Report of the FIP Commission on

Prefabrication*

SYNOPSIS

This report is a compilation of fourteen separate sections on items of interest. They are: stressing together of precast units—elimination of mortar or cast concrete joints; self-stressing concrete; extrusion and vibro-stamping—principles and limitations; the electrical heating of high-tensile bars applied to the development of mass production techniques; large units used under water—principles and methods of jointing only; significance of early tension cracks in the compression zone (including the effect of transporting); storage of precast elements, with a note of precautions necessary owing to relaxation, creep and deformation generally; columns—advantages of prestressing; a résumé of existing knowledge on shear at the interface in composite construction; precast piles over 30 m (98 ft.) long; an advisory note on the sudden release of pre-tensioned reinforcement; precasting of prestressed bridge beams by casting concrete in forms vibrated from below; poles for power lines; accelerated hardening. The report has been edited, and some portions deleted, to make it more suitable for PCI JOURNAL presentation..

INTRODUCTION

The Commission on Prefabrication considered that the best way that they could serve those concerned with precasting was to present reports and notes on items of interest on which information did not appear to be readily available.

The Commission is continuing its studies of items which appear of sufficient interest and in particular of:

(1) Stressing together of precast units—elimination of mortar or cast concrete joints;
(2) Columns—advantages of prestressing;
(3) The electrical heating of high-tensile bars applied to the development of mass production techniques;
(4) Self-stressing;
(5) Accelerated hardening of concrete—temperature and stresses.

Suggestions as to other items, the holding of symposia and ways in which the Commission's useful service could be increased would be much appreciated.

STRESSING TOGETHER OF PRECAST UNITS—ELIMINATION OF MORTAR OR CAST CONCRETE JOINTS

In an endeavour to avoid the use of concrete or mortar in joints between precast units stressed together, "dry" joints and "glued" joints have been employed.

Non-bonded joints

The use of "dry" joints without a

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filling requires the two faces of the joint to fit together perfectly. The edges of the joint faces are chamfered in order to prevent local spalling. Elements are cast against each matching unit, so as to provide perfect fit of adjoining faces.

Another method is applied by a British company which manufactures large-diameter piles from relatively short units (e.g. cylindrical units 53 cm (21 in.) in diameter and 3 to 5 m (10 to 16 ft.) long) assembled together by means of prestressing. The units are cast vertically in steel moulds, and have accurately formed ends which bear directly on neoprene sheets in the joints.

Adhesive-bonded joints

In this system a layer of adhesive ensures sound fitting of consecutive units. The strength of the joints can be greater than that of the concrete in tension, in compression and in shear, and the glue ensures watertightness.

Epoxy resins of petrochemical origin are preferred. They may contain a mineral powder filler (cement has been used in the USSR) and can be mixed with sand for use in thick joints.

In general it is considered that, for good adhesion, the surfaces should be well cleaned and not too damp. Tests carried out with epoxy resins by the Centre Expérimental du Bâtiment et des Travaux Publics have shown the tensile strength of an adhesive-bonded joint to be higher than that of the hardened concrete in 80% of the cases when the joint faces were dry, and in 30% when the faces were damp. The adhesion of polyester resins is also weaker on damp concrete.

The surfaces of the joint must be given very careful preparation. Tests show that sandblasting and brushing in conjunction with the use of compressed air give good results on site, and that grease must be avoided on the surfaces to be bonded.

When a "pure" adhesive is employed, the thickness of the joint is kept down to a few tenths of a millimeter for economy and to reduce deformation of the joint. The joints can therefore be prestressed before the adhesive has set, causing the adhesive to ooze out at the edges.

Thicker joints, some millimetres wide, are used when the adhesive contains a filler. Franz advises that such joints should be 1 to 2 cm (0.4 to 0.8 in.) thick because, in his opinion, it is more expensive to form precision joint faces which will fit together than to make a thick joint. In the USSR, a compound of epoxy resin and cement was used for joints 0.2 to 0.8 mm (8 to 32 mils) thick.

