Dowels for Anchoring New Concrete Facing to Existing Lock Walls

by T. Liu and T. Holland

Synopsis: This paper presents the results of laboratory and field dowel pullout tests and shear transfer tests. These tests were conducted to evaluate the pullout performance of dowels and the influence of dowel spacing on the load-carrying capacity of the replacement concrete cast during rehabilitation of vertical walls in navigation locks. The results of these tests and a review of the existing literature on the interface shear transfer and the dowel action mechanisms were the bases for the development of rational design criteria for dowels for anchoring replacement concrete to vertical lock walls. The design criteria include surface preparation, minimum dowel size, dowel spacing, and anchorage requirements. An example of designing dowels for a typical lock wall rehabilitation project is included.

Keywords: anchors (fasteners); concrete construction; dowels; joints (junctions); load transfer; locks (waterways); pullout tests; renovating; shear tests; structural design

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SIGNIFICANCE

The research described in this paper is believed to be significant because a rational method for designing the dowels used in the rehabilitation of lock walls was developed to replace the existing "rule of thumb" approach. The new method is simple to apply and is expected to lead to significant cost savings as it is used on future projects.

BACKGROUND

The U. S. Army Corps of Engineers operates and maintains over 200 concrete navigation locks in the navigable rivers of the United States. Of these lock structures, more than half are over 50 years old. With structures of such age, major repair or rehabilitation is often required to assure the safety and continuation of operation of the structures.

A significant portion of most navigation lock rehabilitation projects is work on the lock walls. Typical damage found on lock walls is shown in Figure 1. The deteriorated concrete seen in this figure will be removed, and the lock walls will be restored to original dimensions by the placement of new concrete. A typical construction drawing showing resurfacing details is given in Figure 2.

The first step in the repair technique is to remove the surface concrete to a depth of 12 to 24 in. (300 to 600 mm) by means such as drilling and blasting. The next step is to install dowels in the walls. The dowels serve to position vertical and horizontal reinforcing steel in the replacement concrete and to anchor the replacement concrete to the existing wall elements. The dowels are usually reinforcing bars with a 90-degree bend. They may be anchored into the old concrete by the use of hydraulic-cement grout, epoxy resin, or polyester resin with a 15- to 18-in. (380- to

450-mm) embedment being typical. Figure 3 shows a section of wall in which the installation of dowels has been completed.

Once the dowels are in place, reinforcing steel is placed, exterior forms are positioned, and concrete is placed to restore the walls to original dimensions.

The steps most open to question in the rehabilitation process have been the design and installation of the dowels. No engineering data were found upon which to base dowel size and spacing. As a result, a large number of dowels on close centers (usually 2 ft (600 mm) center-to-center) have been specified. Since installation of the dowels is very labor intensive, it turns out to be a very costly segment of the project.

Some of those who have observed current practice questioned whether the typical size and spacing of dowels are too conservative. A reduction in the number of dowels required for a project could lead to significant savings. The study described in this paper was undertaken at the U. S. Army Engineer Waterways Experiment Station (WES) to resolve this question. This study consisted of five parts, and the results have been reported (1). The parts were:

a. Laboratory pullout tests of dowels anchored with polyester-resin cartridges.

b. Field pullout tests of contractor-installed dowels (the same type of polyester-resin cartridges was used).

c. Literature survey of load-transfer mechanism across the concrete joints.

d. Laboratory tests of load-carrying capacity across such joints that contain dowels representing various percentages of steel.

e. Development of a design method and design guidance for dowels anchoring replacement concrete to vertical lock walls.

LABORATORY PULLOUT TESTS

Laboratory pullout tests were conducted to evaluate the effects of embedment lengths on the pullout resistance of No. 6 (19.0-mm) reinforcing bars. A large concrete block was used as a test bed. Holes for the embedment of reinforcing bars were drilled into the test block. Commercially available polyester-resin cartridges were used as the bonding agent. A total of eight pullout tests were conducted in the laboratory.

The test block in which the reinforcing bars were embedded was cast in 1972 as part of a mass concrete slipform construction program conducted at the WES (2). The block measured 3 by 6 by

10 ft (0.9 by 1.8 by 3.0 m) and contained 6-in. (150-mm) nominal maximum size limestone aggregate. Five 6-in.- (150-mm) diameter cores were drilled from the test block using a diamond bit. These cores were taken from the center and the four corners of the test block to obtain representative samples. Five 6- by 12-in. (152- by 305-mm) specimens were prepared from the cores and were tested for compressive strength and tensile splitting strength. The average compressive strength and the tensile splitting strength were 2510 and 440 psi (17.3 and 3.0 MPa), respectively.

