Importance of Proper Anchoring

By EDWARD COHEN

Reliability, economy, and simplicity should be the criteria for selecting an anchorage or connection detail. Each anchorage must be of adequate strength to perform its assigned task, must be the proper type for the application, and must be able to provide satisfactory service throughout the life of the structure.

Experience has shown that the critical points of structures are the connections. As a result of the increased use of higher strength materials and the higher stresses with which we now work, more stress per square inch or cubic inch must be transferred between structural elements than ever before. It is at these points that stresses are concentrated and the greatest care in design, detailing and construction is required. Because connections often control the ductility and strength of the over-all structures, it is here that designers and constructors should exercise extra caution in evaluating the loadings caused by volumetric effects, and lateral and vertical forces, in establishing the required strength and in providing adequate ductility.

Examples are presented to point up the difficulties, accomplishments and goals for proper anchoring of and to concrete.

Keywords: anchorage; anchors; bridge anchorages; bridges (structures); buildings; concretes; connections; construction: ductility; fasteners; loads (forces); precast concrete; quality control; reinforced concrete; strength.

■ WHETHER AN ANCHORAGE REQUIRES 207,000 cubic yards of concrete, as for the cables of the Verrazano-Narrows Bridge or a small diameter anchor bolt, reliability, economy, and simplicity are of prime importance. Every anchorage must be of adequate strength to perform its assigned task, must be the proper type for the application, must be capable of required quality installation under field conditions, and must be able to provide satisfactory service throughout the life of the structure.

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Joints and connections require great care in design detail and construction. Joints and connections are regions in which the increasingly higher stresses used with modern higher strength materials are transferred. However, they are often the most difficult portions of the structure to construct because of the congestion of reinforcement and difficulty of placing concrete or because of difficulty of controlling field operations. Yet it is at these points that the designer bears his most difficult burden for, if the full capacity of the member can not be developed at the connection, part of the available structural strength is wasted.

Based on experience and extensive laboratory tests of structural members factors of safety have been reduced. Because of the need to avoid catastrophic errors and the lack of complete control over many of the factors this reduction has been a slow and careful process. Still, within the main lengths of members reductions in factors of safety have been initiated and further reductions are contemplated. The question which must be answered affirmatively is whether the same degree of knowledge and security is applicable to connection design and construction.

JOB OF THE CONNECTION

The ability of reinforced concrete to sustain unanticipated loads and distortions by redistribution of stress has been one of the important factors in maintaining its excellent safety and serviceability record. Many variations of creep, shrinkage, settlement, loading, and strength

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from those assumed in the calculations have occurred without trouble under the umbrella of ductility.

In addition, for structures resisting dynamic loads or distortions such as earthquake or blast, ductility is of equal or greater importance than initial strength. In the case of earthquakes the calculated stresses are often fictitious values based on empirical methods derived from previous experience or on past earthquake records at other sites. In each case there is no reliable guide to the magnitude of future earthquakes or the degree of damage or the degree of catastrophy that can be anticipated under future conditions unless large plastic distortions can be experienced without failure. In the case of blast loading, the plastic energy absorption after yield may be hundreds of times that in the elastic range. Connections and anchorages which can develop the ultimate strength of structures are desirable in all cases and particularly where dynamic loadings are to be resisted or where settlement, volumetric efforts, etc., may require substantial ductility.

Where a mechanical connection is provided between two simple spans, this connection must be able to carry not only the stress indicated by elastic analysis for the superimposed live loads but also the higher stresses which may be induced by creep, shrinkage, temperature, or other volumetric effects. The alternative is to assure sufficient ductility in the connection to permit self relieving strains to control the stress within the capacity of the section.

Although the concept of high ductility may be more important than high strength, the two cannot be separated. If we consider the ideal stress-strain curve we learned about at school, with a sharp yield point and a long flat region of strain at constant stress, the yield stress criteria for design of a bar to bar tensile connection would appear adequate. However, the possible stress-strain curves of the high strength steels which are now in use exhibit gradual yield and a continuously rising stress-strain relationship. It appears, therefore, that in such cases the maximum ductility may be achieved only if the connection can develop the full strength of the bar.