The mating surfaces must fit together with greater precision when the joint is thin. To ensure a good fit, the precast units which are subsequently to be joined together are often cast one against the other. Sometimes the mating surfaces are ground flat, but this may reduce adhesion.

When the precast units are assembled together by prestressing before the adhesive has set, it is necessary to prevent relative sliding of the units, for which purpose only it is expedient to provide projections on one unit fitting into recesses on the next, so as to key them together. In the USSR such recesses in the joint are designed to resist working shear forces.

Recent examples of execution

In southern and eastern parts of the USA, post-tensioned piles up to 160 ft. (49 m) long and 54 in. (137 cm) in diameter have been used;

these have been made up of centrifugally spun 16 ft. lengths, prestressed together with epoxy joints. Mention must be made of concrete piles constructed of units assembled with the aid of an Epikote resin-based adhesive compound. Adhesive is placed in a collar fitted to the top of the pile installed in position. Next, the pile extension unit is lowered on to the adhesive, which is then steam-heated for 20 min., after which the joint is cooled with water for 10 min. Pile driving can then be continued. This method was employed in the construction of the Cultural Centre at Melbourne.

Bridge piers in Malaysia

These piers, with an internal diameter of 70 cm (28 in.) and an external diameter of 90 cm (35 in.), consist of 3 m (10 ft.) long spun concrete units post-tensioned together. The method of manufacture produced very flat ends, the amount of play being not more than about 1 mm (0.04 in.). The units could thus quite easily be joined by means of epoxy resin-based mortar.

Piers for quay in Indonesia

These are generally comparable to the piers installed in Malaysia but differ in the method of manufacture, the units being 5 m (16 ft.) long and produced by the vacuum process. This affected the jointing operations, since the mould was so designed that it was not possible to obtain satisfactory flatness of the mating surfaces. In this case the surfaces were ground to reduce the amount of play to 3 mm (0.12 in.).

Here again the units were connected by means of an epoxy-based mortar.

Choisy le Roi Bridge over the Seine

The superstructure is formed of four box-girders of constant depth below deck level. It was constructed by cantilevering out from the piers by the addition of precast units 2.50 m (8.2 ft.) long. On the precasting site, the units were concreted on a base having exactly the same shape as the soffit of the bridge superstructure and were cast in contact with each other. To ensure exact location when subsequently assembled, three locating keys were provided, one in the middle of the top slab and the other two in the webs of the box section. The precast units were then taken to the bridge construction site. A telescopic guiding carriage was installed in the preceeding box-girder and the next unit to be assembled was placed on this carriage, which allowed for movement in any direction.

When the prestressing cables had been threaded, the next unit to be assembled was fitted in position and provisionally erected as a trial assembly. If the fit was acceptable, the unit was removed and the joint formed with adhesive. Preliminary tightening of the unit was affected by means of screw-threaded rods and finally the prestressing cables securing the unit were tensioned. The shear across the joint of about 2 to 3 kg/cm^2 (28 to 42 psi) was taken up by the locating keys, which prevented any relative displacement.

SELF-STRESSING CONCRETE

The first significant results in creating expanding admixtures were obtained by the French scientists Lossier and Hendrique at the end of the 1930's.

Further developments by Mikhailov, Litver and Popov (at the Research Institute of Concrete and Reinforced Concrete, Moscow) led to the creation of stressing cement. The expansion of this cement takes place when a strength up to 50 kg/cm^2 (700 psi) has been reached and self-stressing of concrete becomes possible (the stress in the reinforcement can reach 5000 kg/cm^2 (70,000 psi)

Stressing cement is manufactured by grinding together Portland cement, aluminous cement and gypsum. The hardening and self-stressing of reinforced concrete made with stressing cement takes place by curing in water at normal temperature. The type of stressing cement intended for precast factories requires a preliminary curing in water at 20°C (68°F) while the one intended for site use does not.