Eight 1-1/8-in.- (29-mm) diameter holes were drilled using a pneumatic rotary-percussive drill. These holes were drilled deep enough to embed No. 6 (19.0-mm) reinforcing bars at embedment lengths equal to 6.0, 7.5, 11.25, and 15.0 in. (150, 190, 285, and 380 mm) (embedment length to nominal bar diameter ratios, L/D = 8, 10, 15, and 20, respectively). To simulate the field conditions, all holes had a 10-degree inclination from the horizontal. The slight inclination was used to prevent loss of the bonding agent. These drilled holes were spaced to allow for a possible 45-degree conical failure of the concrete.

Standard No. 6 (19.0-mm) deformed reinforcing bars were used as dowels. The polyester-resin cartridges (1-in. (25-mm) diameter by 12 in. (600 mm) long) used as the bonding agent consisted of a pack containing a polyester-resin component with a catalyst. The components are isolated from each other by a physical-chemical barrier that prevents reaction between the components until required.

The procedures for embedding the reinforcing bars in the drilled holes were as follows:

a. Air flush the drilled holes to remove all debris and dust.

b. Insert a polyester-resin cartridge.

c. Force the reinforcing bar into the hole breaking the cartridge.

d. Couple a pneumatic drill to the reinforcing bar and rotate the bar into the hole at 200 to 450 rpm.

e. Stop inward movement when the reinforcing bar reached the desired embedment length, and continue rotating the bar for 15 to 20 sec to mix the resin system thoroughly.

f. Stop rotation and uncouple the drill from the reinforcing bar. (The bar was firmly bonded when the resin set in a few minutes.)

The test apparatus used to conduct the pullout tests is shown in Figure 4. The reaction frame was constructed with 9-in. (230-mm) channel sections. The clear span of the reaction frame was 2 ft 2 in. (660 mm). A 60-ton (530-kN), hollow plunger jack in combination with an electric pump was used to apply the axial load to the reinforcing bar. To account for the 10-degree inclination of the reinforcing bar, a special steel shim plate was inserted between the jack and reaction frame. The jacking system was calibrated, and as the load was applied to the bar, a 10,000-psi (70-MPa) pressure gage was monitored. From the pressure gage readings, the jack load was determined for any increment of pressure.

After the polyester resin had cured for approximately 30 minutes, the bar was tested for pullout resistance. A particular test was terminated when no additional load could be applied to the reinforcing bar because of either slippage or excessive elongation.

The results of the laboratory pullout tests are presented in Table 1. All reinforcing bars yielded when the embedment length was greater than 11.25 in. (285 mm) (L/D = 15). The yielding was evident when the rust on the bar began flaking off and when the bar would not support additional load. The yielding was also evident when the axial load reached 20.8 kips (93 kN), which was the yield strength of the bars.

Slippage along the concrete-resin interface generally caused failure when the embedment length was less than 11.25 in. (285 mm). A typical bar after pullout is shown in Figure 5. No concrete failures were observed for the embedment lengths investigated.

FIELD PULLOUT TESTS

Field pullout tests were conducted to determine the pullout performance of dowels installed under field conditions by a contractor working on a major rehabilitation project. At the site selected,* the dowels were being installed by the use of the polyester-resin cartridge system and the basic procedures that had been used for the laboratory pullout tests.

The original test plan envisioned the use of the laboratory pullout test apparatus. This approach was determined to have two problems:

a. Use of the laboratory pullout test apparatus would require installation of special dowels long enough to be gripped by the jack; day-to-day installations could not be tested. This caused concern that the bars to be tested might receive special attention during placement and would not be representative.

b. Site inspection indicated that the test apparatus probably could not be supported in the proper position for testing. The top of the lock walls did not

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* Locks and Dam No. 3, Monongahela River, near Pittsburgh, Pennsylvania.

provide enough room for a crane to operate. All of the test equipment would therefore have to be capable of being placed in and operated from a small boat.

The test plan was revised to allow a random selection of contractor-placed dowels for testing. This modification was achieved by the following revisions to the test equipment (the revised test apparatus is shown in Figure 6):

a. The reaction frame was eliminated, which allowed the reaction force to be transmitted directly to the wall surrounding the dowel. This was determined to be a reasonable approach since none of the bars tested in the laboratory had shown a conical type of concrete failure.

b. The hollow core jack was replaced by two flat jacks mounted on a steel plate. This significantly reduced the length of bar required for testing.

c. The inclined plate and the prestressing chuck used in the laboratory tests were retained for the field tests.

d. A manually operated hydraulic pump was used to power the two flat jacks. The jacks, pump, and gage were calibrated using a laboratory test machine.