In the burgeoning market of concrete construction the economic pressure to reduce the total costs has yielded many outstanding developments. In the drive to reduce the number of man-hours per cubic yard of concrete and the amount of concrete per spuare foot of building area concrete of 5000 psi is now in common use, and 6000 to 7000 psi concrete does not raise eyebrows. Strengths of reinforcing bars have more than matched these developments in concrete. From 33,000 psi, steel strengths have risen to 40, 50, 60, 75, and occasionally 90,000 psi. Because of the development of techniques and an increased awareness of its potential, precast and prestressed concrete have, in the past 25 years, taken an increasing share of the total concrete construction mar-

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ket. These developments have brought new problems and higher stresses to the critical connection points. To handle these problems, designers, manufacturers and constructors together, have developed new procedures, and adapted some old techniques to get the most out of their members.

ECONOMY AND SIMPLICITY

While the cost differential between providing adequate and inadequate anchorage usually has only a nominal effect on the over-all construction cost, any difference in connection cost becomes a major advantage or disadvantage between bidders. As a result the fabricator and erector are put under considerable pressure to minimize as much as permitted by the engineer. Too often the plans and specifications have left the selection and detailing of connections open, without providing a yardstick to measure the final result and without briefing the bidders as to the design concept and connection requirements for strength, ductility, and dimensional tolerance.

While designers constantly strive to make the connections as simple as possible, other than structural considerations often make simplicity a relative thing. Simplicity of design must begin with an understanding of the task that the connection is meant to perform. Whether large or small, steel or stone, each anchorage must be adequate to its assigned task. It is the definition of the task which is sometimes difficult. Yet, the simplicity of the connection is dependent on the clear understanding of the task.

RELIABILITY

In addition to economy and simplicity one of the important attributes of satisfactory anchorage is reliability. An important aspect of reliability is permanence. To initial adequacy must be added assurance against corrosion, fatigue, and other time dependent effects. Reliability can be achieved in different ways, but the path becomes longer and the pitfalls more numerous as the magnitude of the "fastener" leaps from a suspension bridge anchorage, to a slab insert designed to anchor a rod supporting a light piece of mechanical equipment.

In the case of the suspension bridge anchorage, the concept, the materials, the distortions, the loadings are all thoroughly investigated and engineered to establish a well-defined capacity. Full allowance is made for all possible variations in analysis, material properties, loadings, and construction and time dependent effects.

In the case of mass produced elements, the requirements of uniformity or quality control must be added to the above. This requirement for quality control does not end with the packaging of the components at the

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manufacturing plant. It extends to the actual installation and the workmanship in the field. If the quality or uniformity of field workmanship is reduced, variations in the installed capacities of fasteners increase, and requirements for field testing and inspection increase. The spiraling effect on construction cost and time may make a theoretically sound fastening system impractical if great skill is required for installation.

In some cases difficulties result not from poor workmanship but from lack of proper coordination between trades. For example, lack of mutual understanding of the limits of permissible misalignment of two components to be connected can turn a relatively simple and reliable detail into a nightmare of complexity and doubt. The willing and knowledgeable cooperation of the constructor is the most improtant single factor in buiding a structure that reflects the intent of the plans and specifications. Such mutual understanding is far more effective than rigid inspection during construction, although inspection plays an important part in the construction picture. The designer must not dictate to the fabricator from an ivory tower or from a design office insulated from the practical problems of construction. The designer, with a full knowledge of relative costs and availability and in cooperation with constructors must arrive at an acceptable solution to the connection problem. To gain the cooperation of the builders, pre-bid conferences to explain new details or procedures can be rewarding to both the bidders and the owners.

EXAMPLES

Some examples of anchorage solutions will illustrate the range of problems which may be encountered.