Since the waterproof properties of the stressed concrete are so high that it is resistant to the penetration of gas and petrol the stressing cement is widely used in the making of pressure pipes, when the longitudinal as well as the coiled reinforcement becomes prestressed.

The first plant to produce self-stressed pressure pipes has now been constructed in the USSR; centrifugal and vibro-extrusion methods will be used there.

A considerable programme of research is in hand in the USSR and includes: a method of slowing up hardening by preliminary partial hydration of the cement; a study of the distribution of prestress; and the control of the final prestress.

Since 1957, research on expanding cements and self-stressing concrete has been carried out in the USA at the University of California. The American expanding cement, developed by Klein and Troxell, was a mechanical mixture (without combined grinding) of Portland cement, special sulpho-aluminate cement and slag, but the slag is not now used.

Some interesting investigations with the American expanding cement, on slabs, shells and piles have been carried out by Lin. In 1963 investigations were started in the Research and Development Laboratories of the Portland Cement Association in Illinois, based on the components Portland cement, aluminous cement and gypsum, proposed by the Research Institute of Concrete and Reinforced Concrete, USSR.

In contrast to the Soviet method, the Americans prepared the stressing cement by ordinary intermixing of the components instead of by combined grinding; tests confirmed the results of the Soviet investigations on the properties of the stressing cement and further investigations are in hand.

Part of a highway pavement was constructed in self-stressing concrete in Hamden, Connecticut in 1963. In this case, measurements indicated that a steel stress of 5000 kg/cm^2 (70,000 psi) was reached. Experiments on filling of joints with stressing cement were also carried out in the USA by Palmdel and Lody.

Dzhun Way of the Peking Research Institute of Building Materials has carried out some work on pressure pipes, using expanding cement obtained from the USSR.

EXTRUSION AND VIBRO-STAMPING—PRINCIPLES AND LIMITATIONS

These are techniques whereby pressure and vibration are applied simultaneously to concrete. Whilst normal vibration methods are adequate for most types of precast concrete product, there are limitations on the efficient placing of high-strength concrete in intricate sections. It has been found that with vibro-stamping techniques, it is possible to use a mix with zero slump efficiently, even with very complicated shapes. Under vibration, the stiffest mixes become fluid and the applied pressure induces flow, thus ensuring that the mould is completely filled. The frequency of vibration and intensity of pressure may be varied to suit the workability of the mix.

The raising of the vibro-stamper is aided by introducing compressed air under its bottom face to speed the breaking of the bond with the concrete and to eliminate damage to the surface of the unit.

Vibro-stamping had been widely used in the USSR in the manufacture of staircases, tunnel linings, ribbed floor and roof slabs, channels and I-beams; it has been used effectively in conjunction with pretensioning and post-tensioning techniques. However, owing to certain difficulties, the finish is often poor and its use is not recommended for the manufacture of concrete units where a high-quality surface is desirable.

Extrusion

The technique of extruding concrete is similar in many respects to that of vibro-stamping and involves the forcing of a plastic mix under pressure through a pre-formed die, and the use of a zero-slump mix under combined pressure and vibration permits extrusion into complicated sections, with the concrete having sufficient residual density to retain accurately its formed shape after the passing of a movable form. Since the extrusion process does not normally suffer from the disadvantage of the poor finish experienced with vibro-stamping, it can be used with benefit in the manufacture of highly repetitive units which require a very good surface finish.

Mono-purpose equipment

This type of equipment requires separate extrusion units for each depth or width of member produced. A novel feature of the equipment is that extrusion techniques are employed in their true form in that the machine develops enough force in the moulding chamber to extrude the concrete sections and force the machine along its casting bed. Extrusion is achieved by augers which carry out the dual purpose of extrusion and duct forming. It is claimed that this method of internal pressure eliminates surface tension and results in smooth top and bottom surfaces.

