Title no. 101-S48

Shear Stiffness and Strength of Horizontal Construction Joints

by Basel Djazmati and José A. Pincheira

This paper presents the test results of unreinforced concrete construction joints subjected to in-plane shear forces. The main purpose of the study was to determine whether concrete foundations cast in multiple pours with horizontal construction joints could offer the same initial (uncracked) stiffness of those cast monolithically. The experimental program included 36 push-off shear units as well as six slab specimens. The study included several methods to bond fresh to hardened concrete and considered both technical and practical aspects related to the preparation, casting, and curing procedures of the joints. Based on the test results, it is concluded that members with a properly prepared and moist-cured joint offer the same initial stiffness as that of a member cast monolithically. Recommendations concerning the preparation, curing, and casting procedures to achieve a proper performance of the joint are provided.

Keywords: concrete; construction joint; foundation; stiffness; strength.

INTRODUCTION

Massive concrete elements such as large foundations are sometimes required to support sensitive equipment and machinery. In such cases, stringent alignment tolerances between the work piece and the machine of less than 0.00025 in./ft (0.02 mm/m) are required. To ensure proper operation, these foundations must be very rigid and, as a result, they are designed to have areas of 500 ft^2, or more, and can be 10 ft deep, or deeper. Commonly, these foundations are cast monolithically to ensure that they are sufficiently rigid and that they satisfy the strict vibration and deformation requirements. The construction of these large foundations can be cumbersome, time consuming, and expensive. Common problems encountered during the construction of these foundations are that casting the foundations in one pour requires multiple work shifts, and ready-mix plants have difficulty supplying the required amount of concrete for a single pour, especially when these foundations are needed in remote locations.

For these reasons, the use of multiple pours (with construction joints) can simplify the construction process and reduce costs. Construction joints are, however, potential planes of weakness where slip, dilation, and, ultimately, delamination can occur. Consequently, the stiffness of the foundations could be reduced to a point where proper functioning of the equipment they support is compromised.

Several techniques have been reported in the literature concerning the preparation of a construction joint surface. These documents often describe the current and best practices that have led to good quality construction joints in the past.[1-5] While correlation studies between the joint surface preparation and the performance of the joint exist, most of them have focused on the strength of members with steel reinforcement across a cracked joint. The behavior of uncracked joints and, in particular, the stiffness of unreinforced joints prior to cracking has not been measured in past studies. The latter is of utmost importance to this study because these foundations are not expected to be cracked during their service life, and stiffness rather strength is their main design consideration. Also, these foundations are virtually unreinforced except for temperature and shrinkage reinforcement.

Several methods for transferring shear across concrete construction joints have been studied in the past. Some studies have focused on examining the bond characteristics between concrete cast at different times and the role of dowels across the joint, while others have investigated the concept of shear friction between concrete surfaces.

Tynes and McCleese[1] concluded that casting concrete on dry surfaces and without mortar increases the strength of the joints. Similar conclusions were made by Waters.[2] Hanson[3] investigated the shear capacity between precast and cast-in-place concrete and concluded that in bonded surfaces the capacity of the shear connections averaged approximately 400 psi (2.8 MPa) for specimens with rough and bonded surfaces, which was reached at relatively low slip of approximately 0.001 in. (0.025 mm). He also found that the presence of keys had a slight effect on the shear capacity when the surface was roughened and bonded.

Bass, Carrasquillo, and Jirsa[4] experimentally studied the interface shear capacity between new and existing concrete push-off specimens. It was found that before cracking, the strength of heavily sandblasted surfaces was comparable with that of chipped surfaces and with shear keys. The shear strength varied from approximately 400 to 500 psi (2.8 to 3.4 MPa). In addition, it was found that increasing the reinforcement in the joint increased the shear capacity of the specimens after cracking (at large slip levels).

Other researchers[6-8] have also concluded that increasing the number of dowels and/or their size improved the behavior of the construction joints only after cracking and after relatively large deformation has occurred. The specimen's capacity reached at small slips (at cracking) did not improve with the number of dowels across the joint. It was also found that the behavior of specimens with rough and clean surfaces was as good as the behavior of specimens cast monolithically.

None of the studies found in the literature have, however, focused specifically on the shear stiffness of concrete construction joints before cracking. Most studies were concerned with joint strength where shear deformations could be measured only after cracking. Hofbeck, Ibrahim, and Mattock[9] reported, for example, that "...no movement could be

ACI Structural Journal, V. 101, No. 4, July-August 2004.
MS No. 02-460 received December 9, 2002, and reviewed under Institute publication policies. Pertinent discussion including author's closure, if any, will be published in the May-June 2005 *ACI Structural Journal* if the discussion is received by January 1, 2005.

Basel Djazmati *is a supervisory engineer with Framatome ANP Material and Structural Analysis Unit. He received his BS in civil engineering from the University of Aleppo, Syria; his MS in civil engineering from the University of Wisconsin, Madison, Wis.; and his MBA from the University of Chicago, Chicago, Ill.*

José A. Pincheira, *FACI, is associate professor of civil and environmental engineering at the University of Wisconsin. He is Chair of ACI Committee 369, Seismic Repair and Rehabilitation; a member of ACI Committees 374, Performance-Based Seismic Design of Concrete Buildings, 440, Fiber Reinforced Polymer Reinforcement; and Joint ACI-ASCE Committee 445, Shear and Torsion. His research interests include the behavior of reinforced concrete members subjected to earthquake loads and the seismic rehabilitation of existing concrete structures.*

detected in the initially uncracked specimens until diagonal tension cracks became visible at shear stresses of about 500 to 700 psi (3.4 to 4.8 MPa)." Moreover, the majority of past studies focused on construction joints with steel reinforcement across the joint, which becomes effective only after joint cracking.

RESEARCH SIGNIFICANCE

Past studies suggest that concrete with a rough, clean joint may perform as well as one cast monolithically. Data on the stiffness of construction joints before cracking are lacking. This study was conducted to provide new experimental data on the stiffness and strength of unreinforced concrete joints and to establish whether members cast in two pours can offer the same stiffness of those cast monolithically.

EXPERIMENTAL PROGRAM

The experimental study was divided into two phases. In the first phase, 36 push-off shear specimens were cast and tested to failure under monotonically increasing loading. Of these, 32 units were cast in two pours with a horizontal construction joint, while the remaining four specimens were cast monolithically. The joints were prepared using different methods that included joint surfaces with no special preparation, intentionally roughened, as well as surfaces with smooth finishes with and without a bonding agent layer between the hardened and fresh concrete. Joints with shear keys were also studied. The effects of having a wet or a dry surface before casting the top layer, as well as the effects of concrete compaction on joint stiffness and strength, were also evaluated.

In the second phase of the study, six slab specimens were tested. Three units were cast with a horizontal construction joint at mid-depth while the rest were cast monolithically. The construction joints in the slab units were all roughened with a stiff broom, but the curing conditions and the time elapsed between the top and bottom pours were varied.