In the case of the Verrazano-Narrows Bridge across the entrance to New York harbor, the anchorage is one of the most critical features. The anchorage was required to develop a maximum tension of 250,000,000 lb exerted through four 36 in. diameter high strength steel cables. In this case mechanical "fasteners" were provided to anchor the cables to over 207,000 cu yd of concrete, which resists the pull by dead weight and friction. This is obviously not an ideal spot for lightweight concrete.

The eyebar chains consist of three tiers of 244 bars each, for a total length of 100 ft. The upper head of the middle tier and the lower head of the upper tier bars have elongated holes to allow for shimming needed in cable adjustments. The general details of the anchorage are shown in Fig. 1-1. Fig. 1-2 shows the supporting frame and eyebar chains under construction.

The supporting frames are of ASTM A 7 steel. Adjustment is provided between sections of the frame erection to permit compensation of settlement motions. The pins connecting the eyebars to each other

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and to the anchor girders and strand shoes are forged steel conforming to ASTM A 237. Shims and wedges are provided at all eyebar support points to permit the bars to be set as accurately as possible. Live load tension plus dead load produces a small eccentricity in the eyebars but a far larger eccentricity is caused by unavoidable fabrication and erection inaccuracies and by anchorage settlements.

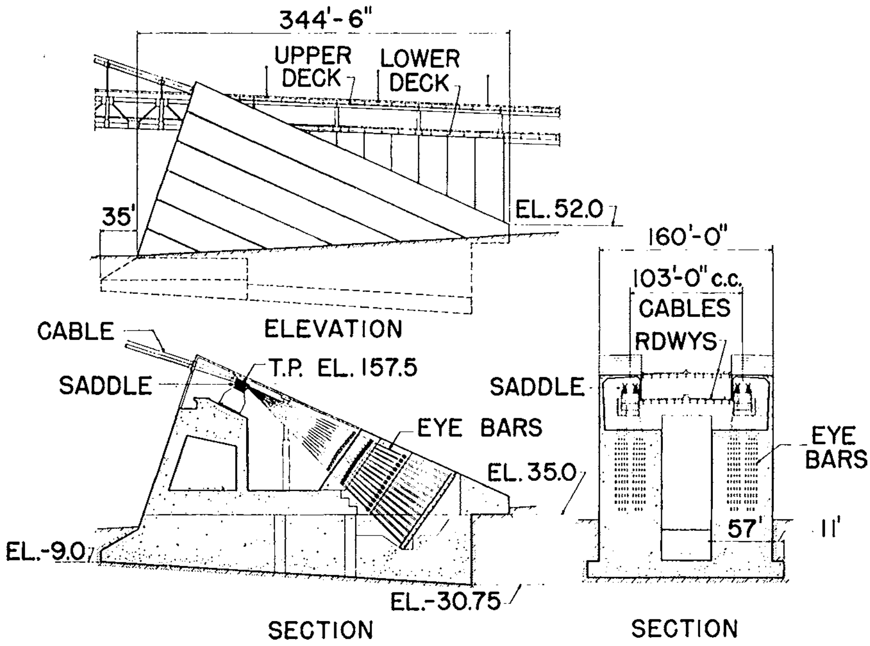

Fig. 1-1—General details Brooklyn anchorage Verrazano Narrows Bridge, Brooklyn, N.Y.

The eyebars are cut from plates conforming to ASTM A 441 which is a weldable grade of steel. The ASTM specification was modified by the addition of requirements for 0.15 percent minimum silicon and the manufacture of the plates by fine grain practice. The fine grained killed steel was considered to be tougher than the basic A 441 steel.

As a further protection, all bars were stress relieved after flame cutting and were carefully examined for edge cracking at the flame cut edges. An extra 1/8 in. width of metal at each edge was used in the shank to permit grinding of edge cracks. The locations of the tensile coupons and bend test specimens were specified to guard against the systematic production of weak or brittle bars. Past practice with quenched and tempered steels required that 3 to 5 percent of the full size bars be tested to destruction. This considerable cost was reduced

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by the elimination of heat treatment and by the use of a standard steel. However, a few full size tests in advance of production fabrication were made to check the adequacy of the design of the heads.