THE ELECTRICAL HEATING OF HIGH TENSILE BARS APPLIED TO THE DEVELOPMENT OF MASS PRODUCTION TECHNIQUES

The USSR has developed electrothermal pre-tensioning of deformed bars for building units up to 24 m (79 ft.) long. The reinforcing bars are heated by an electric current and transferred to their positions in the units when the current has been switched off. The end supports prevent shortening, and so when the bars cool they are automatically pretensioned.

The following items are new developments of the method:

(1) Tensioned bent-up reinforcement is now used. A straight reinforcing bar is placed in the upper supports and heated electrically so that it sags and can then be secured to special anchorages lower in the forms. During cooling the reinforcement is stressed uniformly in its sloping and straight portions.

(2) In order to stress more closely in accordance with the bending moment diagram and relieve anchorage forces at the ends, bars can be held along their central portions only, so that when they

are stressed by cooling, their end portions remain unstressed.

(3) Anchorage and jointing of bars is achieved by pressing steel muff pieces firmly into the deformation on the bars. This is a great improvement on the older method of forming anchor caps by heating or welding and does not reduce the strength of the bar.

Special release devices have been developed both for the end and intermediate anchorages.

Pretensioning by the use of a winding machine has been described at previous Congresses. Electrical heating of the wire or strand reinforcement greatly reduces the mechanical effort required for winding.

SIGNIFICANCE OF EARLY CRACKS IN THE COMPRESSION ZONE (INCLUDING THE EFFECT OF TRANSPORTING)

During the handling and transporting of prestressed precast units, it is economically advantageous to allow part of the unit to cantilever beyond a lifting point or temporary support; this increases the tensile stresses at the top and the compressive stresses at the bottom of the prestressed section.

The allowable length of a cantilevering portion will therefore be limited by either of these stresses, but tensile cracks may sometimes be tolerated. A large negative moment capacity can be achieved by keeping the tensile stresses due to prestress at a low value. This may be done in several ways, such as by deflecting strands, by neutralizing the bond at the ends or by using prestressed top reinforcement.

The allowing of cracks, however, makes high negative moment capacity possible without these means, provided that the allowable compressive stress is not exceeded in the bottom fibres of the section.

Cracks in the compressive zone can have disadvantages such as lower resistance to corrosion of steel, excessive upward or differential camber and loss in positive cracking moment capacity.

By using an appropriate design, early cracks in the compressive zone will be of only small width, governed by the amount, distribution and stress of the reinforcement covering the cracks. This reinforcement is normally non-stressed and not more sensitive to corrosion than a parallel example in normal reinforced concrete. The cracks will close fully or partially when the design load is applied and, except for extremely corrosive environments, the problem of corrosion is not significant.

The loss in stiffness with regard to negative moments may cause excessive upward or differential camber during storage, before the design loading can be applied, and these effects are then only partly reversible. Units such as T-shaped slabs, where constant camber is important, and elements which take negative moments and are designed for composite construction, should therefore not be allowed to crack. For elements such as long beams, upward camber may not be a sufficient reason to prohibit cracking in the compressive zone.

Russian investigators have noted a drop in positive cracking moment capacity of prestressed beams having flexural cracks caused by negative moments, deeper cracks giving rise to larger drops. Accordingly, the Russian specifications require a reduction of the positive moment capacity by 10%, and of the rigidity by 15% when the compressive zone is assumed to crack.

The development of cracks will cause a redistribution of the compressive stresses at the sections involved, a fact that may affect creep losses. When cracks close on loading, the stress distribution will be somewhat different from the original and this may influence the positive moment cracking capacity. As the Russian investigation refers to small beams of a specific type, it is felt that there is a need for further study of this problem with particular attention to the magnitude of compressive stresses, creep and shrinkage.

Although the effect of early tension in the compressive zone on the positive cracking moment capacity is not well understood, early cracks are recognized in some countries.

In the USSR early cracks are permitted except for structures underwater or exposed to corrosive environments or fatigue.