While the use of joint steel reinforcement was considered, it was decided to provide no reinforcement across the joints in any of the push-off or the slab specimens. Past studies have shown that the contribution of the steel reinforcement to joint stiffness and strength is only effective after joint cracking.[6-8] Furthermore, the additional cost associated with the labor involved in providing the joint reinforcement was judged to offset any savings obtained by casting the foundations in multiple pours.

PUSH-OFF SHEAR TESTS

Test specimens

Thirty-two push-off specimens were cast in two pours. The first pour formed the bottom block of the specimen, while the second formed the top block (refer to Fig. 1). In all units, the top block was cast approximately 24 h after casting the bottom block. Four specimens were cast monolithically, which served as a benchmark for the units cast in two pours.

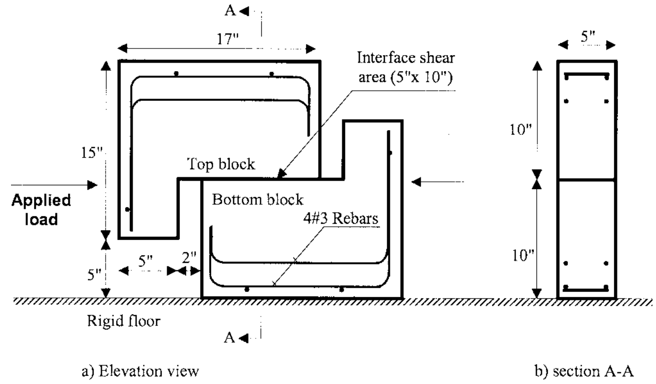

Fig. 1—Details of push-off specimens (Note: 1 in. = 25.4 mm).

All units were designed to be 20 in. (508 mm) high and 5 in. (127 mm) wide, with an interface shear area of 5 x 10 in. (127 x 254 mm). To prevent flexural or load bearing failures, the specimens were reinforced with four No. 3 steel reinforcing bars away from the shear plane as shown in Fig. 1. The preparation procedure of the joint surface was the main variable of this test series. The surface preparation methods were chosen to be simple, practical, and economical. Therefore, labor intensive methods or procedures that result in large amounts of fine particles, such as sandblasting, were not considered.

Past studies have suggested that the bond across a construction joint is improved when fresh concrete is cast on a dry, hardened surface.[1,2] To study this effect, the joint surface in two specimens was sprayed with water before casting the top block. At the time of pouring the fresh concrete, the hardened surface was still damp, but no free-standing water was observed. In the remaining specimens, the top block was cast on a dry hardened surface. The surface preparation procedures used in the study are briefly described as follows:

1. No surface preparation: No attempt was made to roughen the joint surface. The joint surface was simply leveled with a screed;

2. Broomed surfaces: The joint surface was troweled and leveled with a screed. Approximately 1/2 h after casting, a stiff, bristled broom was used to roughen the surface. With this procedure, the broomed surface had an average of approximately four grooves per inch (four grooves per 25.4 mm) and a groove depth that ranged from 1/8 to 1/4 in. (3.1 to 6.3 mm). The angle between the grooves and the direction of the applied shear load, hereafter groove angle, was varied for different specimens. Specimens with groove angles of 0, 45, and 90 degrees with respect to the applied shear load were prepared (refer to Fig. 2);

3. Intentionally roughened surfaces: The joint surface was prepared according to provisions of the ACI code[10] for an intentionally roughened joint, that is, the interface was "roughened to a full amplitude of approximately 1/4 in. (6.3 mm)." The ACI code, however, does not specify the width or the spacing of the grooves (roughness) of the joint. In this study, 1/4 to 1/2 in. (6.35 to 12.7 mm) wide grooves spaced at 3 in. (76.2 mm) were used.

3. Shear keys: These specimens were cast with a 1.5 x 1.5 in. (38.1 x 38.1 mm) square, 1 in. (25.4 mm) deep shear key in the bottom block (refer to Fig. 3). To form the key, the surface was leveled with a screed and a wood block was inserted into the bottom block;

4. Smooth surfaces: The surface of the bottom block was troweled to provide a smooth finish; and

Table 1—Push-off tests: Specimen designation

Specimen designation	Placement condition	Surface preparation
VMP1-NSPW	Compacted using steel rod	None—wet surface
VMP2-NSPW		
VMP1-NSPD		None—dry surface
VMP2-NSPD		
VMP1-B0		Broomed parallel to load—dry surface
VMP2-B0		
VMP1-B45		Broomed at 45 degrees—dry surface
VMP2-B45		
VMP1-B90		Broomed at 90 degrees—dry surface
VMP2-B90		
VMP3-B90		
VM1	Compacted using 1 in. diameter vibrator	Monolithic
VM2		
VM3		
VM4		
VMP1-NSPV		None—dry surface
VMP2-NSPV		
VMP3-NSPV		
VMP1-B0V		Broomed parallel to load—dry surface
VMP2-B0V		
VMP3-B0V		
VMP1-B90V		Broomed at 90 degrees—dry surface
VMP2-B90V		
VMP3-B90V		
VMP-ACI-0		Roughened per ACI parallel to load—dry surface roughened per ACI perpendicular to load—dry surface
VMP-1-ACI-90		
VMP2-ACI-90		
VMP1-SK		Shear key—dry surface
VMP2-SK		
VMP3-SK		
VMP1-SM		Smooth—dry surface
VMP2-SM		
VMP3-SM		
VMP1-BA		Surface with bonding agent
VMP2-BA		
VMP3-BA		

Note: 1 in. = 25.4 mm; 1 psi = 0.0069 MPa.

5. Surfaces with a bonding agent: These specimens had a smooth surface finish; but, in addition, a commercially available bonding agent was applied between the two blocks. Four bonding products were considered in this study. The selected bonding agent was a resin emulsion. This was done at the request of the sponsor to simulate the conditions where an existing foundation, whose surface is already smooth-finished, needed to be enlarged and fresh concrete would be placed on top.

In all specimens cast in two pours, the hardened surface was cleaned from any dust particles and loose aggregate with a stiff brush before casting the second pour. A summary of the test specimens is shown in Table 1.

Materials

A specially designed mixture without additives was used to achieve a specified strength of 5000 psi (34.5 MPa) at the target test date, typically 7 days after casting. The concrete slump was chosen to be 3 in. (76.2 mm). The maximum aggregate size used was 3/4 in. (19 mm). All specimens were moist cured at ambient temperature. Three standard cylinders were cast and tested for strength from the top and bottom blocks of each specimen and from the monolithic specimens. The cylinders were also moist cured and were tested shortly after testing of the corresponding specimens.