The procedure used to perform the pullout tests at the site was as follows:

a. Random dowels were selected from those inside the river chamber and those on the riverside of the middle wall downstream of the river chamber.

b. The bends in the ends of the dowels were straightened using a come-along and a cheater pipe technique. This was necessary to provide enough dowel to grip even with the flat jack apparatus.

c. Surface irregularities in the vicinity of the dowel to be tested were reduced by hand to provide a bearing surface as flat as possible.

d. The pullout apparatus was mounted on the bar. Loading was done in gage pressure increments of 500 psi (3.4 MPa). Each increment was held for approximately 30 sec before the load level was increased.

e. Bars were normally loaded twice in each test. The first loading would exhaust jack travel at a relatively low load because of localized crushing of protruding concrete beneath the inclined wedge. The initial load was then released, and a new grip was obtained on the bar. Loading was then again accomplished in increments as described above.

f. Measurements of actual dowel displacement were not made. The dowels were loaded until a given load could no longer be held, indicated by loss of pressure on the gage, or until the approximate yield range of the steel was reached.

Field pullout test data as shown in Table 2 indicated that none of the bars failed to hold at least the design stress of 40,000 psi (275 MPa).

LITERATURE SURVEY

An understanding of the shear-transfer mechanism across an interface between new and old concrete is of critical importance in dowel design. The shear-transfer mechanism, which acts by combined action of aggregate interlock and friction and dowel action, has been studied by many investigators (3-13), mainly through tests simulating connection and construction joints in precast and cast-in-place concrete. The existing literature was reviewed in this study to evaluate the performance of the combined action of the interface shear-transfer and the dowel-action mechanisms.

Based on the literature review the shear-transfer mechanism across an interface between new and old concrete may be hypothesized as follows.

a. The shear forces initially are transferred through the uncracked interface by bond. When a crack forms at the interface, the shear forces are transferred by the combined action of aggregate interlock and friction and by dowel action.

b. When shear acts along a crack, slip of one crack face occurs with respect to the other. If the crack faces are rough and irregular, this slip is accompanied by separation of the crack faces. This separation will stress the dowel crossing the crack, which in turn will provide a clamping force across the crack face. The applied shear is then resisted by friction between the crack faces and by dowel action of the reinforcement crossing the crack.

The shear-friction theory proposed by Birkeland and Birkeland (3) provides a lower bound to the experimental data available on shear test specimens and was selected as the basis for the dowel design technique. The shear-friction method of calculation assumes that all shear resistance is due to friction between the crack faces. Therefore, a reasonable value of the coefficient of friction for the shear-friction equation must be developed so that the calculated shear strength will be in reasonably close agreement with test results.

LABORATORY SHEAR TRANSFER TESTS

The objectives of the laboratory shear transfer tests were to develop the value of the coefficient of friction to be used in the shear-friction equation and to examine the influence of dowel spacing on the load-carrying capacity of the replacement concrete in a lock wall renovation. These tests were accomplished using laboratory-cast blocks to represent the old and new concretes. Dowels of various diameters were used with the blocks to represent the spacing variations in prototype structures.

The same concrete mixture was used for both old and new portions of the test blocks. The nominal 4500-psi (31-MPa) mixture contained 3/4-in. (19.0-mm) nominal maximum size aggregate. The test blocks were designed to provide a contact area of 576 sq in. (0.4 m^2) between the old and new concretes. The dimensions of the old and new portions of each block were identical. Each portion (old or new) contained approximately 5.7 cu ft (0.2 m^3) of concrete and weighed approximately 850 lb (385 kg).

The old portion of the blocks was cast in a position so that the contact surface between the old and new concretes would be available for finishing. After the concrete had stopped bleeding, a retarder was applied to the contact surface. The forms were removed approximately 24 hr after the blocks were cast, and the contact surface was cut with a low-pressure water jet to remove paste and to provide a roughened surface. The blocks were then stored at 100 percent relative humidity for a minimum of 28 days.

The old concrete blocks were removed from the fog room after at least 28 days. Forms and reinforcing steel were then placed for the casting of the new concrete. A compressible material was used to cast a void between the blocks to allow for movement during testing. After the forms were removed the block assemblies were again stored for 28 days at 100 percent relative humidity.

The compressible material between the blocks was removed prior to the testing. Testing was accomplished using a 440,000-lb (1.96-MN) testing machine that applied loading at a rate of 25,000 lb/min (110 kN/m). Figure 7 shows a block in place in the testing machine. The blocks were loaded to failure (i.e., separation of the new concrete from the old concrete).