Although not specified in any code, early cracking has been recognized in Sweden for some 20 years for large pretensioned beams with the exception of those used in bridges. Cracking zones have been adequately covered with normal reinforcement and the safety factor with regard to the positive cracking moment has been kept at 1.2, the flexural tensile strength being assumed to be 0.1 of the nominal cube strength.

The PCI Code of the USA does not state any limit for flexural tensile stresses in the case of auxiliary reinforcement in the zone subjected to tension, and specified values may thus be exceeded when not detrimental to proper structural behaviour.

The British Standard Code of Practice allows the specified tensile stresses to be exceeded during handling and construction for short periods not exceeding 48 hr., provided that the engineer is satisfied that the increase will not lead to permanent damage or cracking of the concrete or greater losses of prestress than had been foreseen.

According to the German specification, DIN 4227, certain flexural tensile stresses should not be exceeded during handling. Design according to this specification thus gives a small probability of cracks in the compressive zone.

French specifications by ASP require tensile stresses during the handling phases to remain lower than the effective tensile strength at transfer.

STORAGE OF PRECAST ELEMENTS, WITH A NOTE OF PRECAUTIONS NECESSARY OWING TO RELAXATION, CREEP AND DEFORMATION GENERALLY

If deformation due to shrinkage and creep is to be a minimum, it is important that the concrete is well matured before precast units are erected. Similarly, prestressing should be carried out as far as possible in advance of erection. To obtain the maximum effect from storage the following rules are suggested:

(1) During the important time immediately after casting, the concrete must be kept wet to ensure proper hydration. The equivalent of about 4 days at 15° C should be considered a minimum.

(2) For a cold winter climate, dry-curing should be made indoors for a longer time than for a more temperate climate, otherwise very little shrinkage and creep will be developed before erection.

(3) The store for dry-curing must be covered by a roof and also have side protection as necessary against the weather. The capacity of concrete for absorbing water directly is great, but the inter-

change of moisture between concrete and air through absorption or diffusion is slow. Storing in this way will also protect the units from warping and curvature due to unilateral sunshine.

Columns, piles and poles are often stored in several layers. The points of support should be chosen so that there is a minimum of deflexion due to bending. When more than two supports are needed to prevent curvature, then accurately placed, unyielding supports are essential.

Short beams are often stored in several layers and in this respect should be supported as for columns. Their own weight has little influence on stresses during storage and the beams easily obtain camber due to creep. If this camber is to be constant it is essential that the beams should be given the same general curing conditions and that they should be sheltered from sunshine. If the bottom layer is heavily loaded at the ends, horizontal friction forces at the supports could influence the development of camber.

Long beams with small lateral stiffness should be propped or restrained at the centre of the span and at both supports to avoid warping and lateral curvature.

Slabs are very susceptible to variations of the value and position of the prestressing force, of the concrete quality and the general curing conditions, and to yielding of supports. If slabs are stored with an overhang, for which prestressing is not specially arranged, the creep deformations appear to be much greater than would be given by normal calculations, and it is essential that this method of storing should be avoided.

Owing to flaws in the manufacturing process or in storing, undue deformations such as excessive camber, lateral curvature or warping may sometimes occur. By using the effect of irreversible creep from suitable loading of the units, it is possible to make corrections. Corrective measures are usually expensive to apply and, even if perfect when completed, the result is not lasting. Such measures should therefore only be used in isolated cases when there is a possibility of controlling further movement after erection.

COLUMNS—ADVANTAGES OF PRESTRESSING

It is often held that there are advantages to be gained from the prestressing of long columns, particularly as regards their handling and erection. Research and development work carried out by a number of engineers has produced figures which give support to this view.

Ozell and Jernigan have tested 47 columns, of which 41 were pretensioned, with a 20 cm (8 in.) square or 15 cm (6 in.) square section and a length/depth ratio from 10 to 32. The load was, except for one case, applied axially and the proportion of pretensioned reinforcement (0.53 to 2.5%) and use of stirrups were varied. The test results led the investigators to the following conclusions:

(1) Stirrups should be used throughout.