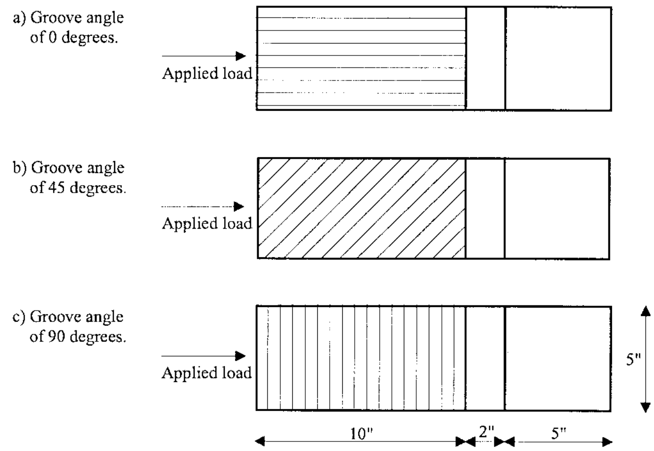

Fig. 2—Groove angles investigated for specimens with broomed surfaces (Note: 1 in. = 25.4 mm).

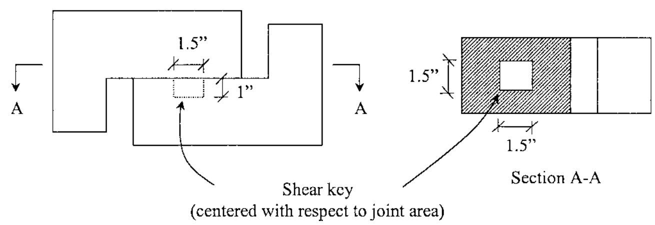

Fig. 3—Dimensions and location of shear key (Note: 1 in. = 25.4 mm).

To obtain a lower bound of the strength and stiffness of the joint, the concrete for some specimens was simply placed in the forms and compacted using a steel rod. This would simulate concrete cast without vibration or poorly vibrated (refer to Table 1). For the remaining specimens, the concrete in both top and bottom blocks was vibrated using a 1 in. (25.4 mm) diameter vibrator.

Test setup and procedure

All specimens were placed in a self-reacting steel frame and tested to failure using monotonically increasing load under displacement controlled (refer to Fig. 4). Most specimens were 7 to 10 days old when the tests were conducted. During the tests, the displacement of the hydraulic ram was held constant at selected load levels to search for cracks or any evidence of distress in the specimens.

Considerable effort was spent to establish a reliable procedure to measure the specimens' deformation during the tests.[11] Elastic finite element analyses of the specimen indicated that deformations in the order of 50×10^{-6} in. (0.127×10^{-3} mm) would be induced per 1000 lb (4.4 kN) of shear force. Also, past studies[4] have suggested that deformations as small as 0.0012 in. (0.03 mm) could be expected at joint cracking. Several deformation measuring techniques were explored[11] including the use of strain gages, Moiré interferometry, laser sensors, noncontacting gages, demec points, and linear variable differential transformers (LVDTs). Of these, high-resolution LVDTs presented the best option within the project budget. The resolution of the chosen devices varied, on average,

Fig. 4—Test setup of push-off specimens.

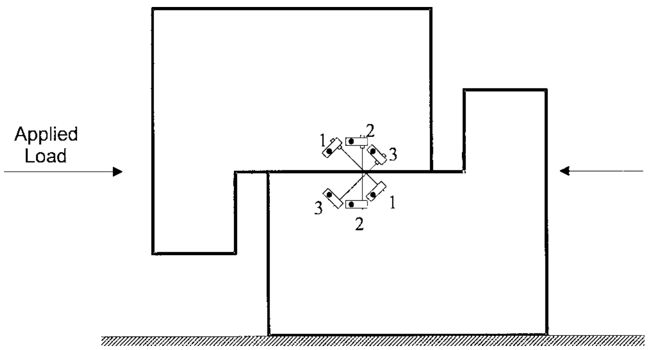

Fig. 5—Forty-five degree angle rosette arrangement of LVDTs.

between ±20 × 10^{-6} to ±40 × 10^{-6} in. (±0.52 × 10^{-3} to ±1.02 × 10^{-3} mm).

The LVDTs were arranged in a 45-degree rosette to allow the calculation of deformations in any other direction in the plane of the rosette (refer to Fig. 5). The displacements reported hereafter are in terms of Δ_{1-1}, Δ_{2-2}, and Δ_{3-3} for displacements measured in directions 1-1, 2-2, and 3-3, respectively (refer to Fig. 5).

PUSH-OFF TESTS: OBSERVED BEHAVIOR AND TEST RESULTS

For the monolithic specimens, failure was very sudden and brittle, without joint cracking or any other evidence of joint shear distress prior to failure. Failure occurred near the shear plane, but it was not defined by a single fracture plane. Instead, failure occurred abruptly along two or three surfaces as shown in Fig. 6. In these units, all the aggregate was sheared off at the fracture planes.

For specimens cast in two pours, failure always occurred along the joint plane (refer to Fig. 7). Usually, most of the aggregate was exposed on the failure surface, but only a portion of this aggregate was fractured. As was expected, the amount of exposed and fractured aggregate was dependent on the compaction technique employed. The specimens compacted using a vibrator had a consistently higher percentage of fractured aggregate than those compacted with a steel rod. The type of surface preparation also had an effect, but no clear trends could be identified. None of the specimens cast in two pours showed, however, an amount of fractured aggregate comparable with that observed in the monolithic units. The percent of fractured aggregate was at best approximately 20% of the exposed aggregate in the units cast in two pours compared with 100% in the monolithic units.

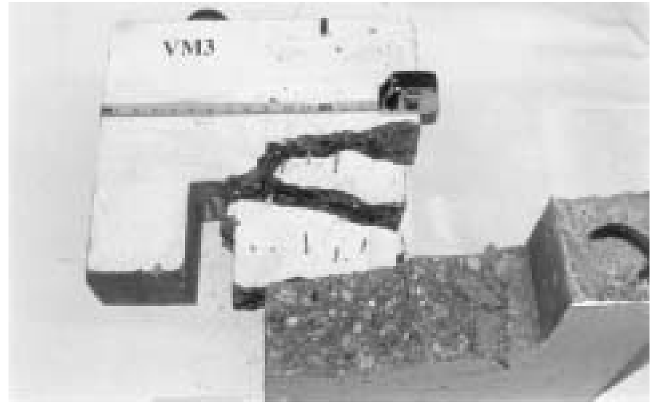

Fig. 6—Typical failure mode and appearance of failure planes for monolithic specimens.

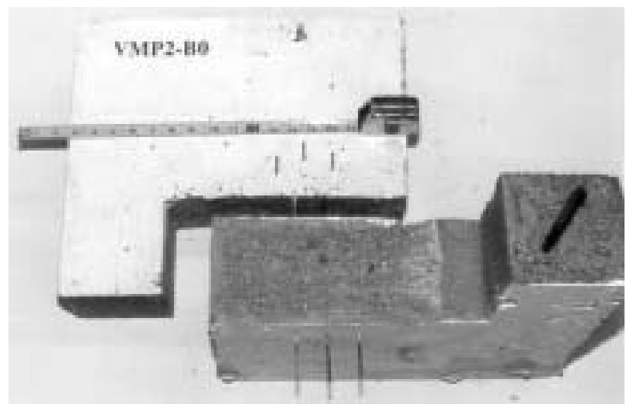

Fig. 7—Typical failure mode and appearance for specimen cast in two pours with broomed surface.