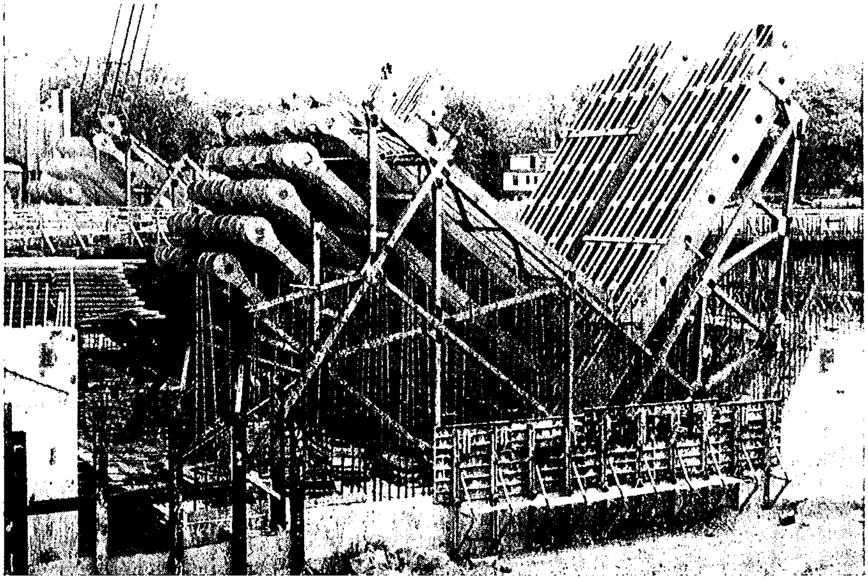

Fig. 1-2—Supporting frame and eyebar chains, Verrazano Narrows Bridge, Brooklyn, N.Y.

A recent circular structure is the Oakland Coliseum in Oakland, Calif. The clear span roof is a cable supported precast concrete system which relies on mechanical fasteners for full strength reinforcing bar splices in both tension and compression. The roof framing consists of 96 radial stiffening ribs, one of which is shown being erected in Fig. 1-3, each supported on a 2 3/16 in. round bridge strand which spans between an exterior cast-in-place ring girder and an inner 45 ft diameter steel tension ring. The end of one rib section is shown in Fig. 1-4.

The constructor was able to erect the radial ribs in two sections each approximately 90 ft long. The depth of each rib varied over its length between 22 ft and 8 ft. All splices were made in the air after all dead load was in place so as to minimize initial stresses in the ribs and the circumferential diaphrams.

Although the precast members were made on the site a close dimensional control was maintained. The connector sleeves were made extra long to provide against some contingencies in length of these large members. The bars were A 432 and the main steel varied in size from #9 to #11.

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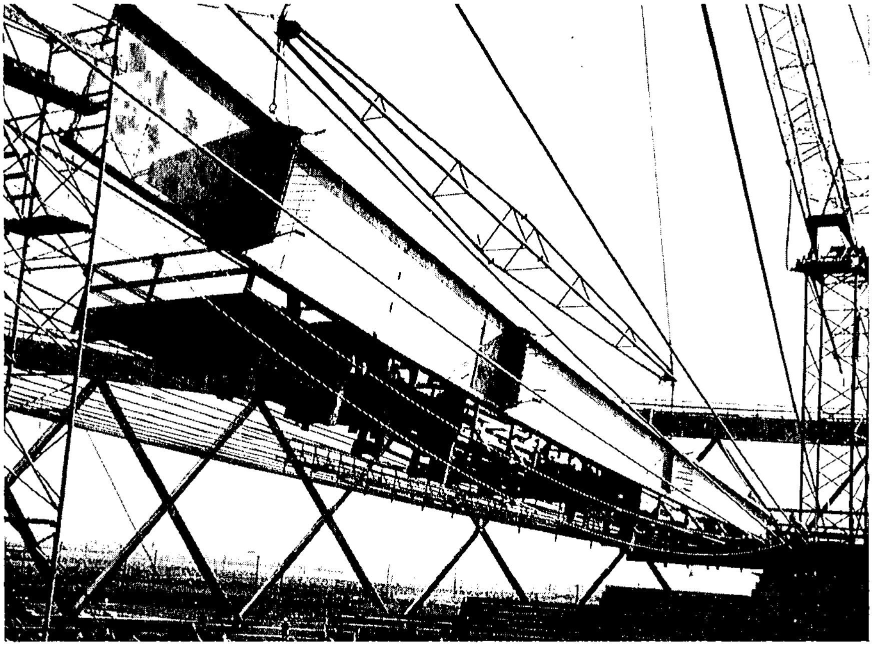