(2) The optimum prestress with regard to the ultimate column strength was about 0.25 of the ultimate cylinder strength, for length/depth ratios between 20 and 30.

(3) For length/depth ratios less than 20, axially loaded prestressed columns had lower ultimate strengths than the reinforced columns, whereas ultimate strengths were almost equal for ratios above 30.

Zung-Teh Zia has approached the problem of the prestressed column theoretically, and found that his analysis compares favourably with the experimental results of Ozell and Jernigan. He concludes that prestressing penalizes the strength for length/depth ratios of less than 25 and that optimum prestressing for length/depth ratios between 25 and 35 corresponds to a ratio of prestressing reinforcement of roughly 1%, using strand reinforcement and concrete with a compressive strength of 6000 lb./in.^{2} (430 kg/cm^2).

Hall reports a series of tests on 165 cm (65 in.) long columns, 5 × 7.5 cm (2 × 3 in.) in section and reinforced with four 5 mm (0.197 in.) diameter wires, with varying pretension. The columns were loaded eccentrically, the eccentricity varying from 1/20 to twice the depth of the columns. Hall concludes that prestressing reduces load-carrying capacity for axial loading, but for eccentricities exceeding 1/10 of the column depth, prestressing gives a marked increase. For very large eccentricities, flexure and not buckling dominates the failure. Since eccentricities less than 1/10 of the depth of the section are rarely encountered in practice, the results of the tests tend to the conclusion that prestressing is generally advantageous for slender columns.

Lorentsen has conducted an investigation on pretensioned and reinforced columns with a 20 cm (8 in.) square section and length/depth ratios varying between 15 and 39. The columns were eccentrically loaded, the eccentricity being 1/300 of the column length. Results indicate that prestressing is favourable for length/depth ratios exceeding 27.

Lin and Itaya deduce theoretically that stresses and deflexions may be calculated with reasonable accuracy according to the elastic theory as long as the column remains uncracked. Above the cracking load, the plastic behaviour of concrete as well as of steel must be taken into account within the cracked region.

More research is undoubtedly necessary to clarify the influence of slenderness, the eccentricity of load and amount of prestress on the ultimate load of columns. An important factor is the influence of creep on stability but no work on prestressed columns has yet been done on this. It is hoped that further investigations will clarify to what extent prestress is beneficial when creep is taken into account. There is undoubtedly a considerable field in which prestressing can add stability to structures containing slender columns, and further research and development on this matter should be well worth while.

A RESUME OF EXISTING KNOWLEDGE ON SHEAR AT INTERFACE IN COMPOSITE CONSTRUCTION

The methods normally employed to improve bond are as follows:

(1) Roughening of the surface of the precast concrete by cross-tamping, water-jetting, hacking or other means, but having first ensured that there is full consolidation up to the surface.

(2) The use of some form of castellation as an alternative to roughening.

(3) The use of resin-bonded glues to promote bond has been advocated but not widely used.

(4) Connexion by bars protruding from the precast work, in such a manner that they can be incorporated in the work cast in situ.

More than one method may be employed at the same time.

Experimental work

Experimental work has shown that where two methods of bond improvement are used together, the shear strength developed at the interface is less than the sum of the two when used separately.

Hanson, who has carried out numerous tests, considers that the structure can no longer be considered monolithic when the displacement at the interface exceeds 127 microns (5 mils). For castellations 6.3 cm (2.5 in.) high and 12.5 cm (4.9 in.) long with smooth unbonded surfaces and a concrete of cylinder strength 350 kg/cm^2 (5000 psi) for precast concrete and 210 kg/cm^2 (3000 psi) for in situ concrete, he found a limiting shear stress of 25 kg/cm^2 (350 psi), which is about half the value that would be calculated by considering the criterion to be a failure in compression on the vertical faces.

The Russian Standard Specification, SNIP II—B.1-62, gives recommendations for the design of castellations.