The units with shear keys had nearly all of the aggregate sheared off within the key region. Outside of the key region, the aggregate was exposed, but not all of it was sheared off. This result suggested that the concrete within the key was as effective as that in the monolithic specimens. The specimens with a smooth finish, with or without a bonding agent, left a clean, smooth failure plane with no exposed aggregate, as was expected.

The failure mode of the specimens cast in two pours was also brittle in nature, with little or no warning before failure. An exception to this behavior was the response observed for the units provided with the bonding agent on the joint surface. These units were considerably more flexible than any of the other specimens, as was evidenced by the relatively large deformations recorded during the test, even at the early loading stages. This behavior, which was observed in all three tested units, showed that painting the joint surface with the bonding agent was, in fact, detrimental to the joint stiffness.

The measured load and deformation relations obtained for one specimen cast in two pours (VMP2-B90V) are presented in Fig. 8. The figure shows the data obtained along three directions (1-1, 2-2, and 3-3) on one face of the specimen. In the figure, elongation of the unit in a given direction is considered a positive deformation while shortening is considered a negative deformation.

In Fig. 8, the deformations recorded along directions 2-2 and 3-3 indicate a nearly linear relation with increasing load up to a load level of approximately 22 kips (97.9 kN). At higher load levels, the deformation increased at a higher rate

Table 2—Measured strength of specimens

Specimen	f_c^b, psi	f_c^t, psi	f_c', psi	V_u, kips	v_u, psi	$\left(\frac{v_u}{\sqrt{f_c'}}\right)$
VMP1-NSPW	7555	7516	7536	15.2	304	3.5
VMP2-NSPW	6359	6425	6392	12.9	258	3.2
VMP1-NSPD	5250	5443	5347	22.4	448	6.1
VMP2-NSPD	6573	6075	6324	19.4	388	4.9
VMP1-B0	5526	5553	5540	18.5	370	5.0
VMP2-B0	5914	5718	5816	12.1	242	3.2
VMP1-B45	5812	5765	5789	20.8	416	5.5
VMP2-B45	5292	5326	5309	12.3	246	3.4
VMP1-B90	5606	5970	5788	10.8	216	2.8
VMP2-B90	5664	5039	5352	14	280	3.8
VMP3-B90	5863	5363	5613	14.6	292	3.9
VM1	—	—	5091	43.4	869	12.2
VM2	—	—	7184	49.2	984	11.6
VM3	—	—	6685	46.0	920	11.3
VM4	—	—	6748	46.0	919	11.2
VMP1-NSPV	6835	6663	6749	29.3	585	7.1
VMP2-NSPV	5999	5287	5643	35.2	704	9.4
VMP3-NSPV	6272	5952	6112	34.6	691	8.8
VMP1-B0V	6131	7268	6700	30.6	611	7.5
VMP2-B0V	6489	6855	6672	28.1	561	6.9
VMP3-B0V	6178	5987	6528	31.7	633	7.8
VMP1-B90V	6188	6586	6387	31.0	620	7.8
VMP2-B90V	6442	6840	6641	28.0	560	6.9
VMP3-B90V	6806	6249	6528	28.3	566	7.0
VMP-ACI-0	6679	5942	6311	34.7	694	8.7
VMP1-ACI-90	6167	5741	5954	29.6	591	7.7
VMP2-ACI-90	6165	6042	6104	27.1	542	6.9
VMP1-SK	5652	6278	5965	20.3	405	5.2
VMP2-SK	6288	6232	6260	27.2	543	6.9
VMP3-SK	6660	6272	6466	29.6	591	7.3
VMP1-SM	6056	6255	6156	17.2	343	4.4
VMP2-SM	6317	5735	6026	16.4	327	4.2
VMP3-SM	6495	5973	6234	20.7	413	5.2
VMP1-BA	7003	6754	6879	16.0	320	3.9
VMP2-BA	5974	6232	6103	18.9	378	4.8
VMP3-BA	6756	6466	6611	11.2	224	2.8

Note: 1 psi = 0.0069 MPa; 1 kip = 4.44 kN.

with small load increments. The deformation recorded along direction 1-1 (Fig. 8) showed shortening with increasing load (as was expected) up to a load level of approximately 22 kips (97.9 kN). At higher loads, however, shortening occurred at a decreasing rate and eventually the unit began to dilate in that direction. The reason for this change in deformation direction may be explained as follows. Initially, at low loads, shortening of the joint, parallel to the joint surface, predominates over joint dilation (that is, along direction 2-2), which results in a net shortening of the joint along direction 1-1 with increasing load. At higher loads, typically shortly before failure, joint dilation increases at a faster rate than shortening parallel to the joint does. As a result, shortening along the diagonal direction 1-1 decreases with increasing load and eventually changes to a net dilation. This change in the deformation along direction 1-1, accompanied with the change in the rate of elongation along the other two directions was observed in nearly all units, and it was always an indication that joint failure was imminent. It must be mentioned that visual

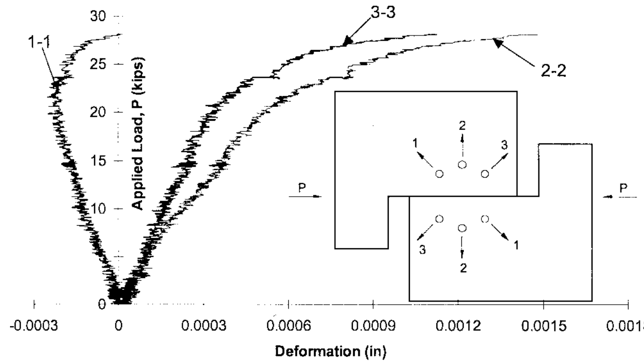

Fig. 8—Load versus deformation measured on west side of Specimen VMP2-B90V (Note: 1 in. = 25.4 mm; 1 kip = 4.44 kN).

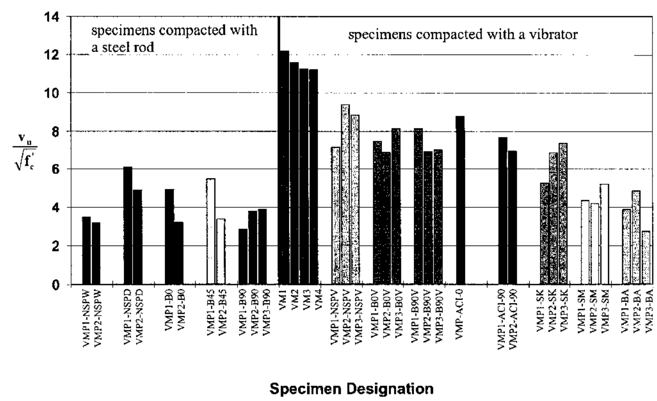

Fig. 9—Normalized shear stress of push-off shear specimens at failure.

inspection of the units just before failure showed no visible signs of joint distress.