Data from the shear-transfer tests are presented in Table 3. The specimens that did not contain dowels showed an average shear strength of 196 psi (1.35 MPa). The average ultimate shear strengths were 203, 234, 242, and 233 psi (1.40, 1.61, 1.67, 1.60 MPa), respectively, for specimens containing No. 3, 4, 5, and 6 (9.5-, 12.7-, 15.9-, and 19.0-mm) dowels.

The effect of the dowels within the range of the percentage of steel tested did not appear to increase significantly the load-carrying capacity of the blocks. Instead, the bond of the new to the old concrete appeared to have been much more significant than the amount of steel present.

Figure 8 plots the average ultimate shear stresses against the values of ρf_y .* There appears to be a slight upward trend in the data that shows a small but increasing contribution from the dowels as dowel size increases.

Based on the test data, the relationship between average ultimate shear stress, V' , and the value of ρf_y was derived using the least-squares fit technique:

$$V' = 200 + 1.35\,\rho f_y \quad (f_y \text{ in psi}) \qquad (1)$$

$$V' = 1.38 + 1.35\,\rho f_y \quad (f_y \text{ in MPa}) \qquad (1a)$$

The value of the coefficient of friction obtained from the test data, 1.35, is consistent with values reported in the literature.

The specimens instrumented during loading experienced essentially no differential movement between the old and new concretes prior to failure with failure defined as the maximum load the block would carry. Two distinct modes of behavior for the replacement concrete were noted at failure. In the specimens without dowels, failure resulted when the top block (new concrete) dropped completely down onto the bottom block as a result of a brittle fracture. In the specimens with dowels (regardless of dowel size), the failure was more ductile; the dowel was able to carry the dead load of the new concrete, thus preventing the new concrete from dropping completely onto the old concrete. This could be of importance in a prototype structure.

Examination of the failure surface of several of the specimens showed that failure did occur on the plane between the old and new concretes. These surfaces showed bond failures with some plucking of aggregate particles from the old and new concretes. A very small percentage of aggregate particles was broken.

DERIVATION OF DESIGN EQUATION

Laboratory test results indicate that well-bonded concrete is relatively strong in shear transfer; however, there is always the possibility that a crack will form at the interface because of

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* ρ = reinforcement ratio

$$= \frac{\text{Area of dowel reinforcement}}{\text{Area of shear plane}}$$

f_y = yield strength of the dowel

shrinkage, thermal stresses, or other reasons. Therefore, in the design of dowels, a crack should be assumed to be present along the interface, with relative displacement along the assumed crack resisted by friction maintained by dowels across the assumed crack.

According to the American Concrete Institute (ACI) Building Code (14), the design of cross sections subject to shear should be based on

$$V_u \leq \phi V_n \tag{2}$$

where

V_u = factored shear force at section considered, lb (N)
ϕ = strength reduction factor = 0.85 for shear
V_n = nominal shear strength, lb (N)

Based on the shear-friction concept,

$$V_n = A_d f_y \mu \tag{3}$$

where

A_d = cross-sectional area of dowel, sq in. (m^2)
f_y = specified yield strength of dowel, psi (Pa)
μ = coefficient of friction

Substituting Equation 3 into Equation 2 and solving for A_d

$$A_d = \frac{V_u}{\phi f_y \mu} \tag{4}$$

The shear-friction concept is valid for conditions in which failure is attained by yielding of the reinforcement crossing the crack. Thus, dowels must be anchored in both sides of the concrete by embedment or hooks to develop yield in the steel.

For the polyester-resin cartridges tested in the laboratory, an embedment length of not less than 15 times the nominal diameter of the dowel was found to be satisfactory. For cement grouts, Stowe (15) also recommended an embedment length of 15 times the nominal dowel diameter. For the epoxies he tested, Stowe recommended a somewhat shorter embedment length of 10 times the nominal bar diameter. Overall, for portland-cement, epoxy-resin, or polyester-resin grouts, an embedment length of at least 15 times the nominal diameter of the dowel should be satisfactory for concretes with a compressive strength of 3000 psi (21 MPa) or greater.

Tests of laboratory specimens indicated the average coefficient of friction between old and new concretes was 1.35. To account for the variations expected in the field construction, a more conservative value of 1.00 should be used. This reduced value of 1.00 is also in agreement with ACI 318-77 (14) recommendations for concrete placed against hardened concrete. To ensure that the coefficient of friction of 1.00 is attainable, all unsound, damaged, dirty, porous, or otherwise undesirable old concrete should be removed, and the old concrete surface should be clean, free of laitance, and intentionally roughened to a relief of at least 1/4 in. (6 mm).