Hanson found that for similar concrete the shear strength developed by a rough surface was 35 kg/cm^2 (490 psi) which accords well with C & CA findings.

Further tests on rough surfaces, with adhesion eliminated and separation of the surfaces prevented by stirrups, gave a shear strength of 17 kg/cm^2 (240 psi) after the direct stirrup effect had been deducted. A smooth surface gave 13 kg/cm^2 (180 psi) at the first slip and 7 kg/cm^2 (98 psi) thereafter.

Hanson found that there was an increase in shear strength of 12.25 kg/cm^2 (170 psi) for each additional 1% of reinforcement passing across the interface, which is broadly confirmed by C & CA.

Other tests have been made by Zelger and Dashner on beams with a smooth horizontal joint and on small elements with rough or smoother interfaces. The results, lower than Hanson's, are:

(1) For small elements, smooth surface without stirrups	7-13 kg/cm^2 (98-182 psi)
(2) For small elements smooth surface with stirrups (0.9% of the interface)	13-19 kg/cm^2 (182-266 psi)
(3) For small elements, rough surface without stirrups	8-19 kg/cm^2 (112-266 psi)
(4) For small elements rough surface with stirrups (0.9% of the interface)	10-14 kg/cm^2 (140-196 psi)

Tests on beams have shown that a vertical pressure increases shear resistance up to 20 or 35 kg/cm^2 (280 or 490 psi).

There is obviously a case for improving adhesion by the use of glues. Here mention may be made of a composite bridge comprising a concrete slab on steel girders with bond improved by gluing.

Most of the composite prestressed concrete bridges constructed to date have very low stresses at the interface and the projecting bars and transverse prestress are easily able to ensure resistance to sliding at these faces.

PRECAST PILES OVER 30 M (100 FT.) LONG

For piles up to 30 m (100 ft.), the phenomenon of reversed tensile stresses can normally be overcome by reducing the drop during early driving and increasing the weight of the hammer, and information is available within this range. Beyond the range, little is known and it is obviously impossible to increase the hammer weight indefinitely. More attention must therefore be paid, in the light of known ground conditions, to the value of the prestress, the height of the drop and the head packing. Computer programs are available to give the stresses set up under defined driving and ground conditions but have not been widely used. Field research under favourable conditions to give comparisons with computer results could perhaps lead to the confident prediction of performance for long piles.

In the meantime there seems to be an immediate need for any details available of long piles already driven, and the notes given in Table I are therefore presented.

ADVISORY NOTE ON THE SUDDEN RELEASE OF PRETENSIONED REINFORCEMENT

It is generally accepted that release of prestress by screws is impracticable and therefore direct cutting of the wires must be resorted to if jacks cannot be used.

A number of firms now release by cutting the tendons rather than by jacking. Sometimes this is done to a carefully worked-out pattern and by heating a 6-9 in. (15-23 cm) length of tendon so that the break is not sudden. In many cases, however, the cutting is sudden and many thousands of building units in service have been released in this way, but some disturbance to the bond must have occurred and this should have been considered in the design.

An article on the influence of concrete strength on strand transfer length describes some experiments which indicate that sudden release of strand increases the transfer lengths from between 20% and 30% and for strand over 1.25 cm (0.5 in.) diameter can destroy bond length altogether for up to 30 cm (12 in.) from the end, and this could have an appreciable effect on shear strength.

It should be recognized that the sudden release of any tendon decreases bond and if this method of release is to be used in the factory it must be known to the designer and its effect taken into account in his calculations. For some designs he will have to ban the method altogether.

In all cases, the exact procedure to be used as regards the pattern and manner of flamecutting must be carefully worked out and agreed by the parties concerned. It is most important that the operation should be continuously supervised by a responsible and experienced man.

PRECASTING OF PRESTRESSED BRIDGE BEAMS BY CASTING CONCRETE IN FORMS VIBRATED FROM BELOW

This is a useful method which is not widely employed except by the French. The process is derived from techniques of casting on vibrating tables and its main use has been for double-T beams for bridge work.