The maximum measured load V_u and the average shear stress at failure v_u, calculated as the maximum measured load divided by the shear area, are presented in Table 2. Also shown in Table 2 is the cylinder compressive strength of the concrete for both the bottom pour f_b^c and top pour f_c^t. The concrete strength was calculated as the average strength for each block of three cylinders tested at the time of testing the specimens. The results show that the compressive strength for the bottom and top pours, f_c^b and f_c^t, respectively, are generally very close in value. Thus, the average of these two strengths is reported in Table 2 as f_c'. The last column in Table 2 shows calculated shear stress v_u normalized with respect to the square root of the average cylinder compressive strength $\sqrt{f_c'}$. This was done to account, albeit approximately, for the variation in the concrete compressive strength f_c' for different specimens.

PUSH-OFF TESTS: DISCUSSION OF TEST RESULTS

Joint strength

Figure 9 shows the measured strength of all specimens expressed in terms of the applied shear stress v_u normalized with respect to $\sqrt{f_c'}$. The data show that the compaction procedure had a significant influence on the strength of the specimens, irrespective of the type of joint surface preparation. When the concrete was compacted with a rod, specimens with no surface preparation and a dry surface (NSPD series) tended to develop higher strengths than those with roughened and dry surfaces (B0, B45, and B90 series). This result was unexpected and puzzling at first.

Close examination of the joint surface after the tests showed that the effective shear area, for example, the area that was actually bonded between the top and bottom blocks was smaller in the specimens with roughened joints. Because the concrete was compacted with a rod instead of a vibrator, the cement paste of the top pour did not completely fill the grooves of the roughened joints. Such a problem was reduced or eliminated in the specimens without surface preparation (or not intentionally roughened), which had a smoother and more uniform surface. Thus, a larger effective shear area was, in effect, provided in the latter units. This explanation is also supported by the results obtained from the specimens with broomed surfaces compacted with a vibrator. The latter units had considerably more strength than those compacted with a rod. This result suggests that the use of a vibrator allowed the cement paste to fill in the voids in the bottom block, which resulted in a larger effective shear area.

The monolithic specimens developed the highest strength of all, with an average shear strength of 923 psi (6.4 MPa) or approximately $11.6\sqrt{f_c'}$. For specimens without surface preparation (NSP series) and compacted with a rod, a wet joint had a detrimental effect on the joint strength. Similar results have been reported in past studies.[1,2] A possible explanation for this result is that the water poured on the joint surface worked as a bond breaker that ultimately reduced the adherence between the two surfaces.

The groove angle (the angle between the groove created on the surface and the direction of the load) did not appear to have a significant effect on strength. The shear strength for groove angles of 0, 45, and 90 degrees was similar and within the expected variability for these kinds of tests.

Specimens with joint surfaces roughened per the provisions of the ACI code developed strengths similar to those with broomed joints or without surface preparation. The use of shear keys did not result in a significant gain in strength. These specimens developed strengths equal to or lower than those with a rough surface, whether it was intentionally roughened or not.

As expected, the specimens with a smooth joint surface (SM series) developed lower strengths than those with rough surfaces. Furthermore, the addition of a bonding agent was detrimental to the strength of the joint. These specimens (BA series) developed the lowest strengths of all specimens compacted with a vibrator.

Joint stiffness

Although a considerable effort was spent to carefully set up the LVDTs, the measurements were not always accurate and/or reliable at small deformations (that is, at approximately 1×10^{-5} in. or smaller). Full details of the procedure to record the deformations are described elsewhere.[11] In the following, the results of those specimens where reliable readings of the strain field were obtained in the joint region are presented. Also, only specimens compacted with a vibrator are presented.

Figure 10 shows the average shear stress v_u and average shear strain relation. The figure shows that most surface preparation methods resulted in comparable joint stiffnesses. Specifically, units with roughened surfaces had an initial stiffness comparable with that of the monolithic units up to a shear stress of approximately 400 psi (2.8 MPa), irrespective of the procedure employed to roughen the surface. Units

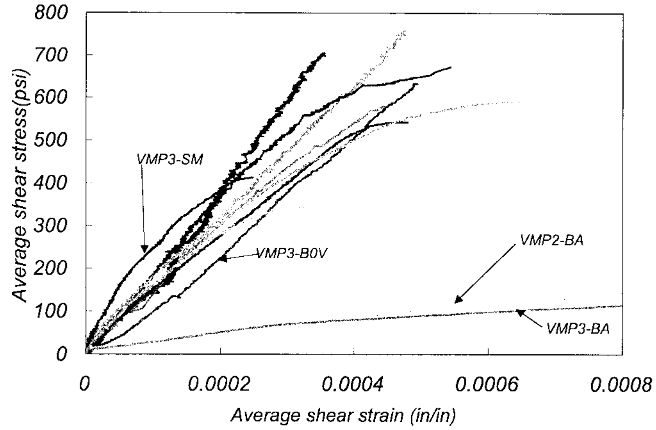

Fig. 10—Applied shear stress and shear strain relation of selected push-off shear specimens.

with smooth joints, on the other hand, had a comparable stiffness up to only approximately 250 psi (1.7 MPa). Units with joints painted with a bonding agent were much more flexible than the rest of the specimens. The bonding agent acted as a soft, flexible layer between the top and bottom, much more rigid, concrete blocks. As a result, most of the deformation was concentrated in the softer bonding agent layer, which led to a significantly smaller joint stiffness. It must be noted that the bonding agent layer was so flexible that the measured joint shear strains were approximately 50 times larger that those recorded for the rest of the units. Even at these larger deformations, failure of the joint did not occur, that is, the top block was still bonded to the bottom block.

In summary, the results obtained from the push-off specimens showed that a roughened joint and compacted with a vibrator had an initial stiffness comparable to that of the monolithic units up to a shear stress of 400 psi (2.8 MPa). The procedures employed to roughen the joint surfaces did not affect the initial stiffness. Specimens with a smooth joint also had an initial stiffness comparable with that of the monolithic units, but it could be maintained only for shear stresses up to 250 psi (1.7 MPa).

SLAB TESTS: EXPERIMENTAL PROGRAM

Based on the results from the push-off experimental program, an intentionally roughened joint surface was judged, in conjunction with the sponsor, to be the best suited procedure for foundations cast in multiple pours, from both practical and economical aspects. Therefore, it was the only surface preparation procedure employed in subsequent series of slab tests.

The slabs used in the experiments were designed and loaded to represent the stress field along the horizontal joint of the foundations of interest to this study. Three slabs were cast in two pours using an intentionally roughened joint, while three additional slabs were cast monolithically, which served as a benchmark for the slabs cast in two pours.