Fig. 1-3—Radial stiffening roof framing rib, Oakland Coliseum, Oakland, Calif.

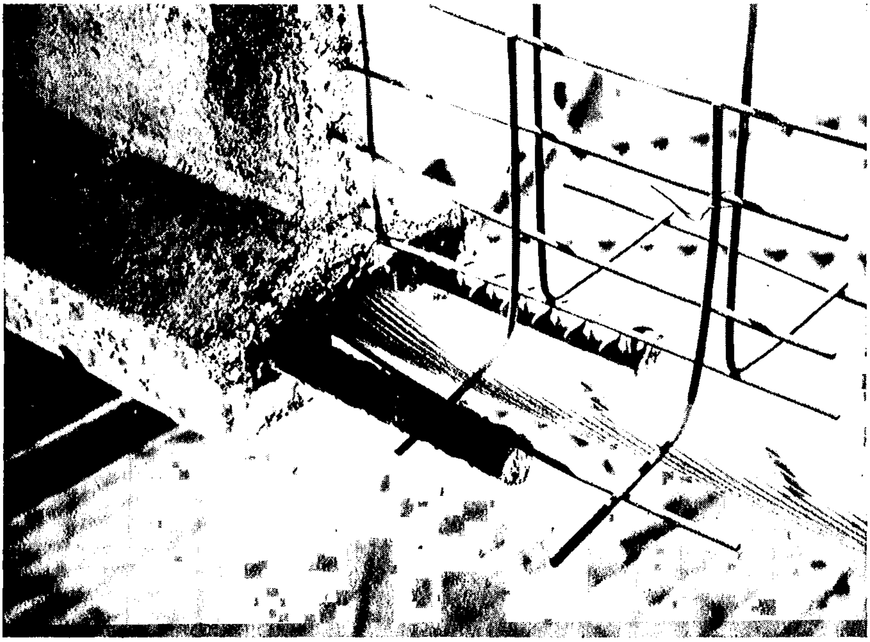

Fig. 1-4—Radial rib end, Oakland Coliseum, Oakland, Calif.

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The Science Museum at the New York World's Fair, now a permanent feature of the New York City Department of Parks, is a structure completely enclosed by a free-standing, serpentine, cellular, ribbed wall approximately 100 ft high. This wall originates at the terrace level and is supported at this level by a radial beam and ring girder system, which, in turn, is supported by columns. The deep beams and girders were heavily reinforced with #11 bars and larger. When it came to joining the keystone shaped ribs of the walls to these beams and girders, there was just not enough room for the #8 and #9 vertical bars of the ribs to fit between the heavy reinforcement of the beams and girders.

The solution involved the first full tension splice utilizing Cadweld sleeves ever used in New York City, and, thus, an innovation into the city building code. A steel plate was attached under the vertical ribs to the beams or girders by means of vertical bars of sufficient number to develop the tension at the connection. These bars were inbedded in the girders within the periphery of the plate and the plate was then welded to the bars. A template of the rib was then laid on the plate to locate the position of the connection sleeves which were subsequently fillet welded to the plate to take the rib's reinforcing bars (Fig. 1-5). The rebars of the ribs were then Cadwelded to the plate. This unique solution to a difficult problem demonstrates the application of a fastener system beyond the original intention of the manufacturer.