Steel forms are used, carried on elastic blocks set in concrete supports. Vibration is applied through power vibrators (7 kw at 50 c/s) attached to the form bottoms. The vibrators give rise to considerable centrifugal forces (up to 8000 kg (8.8 tons) which impart vibration waves of large amplitude to the forms.

The workability of the concrete is such that it does not flow easily under this vibration, but forms a

TABLE I

Designation	Size	Length (ft.)	Hammer	Hammer/pile ratio	Prestress (psi)	Remarks
Wells-Fargo Building, San Francisco	18 in. square	100-138	McKiernan Terry S–14	0.4–0.3	750	Piles placed in pre-drilled holes, and only driven for last 4-8 ft. into bedrock. Steel pointed.
Ala Moana Building, Honolulu	18 in. octagonal	170	Vulcan 0.10	0.54–0.18	900	In 3 sections, with prestress through joints. Long, hard driving through alternating layers of sand, coral and clay.
Martinez-Benicia Bridge, California	20 in. square (11 in. diameter core)	126	McKiernan Terry A–14	0.3	737	Easy driving at 10-20 blows/ft. through silt to decomposed sandstone where blows per ft. increased to 50-100.
Oil Terminal, Slagen, Norway	22 in. octagonal (12 in. diameter core)	115	6.5 tons	0.5		Some splicing was done, with extension pieces of normal pile section. Piles were steel pointed for driving into rock. Some piles raked 1 in 3.5.
Lake Maracaibo Bridge, Venezuela	36 in. diameter, wall thickness 5 in.	up to 215	Steam hammer with rams up to 40,000 lb.		800 (pre-tensioned)	
Port of Baton Rouge, Louisiana	36 in. diameter and 54 in. diameter, wall thickness 5 in.	160	McKiernan Terry S–16		800	
Wharves in Lisbon	29 in. octagonal (18 in. diameter core)	125	10 tons	0.43	700	Some splicing was done.
Marine Works in Fiji	30 in. diameter (21 in. diameter core)	up to 140	12 tons	0.5	750	Final sets 2 in. per blow, in silty clay which tightened up considerably after completion of driving.
Ilikai Building, Honolulu	16½ in. octagonal	120	McKiernan Terry S–10		1,000	In 2 sections, with no prestress through joints.
Marine Terminal, Milford Haven	27.5 in. diameter, wall thickness 3¼ in. at center and 4½ in. at ends	up to 145	B.S.P. 10 tons semi-automatic steam	0.55	1,200	
Bridge over Napa River, Vallejo, California	54 in. diameter (44 in. diameter core)	up to 123	Vulcan 200C	0.35	844	Water-jetted. Driven for last 20 ft. only. Monolithic units made on long-line method.
Bridge over Napa River, Vallejo California	24 in. square	up to 127	Vulcan 200C	0.55	760	
Wharf in San Francisco	18 in. octagonal	up to 155 vertical	Vulcan 140C	0.38	800	
Wharf in San Francisco	20 in. square	up to 138 raking 2 in 5	Vulcan 140C	0.25	100	

slope at an angle of about 45°. Placing is begun at one end of the form, the slope being progressively fed until the other end is reached. The vibrators are gradually moved forward so that they are generally beneath the area of deposition of the concrete.

An elaborate system of trackways and easy-release and re-clamping devices have been worked out for changing the position of the vibrators.

POLES FOR POWER LINES

For certain mass-produced elements, such as poles for power lines, tests are used as a basis for the design. Such tests show that the actual resistance of concrete to tensile stresses is very large. Tensile stresses up to 0.15 times the cube strength should be allowed in the design. 80 kg/cm^2 (1100 psi) has been successfully adopted for box-section poles for power lines, in France and North Africa.

Discussion of this report is invited. Please forward your discussion to PCI Headquarters before January 1 to permit publication in the April 1968 issue of the PCI JOURNAL.