Test specimens

In practice, the foundations of interest to this study are designed so that they remain elastic (uncracked) under the anticipated service loads. Typically, the maximum expected shear stress may locally reach 70 to 80 psi (0.48 to 0.55 MPa). Based on these considerations, the prototype slabs were designed, proportioned, and loaded to develop shear stresses at the horizontal construction joint of approximately 80 psi (0.55 MPa) before reaching flexural cracking. In this manner,

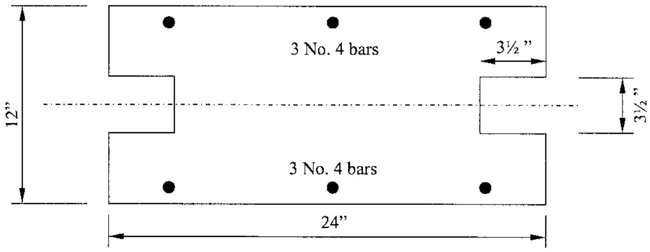

Fig. 11—Cross-sectional dimensions of slab specimens.

Fig. 12—Close-up view of broomed joint surface.

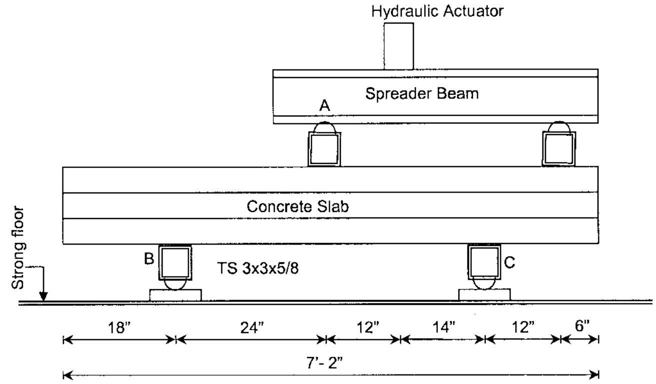

Fig. 13—Test setup and loading system for slab specimens (elevation).

Table 3—Description of slab units

Specimen	Casting procedure	Time elapsed between pours	Curing condition
SM1	Monolithic	NA	Moist cured covered with plastic
SM2		NA	
SM3		NA	
SMP1	Two pours	1 day	
SMP2		3 days	
SMP3		7 days	Uncovered, exposed to laboratory air

the slabs were expected to exhibit essentially linear elastic behavior up to the shear stress level of interest (that is, up to approximately 80 psi).

Figure 11 shows the cross section dimensions of the test specimens. An I-shaped cross section with the construction joint located at mid-depth was chosen. As shown in the figure, three No. 4 steel reinforcing bars were provided at the top and bottom of the slab to prevent sudden collapse following flexural cracking.

Table 3 shows the specimen designation according to the casting procedure employed, that is, either monolithically or in two pours. The joint surface of Specimens SMP1, SMP2, and SMP3 was intentionally roughened with a stiff, bristled broom shortly after casting the concrete. A closeup view of the broomed surface is shown in Fig. 12. Typically, the broomed surface resulted in grooves of approximately 1/8 to 1/4 in. (3.1 to 6.3 mm) deep, similar to those of the units used in the push-off shear tests. No attention was paid to the orientation of the grooves as the results from the push-off shear tests indicated no correlation between the groove angle and the stiffness, or strength, of the joint.

Materials

All the slabs were cast using a ready-mix concrete company. The strength of the concrete was specified as 5000 psi (34.5 MPa) at 7 days. The specified concrete slump was 3 in. (76.2 mm) and the maximum aggregate size specified is 3/4 in. (19 mm). All slabs (top and bottom layers of units cast in two pours) were compacted thoroughly using a 1 in.-diameter (25.4 mm) vibrator. The joint surface was thoroughly cleaned to remove all dust and particles. As shown in Table 3, the time between casting the top and bottom layers as well as the curing conditions varied between specimens. This was done to evaluate potential changes in the stiffness of the slabs due to variations in the curing time and conditions, and to reflect the fact that there is no assurance in the field that the top layers of the foundations would be cast always a day after casting the bottom layer.

Curing of all the units, except Specimen SMP3, was done at ambient temperature by spraying water on the top surface that was later covered with a plastic sheet. The top surface of the bottom layer was kept moist, but there was no freestanding water when the top layer was cast. The joint surface of Specimen SMP3, however, was left uncovered, and exposed to the laboratory air to simulate poor curing conditions.

Three standard cylinders were cast for each batch of concrete to obtain an estimate of the concrete strength. The cylinders were tested in compression on the same day the slabs were tested.

Test setup and procedure

Figure 13 shows the elevation view of the test setup of the specimens. As shown, a staggered two-point loading system was used. Figure 14 shows the shear stress contours for the slabs under the adopted loading scheme. As shown in the figure, the maximum shear stresses that develop between loading Point A and the reaction Point C (refer to Fig. 13) are nearly constant over this length and are approximately 72 psi when flexural cracking was estimated to occur.

The load was applied through a stiff, steel spreader beam. The spreader beam was in turn supported on 3 x 3 x 5/8 in. (76.2 x 76.2 x 15.8 mm) tube sections to spread the load over the slab width. Similarly, the slab was supported on the structural floor using tube sections of the same dimensions (refer to Fig. 13).

The slabs were carefully aligned vertically and horizontally with respect to the test frame so that all specimens were subjected to nearly the same loading and support conditions. To provide a uniform contact surface at the load and support bearing areas, plaster was provided between the slab surface and the steel tubes.

Vertical deflections were measured using two LVDTs (one on each side of the slab) at the central loading point, A

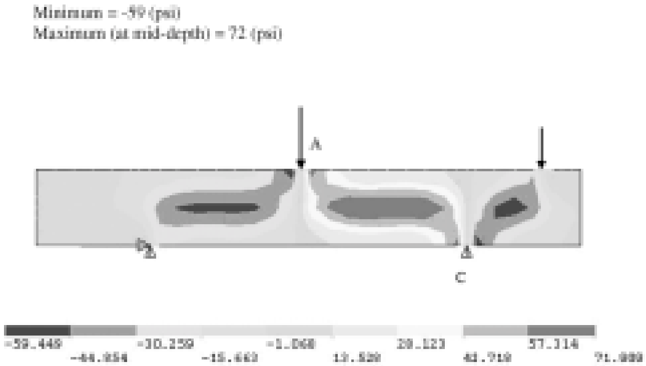

Fig. 14—Shear stress contours in slab units at load of 25 kips (111 kN).

(refer to Fig. 13 and 15). In addition, four high-resolution LVDTs were installed on the slab at Supports B and C (one on each side of the slab) to record deformations from minor, but unavoidable misalignment of the slabs or from the inherent flexibility of the tube/plaster supports. The recorded deformations at the support points were later used to correct the deformation readings at Point A. The nominal range of the LVDTs used at the supports was ±0.005 in. (0.127 mm) and had a resolution of approximately 20×10^{-6} in. (5.08×10^{-4} mm). Further details are discussed elsewhere.[11]

The slabs were all tested under displacement-controlled conditions corresponding to load increments of approximately 10 kips (44.5 kN). The load and vertical deflections were continuously recorded by a data acquisition system.