For the Metropolitan Opera House at New York's Lincoln Center precast concrete columns were placed 3 ft on center around three sides of the building. The precast units were 10 x 19 in. and 10 x 27 in. The larger units were 60 ft in height. The public area of the building was framed in concrete whereas the stage area was steel framing. Connections had to be devised between the precast columns and the steel and concrete

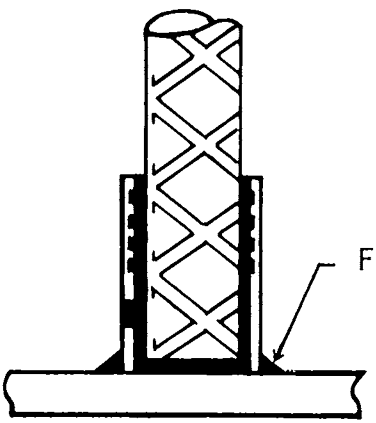

Fig. 1-5—Reinforcing bars in connection sleeves, New York Science Museum

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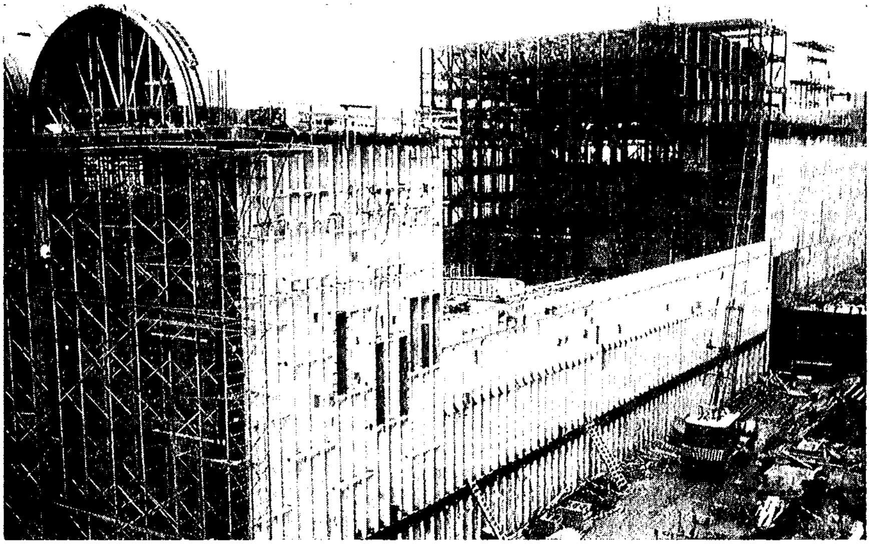

Fig. 1-6—Precast concrete columns and steel framing, Metropolitan Opera House, New York

framework. Fig. 1-6 shows the two distinct sections of the Opera House and some of the precast members in place.

In connecting the precast units to form the perimeter columns, care had to be taken to produce a neat joint without chipping the rounded corners from the precast units. From a recess in the top of the lower precast unit, two 1¼ in. steel dowels protruded. A neoprene gasket was fitted around this recess. Sleeves to receive the steel dowels made from ¼ in. plates welded to a 4 x ¼ in. plate with 1¾ in. holes were in the bottom of the next precast unit. One ⅜ in. grout injection pipe led into the sleeves from outside the column unit. (Fig. 1-7). The columns were erected in series of some 60 columns at one time. When the next level of precast units were in place for the series of columns the alinement was adjusted. Once they were all alined the grouting in place was a simple and swift operation.

The concrete beams were connected to the precast concrete units as shown in Fig. 1-8. The 1¾ in. recess in the precast unit, against which the beam was cast, was sufficient to keep the beam from sliding. Through sleeve nuts above and below the recess, anchor bolts in the precast unit were connected to 1⅛ in. anchor bolts, two at the bottom and two at the top, in the floor beam. Because of the restricted clearance which was available in the top of the beam the anchor bolt lengths were detailed individually to avoid placing problems.

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