and the techniques of handling pontoons must be considered. Poor handling can ruin a good pontoon. In the plant, overhead cranes, dollies, and fork lifts with spreader bars and cribbing are used to distribute the stresses in moving the pontoons from casting beds to trucks for hauling. Trucks with jib

Fig. 2—Lightweight concrete pontoons with monolithic concrete deck and box, and timber facers bolted to inserts in the concrete
Crane mounted on the truck bed are handy for loading and unloading the pontoons into the water (Fig. 3). If the shore slopes into the water, a sliding way can be erected for launching the pontoons but care must be exercised to see that the pontoons do not come into contact with other structures or each other during launching. After they are in the water, the pontoons must be secured in place immediately to prevent any solid contact against the thin walls (Fig. 4).

**TABLE 1—COMPARISON OF TYPES OF CONCRETE PONTOONS**

<table>
<thead>
<tr>
<th>Type of pontoon</th>
<th>Stability</th>
<th>Buoyancy</th>
<th>Original cost</th>
<th>Maintenance cost</th>
<th>Durability</th>
<th>Handling</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal weight concrete</td>
<td>1</td>
<td>4</td>
<td>2</td>
<td>3</td>
<td>1</td>
<td>3</td>
</tr>
<tr>
<td>Lightweight concrete</td>
<td>3</td>
<td>2</td>
<td>1</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Concrete coated</td>
<td>4</td>
<td>1</td>
<td>4</td>
<td>1</td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td>Shotcrete</td>
<td>2</td>
<td>3</td>
<td>3</td>
<td>4</td>
<td>3</td>
<td>4</td>
</tr>
</tbody>
</table>
Comparison
With the above considerations as a guide, each type of pontoon described can be graded according to a number of important considerations as in Table 1. Such a simplified list overlooks many factors peculiar to a given situation which should be considered. However, the basic list of Table 1 indicates that, with all considerations being given equal weight in selection, the order of preference should be: (1) lightweight concrete, (2) normal weight concrete, (3) concrete coated, and (4) shotcrete pontoons.

CONCRETE PONTOON DESIGN

Usually the pontoons are cast in size modules determined by production forms and their spacing and arrangement are varied to satisfy the flotation requirements. However, pontoons are deepened for special buoyancy requirements under gangways or a storage location.

The usual sizes found in marinas are 3 x 6, 4 x 6, 3 x 8, 4 x 8, and 5 x 8, 6 x 8, ft. Depths vary between 14 and 30 in., the deeper ones being used with heavier aggregate concrete or where the decks of the float are concrete, to offset the heavier displacement from the greater pontoon dead weight. Usually, the thickness of concrete varies between 1\(\frac{1}{2}\) to 2 in. It is important to provide a dense, workable, waterproof concrete. During concrete placement, with such thin walls, it is important to keep the reinforcing mesh in place at the center of the concrete section. This is difficult, and for that reason some man-
ufacturers do not use wire mesh in the box, as it leads to concrete deterioration if sea water reaches the metal. This can be done as long as spans between supports are not longer than 4 ft, and the pontoon is only subjected to compressive stresses from flotation, and deck stringers are directly over the pontoon walls. The tops are reinforced because they are subjected to live loads during installation. To provide necessary support, a full depth center bulkhead is cast monolithic with the box across the width of the pontoon (Fig. 8).

A pump hole, usually about 2 in. in diameter, is left in the top on each side of the center bulkhead. It is best to use a neoprene tapered plug to fill the hole as it is easy to remove when pumping or filling of the pontoon is desired. Metal plugs tend to lock in place.

Two design approaches are used for concrete pontoons. In early designs, the pontoons supported a wood deck and they were spaced 8 to 12 ft apart. The pontoons supported 2 x 8, 2 x 10, or 4 x 6-in. stringers. They were held in place with cribbing and by the buoyant force of the pontoons.

Recently, a design was developed which uses the top of the pontoon as the deck of the float. The pontoons...
are not spaced but are bolted together with timber facers attached to each side of the pontoon by inserts cast in the concrete (Fig. 2 and 5). The inserts transfer the beam stress from the facers to the concrete. This arrangement results in diagonal tension stresses in the concrete. To distribute these stresses deeper into the concrete to prevent cracking, spacers are used between the concrete surface and the inserts which are welded to a continuous reinforcing bar around the perimeter of the deck (Fig. 5). An attempt has been made to eliminate these tensile stresses by eliminating the bolts and facers. Instead, post-tensioned cables through each pontoon hold them together for assembling into a line of float. This method would seem to be a practical approach, but has not been tested sufficiently. If the pontoons are located in rough water, it is better to fasten them together with galvanized rods from facer to facer, between the pontoons.

**METHODS OF MANUFACTURE**

The primary methods of manufacture are (1) casting a monolithic box and top, and (2) casting the top and box separately and later joining them (Fig. 9-10).

Originally, portland cement grout was used to bond the top to the box. Galvanized wire mesh, extending across the joint between the box and the top, reinforced the joint. With the development of epoxy cement, the tops are now glued in place. The joint is strong and waterproof and requires no reinforcement across the joint.

To place a monolithic pontoon, a waterproofed, reinforced cardboard box has been used as an interior form which was left in place. The tendency now is to omit the cardboard box and use a solid block of expanded poly-
styrene for the inner form. When the tops are cast separately, they are cast on a slab and frequently glued in place before the concrete attains its final set. The top then tends to conform to any slight deviations in the height of the box sides. However, this is only done when the pontoon will be put in use within a few days after obtaining full strength, or where the climate is not hot. The tops are normally left off to prevent large differentials in temperature between the inside and outside of the pontoon which cause cracks to form in the concrete.

**Important considerations**

The following procedures can make the difference between success and failure of concrete pontoon service. Other factors play a role, but these are fundamental.

1. Aggregate segregation must be prevented by careful control of mixing water and vibration.
2. Walls of the pontoon should be spaded to eliminate air pockets.
3. The wire reinforcement should be carefully centered and its position must be maintained throughout placement.
4. Chamfer all angles to distribute the stress in the concrete.
5. If a monolithic placement is used, care should be taken to insure that the inside form stays in place, otherwise the concrete walls will vary in thickness.
6. Proper curing of the concrete is mandatory. For volume production steam curing should be used.
7. Avoid impact or point loading during handling. Design of the boat slip should include a facer or rail to prevent impact. The design should place the concrete in compression, and avoid tension.