SLAB TESTS: OBSERVED BEHAVIOR AND TEST RESULTS

All slabs showed similar overall behavior, irrespective of whether they were cast monolithically or in two pours. Typically, the slabs showed linear elastic response with no sign of shear distress (that is, no cracking or slip was observed) until the onset of flexural cracking on the bottom surface. As expected, this event led to nonlinear response, and consequently, a significant reduction in the stiffness of the slabs. Post-cracking behavior was not of interest to this study and therefore all tests were terminated at or shortly after the onset of flexural cracking.

Measured response

Figure 16 shows the applied load and deflection obtained for all six specimens. The load reported in the figure corresponds to the load applied by the actuator during the tests. The deflection values correspond to the measured deflection underneath loading Point A corrected for vertical movement of the slabs at the supports.[11] Also shown in the figure is the maximum shear stress (at mid-depth) in the region of the maximum shear force, calculated assuming an elastic, homogeneous cross section.

The load corresponding to flexural cracking varied with each specimen and it depended on the concrete strength of the bottom pour for specimens cast in two pours. Specimen SMP3 (which was left uncovered and exposed to the laboratory air) had the lowest resistance to flexural cracking, which occurred at a shear stress of approximately 85 psi (0.58 MPa). Flexural cracking for the rest of the specimens occurred at a shear stress of 110 psi (0.76 MPa) or higher.

In Fig. 17, the stiffness prior to flexural cracking of all specimens is compared. The stiffness value reported in the

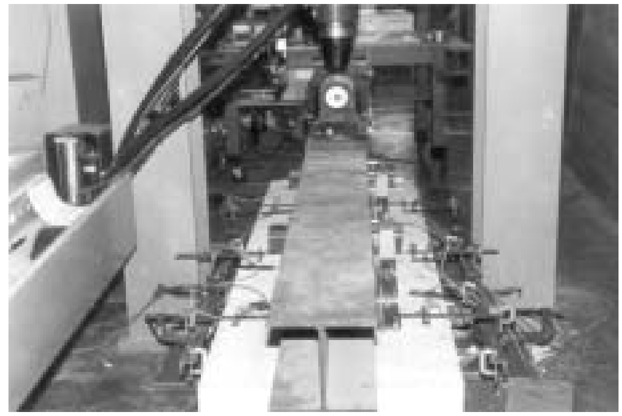

Fig. 15—Overall view of test setup and instrumented slab specimen.

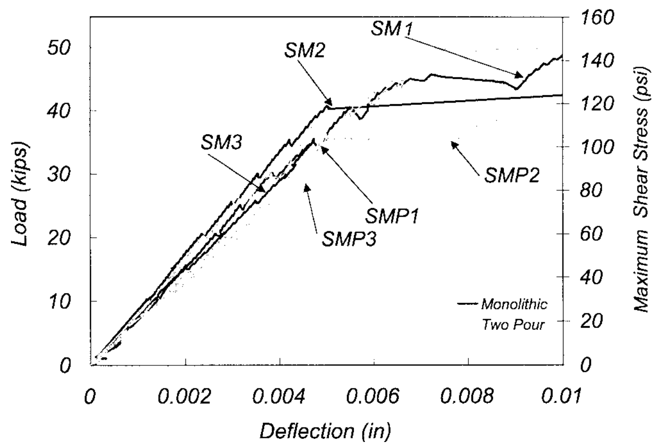

Fig. 16—Applied load and shear stress versus deflection of slab units.

figure corresponds to the slope of the elastic portion of the load deflection curves shown in Fig. 16. On average, the stiffness of the monolithic units is somewhat larger than the stiffness of the units cast in two pours. Specimen SMP2 (cast in two pours) exhibited, however, an elastic stiffness larger than two of the monolithic units (SM1 and SM3). The scatter of these data may be attributed in part to differences in the strength of the concrete at the time of testing.

To account for the differences in the modulus of elasticity, the stiffness of the units was divided by $\sqrt{f_c'}$ and plotted again in Fig. 18. The measured compressive strength of the slab units (average of three cylinders) is shown in Table 4. Figure 18 shows that the average stiffness of the monolithic and that of the units cast in two pours are within 3%. Moreover, Specimen SMP2 (cast in two pours) appears to have the largest stiffness of all, while Units SM3 (monolithic) and SMP1 (cast in two pours) show virtually the same stiffness. Unit SMP3, cast in two pours with a 7-day interval, still shows the smallest stiffness of all. This result is attributed to the poor conditions the joint surface of this unit was cured under. Such conditions prevent complete hydration of the cement paste and reduce the strength and stiffness of the concrete. It must be noted, however, that during the test there was no indication of distress at the joint level of Unit SMP3.

The number of specimens tested in this study is insufficient to provide a statistically reliable measure of the stiffness of the foundations, but the data obtained from both the push-off shear and slab tests do show that a member with an intentionally

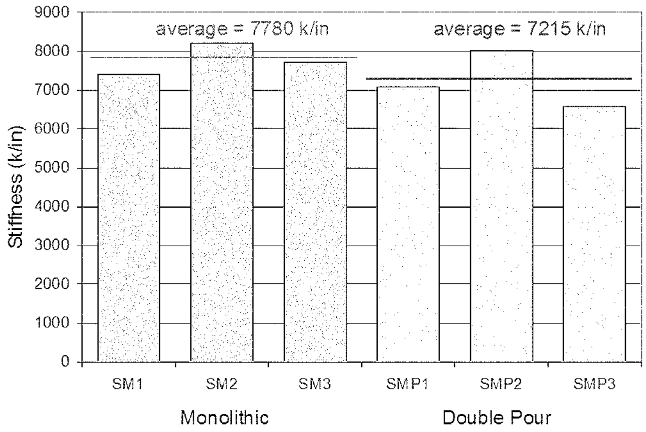

Fig. 17—Comparison of calculated stiffness of slab units.

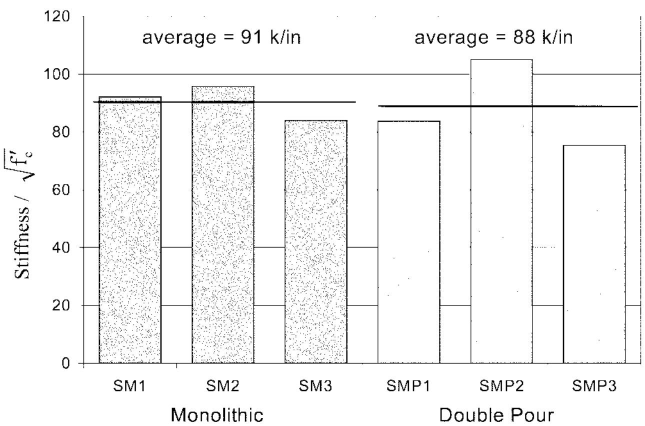

Fig. 18—Comparison of calculated stiffness of slab units normalized to $\sqrt{f_c'}$.

roughened, moist-cured, and properly compacted joint has a stiffness comparable with that of a monolithic unit.

CONCLUSIONS

Based on the results of this study, the following conclusions can be drawn:

1. Compaction of the concrete has a significant effect on the strength of the joint. The data showed that units compacted with a vibrator developed strengths between 50 and 100% higher than those compacted with a steel rod;
2. A wet joint (hardened concrete joint surface saturated with water before casting the top layer) has a detrimental effect on strength. Units with a dry joint were nearly twice as strong as the units cast with a wet joint;
3. A roughened joint surface offers an initial stiffness comparable with that of a monolithic unit up to an average joint shear stress of approximately 400 psi (2.8 MPa) or about $5\sqrt{f_c'}$ ($f_c' \leq 7500$ psi). The procedure employed to roughen the joint surface (broomed or roughened per the ACI code at different angles) does not have an influence on the initial joint stiffness;
4. The use of shear keys resulted in a stiffness and strength comparable with that of the units with intentionally roughened joints;
5. Smooth joints, without a bonding agent, can also offer an initial stiffness comparable with that of the monolithic units. The stiffness, however, can only be maintained for shear stress levels of 250 to 300 psi (1.7 to 2.4 MPa) or 3 to $3.5\sqrt{f_c'}$ ($f_c' \leq 7500$ psi);
6. Smooth joints painted with a bonding agent (resin emulsion) are much more flexible than a monolithic unit and thus are not recommended; and
7. Based on a comparison of the measured deflection of six slab units (three units cast monolithically and three units cast using two layers), it is concluded that slabs cast in two pours can offer the same stiffness of those cast monolithically. These results hold true provided that the concrete is properly compacted, and that the joint surface is intentionally roughened and moist cured until the top layer is cast. Thus, foundations cast in multiple pours with construction joints similar to those used here are expected to perform as well as a foundation cast in a single pour.

Table 4—Compressive strength of slab units

Specimen	f_c^b, psi	f_c^t, psi	f_c', psi
SM1	—	—	6460
SM2	—	—	7365
SM3	—	—	8409
SMP1	7525	6749	7137
SMP2	5937	5724	5830
SMP3	6985	8135	7560

Note: 1 psi = 0.0069 MPa.

RECOMMENDATIONS

The joint surface of the push-off shear specimens and also those of the slab units were prepared under laboratory conditions, that is, the surface was thoroughly cleaned and carefully monitored to ensure that it was properly cured and compacted. Field conditions may, however, deviate from those used in the laboratory that may adversely affect the performance of the foundations cast in multiple pours. As evidenced by the slab unit that was left uncovered and exposed to the laboratory air (not moist cured), poor curing conditions will reduce the stiffness and strength of the unit. The data obtained in this and past studies show that moist curing is key to the successful performance of a construction joint.

In summary, it is recommended that joint surfaces be carefully cleaned to remove all dust and loose particles. The joint should be moist cured for a minimum of 24 h, but it should not have freestanding water when the top layer is poured. It may be possible to allow the top layer to be cast at a later time (after 48 or 72 h, for example), provided that the joint surface is kept clean and moist cured during this time.

Although shear keys can provide a joint comparable with that of a monolithic unit, their fabrication is labor intensive, costly, and impractical considering that a large number of keys would be required over the joint surface in large foundations. Thus, their use is not recommended for foundations cast in multiple pours.

Only horizontal construction joints were cast and tested in this study. Vertical joints or joints at an angle with the horizontal may perform differently due to segregation of the coarse aggregate. Such joints may require different preparation and construction procedures to ensure proper bond along the joint.

ACKNOWLEDGMENTS

The study presented in this paper was funded and supported by Giddings and Lewis, Inc., whose support is gratefully acknowledged. Special thanks are extended to E. Ruethling and W. J. Zippel during the testing of the push-off shear and slab specimens. The assistance of W. G. Lang and J. W. Dreger during the setup, instrumentation, and testing of the specimens is gratefully acknowledged. The opinions, findings, and conclusions reported herein are solely those of the authors and do not necessarily represent the views of the sponsor.

NOTATION

f_c^b	=	concrete compressive strength in bottom block
f_c^t	=	concrete compressive strength in top block
f_c'	=	average concrete compressive strength
V_u	=	applied shear load
v_u	=	applied shear stress = applied shear force divided by nominal shear area
$\Delta_{1\text{-}1}$	=	displacement in direction 1-1 of rosette
$\Delta_{2\text{-}2}$	=	displacement in direction 2-2 of rosette
$\Delta_{3\text{-}3}$	=	displacement in direction 3-3 of rosette

REFERENCES

1. Tynes, W. O., and McCleese, W. F., "Investigation of Methods of Preparing Horizontal Construction Joints in Concrete: Evaluation of High-Pressure Water Jet and Joint Preparation Procedure," *Technical Report* No. 6-518, Report 4, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., Aug. 1973, 22 pp.

2. Waters, T., "A Study of the Tensile Strength of Concrete Across Construction Joints," *Magazine of Concrete Research*, V. 6, No. 18, Dec. 1954, pp. 151-153.

3. Hanson, N. W., "Precast-Prestressed Concrete Bridges 2: Horizontal Shear Connections," *Development Department Bulletin* D 35, Portland Cement Association, 1960, pp. 38-58.

4. Bass, R. A.; Carrasquillo, R. L.; and Jirsa, J. O., "Shear Transfer Across New and Existing Concrete Interfaces," *ACI Structural Journal*, V. 86, No. 4, July-Aug. 1989, pp. 383-393.

5. Kosmatka, S. H., and Panarese, W. C., "Design and Control of Concrete Mixtures," *Engineering Bulletin*, 13th Edition, Portland Cement Association, 205 pp.

6. Valluvan, R., "Issues Involved in Seismic Retrofit of Reinforced Concrete Frames Using Infilled Walls," PhD dissertation, the University of Texas at Austin, Austin, Tex., Dec. 1993, 300 pp.

7. Paulay, T.; Park, P.; and Phillips, M. H., "Horizontal Construction Joints in Cast-in-Place Reinforced Concrete," *Shear in Reinforced Concrete*, SP-42, V. 2, American Concrete Institute, Farmington Hills, Mich., 1974, pp. 599-616.

8. Seible, F., and Latham, C. T., "Horizontal Load Transfer In Structural Concrete Bridge Deck Overlays," *Journal of Structural Engineering*, ASCE, V. 116, No. 10, Oct. 1990, pp. 2691-2710.

9. Hofbeck, J. A.; Ibrahim, I. O.; and Mattock, A. H., "Shear Transfer in Reinforced Concrete," ACI JOURNAL, *Proceedings* V. 66, No. 2, Feb. 1969, pp. 119-128.

10. ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI 318-02) and Commentary (318R-02)," American Concrete Institute, Farmington Hills, Mich., 2002, 443 pp.

11. Djazmati, B.; Pincheira, J. A.; and Zippel, W. J., "Shear Stiffness and Strength of Horizontal Construction Joints," *Final Report to Giddings and Lewis, Inc.*, University of Wisconsin-Madison, Madison, Wis., Apr. 2000, 129 pp.