INVESTIGATION OF GROUTED PRECAST CONCRETE WALL CONNECTIONS AT SUBFREEZING CONDITIONS

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ABSTRACT

The effect of exposing grouted precast wall connections to subfreezing curing temperatures at early-age was explored in this study. In cold weather construction, heating of the surrounding environment of grouted precast wall connections is usually conducted for short periods of time. Hence, subfreezing conditions can affect the strength of the grout and the bond strength of the connection, which can ultimately compromise the integrity of the structure. In this study, grout specimens typical of that used in precast wall construction were cured at ambient conditions for one day, and then placed in an environmental chamber at subfreezing temperatures (-10°C and -20°C). The compressive strength development of the grout was monitored, and the bond strength of grouted connections cured at cold temperature were quantified and compared to that of specimens cured at ambient temperature. The bond was investigated on 25M deformed steel bars, which is the typical size used in precast concrete wall grouted connections. Test results indicate a reduction in grout strength and the need for a longer embedment length when early-age curing is conducted at subfreezing conditions.

Keywords: Precast concrete, Bond strength, Grout, Cold weather curing, Development length.

1. INTRODUCTION

Precast concrete load bearing wall panels have become a popular choice for the construction of low-, medium-, and high-rise buildings in North America. This structural system is advantageous because of its cost-effectiveness, ease and speed of erection, and high quality control achieved at the manufacturing plant. However, a major concern when using precast wall panels is the reliability and ease-of-installation of their connections because this directly influences the structure's integrity and stability. A commonly used connection type is the grouted dowel connection, where a reinforcing bar protruding from the lower wall panel is grouted into a corrugated duct in the upper wall panel as illustrated in Figure 1. The strength of this connection depends on the bar size, embedment length, and the bond developed between the grout and the reinforcing bar.

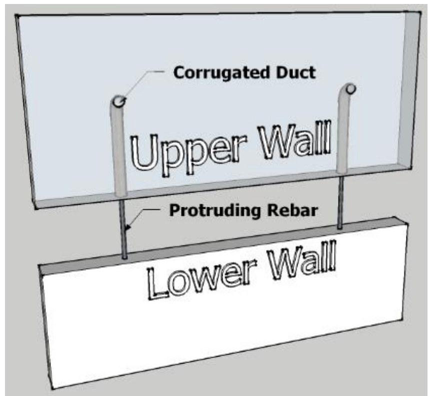

Figure 1: Typical grouted dowel connection.

Canada has a cold climate where the subfreezing temperatures can last for long periods of time. The ACI Committee 306R-10 defines cold weather as a period of three or more successive days when the average daily air temperature drops below 4°C and does not exceed 10°C for more than one-half of any 24-h period (ACI Committee 306, 2010). Concrete has to be protected against freezing until it reaches a compressive strength of 3.5 MPa; if concrete is exposed to freezing before it reaches 3.5 MPa, its compressive strength will be reduced significantly. Precast concrete construction usually continues throughout this cold weather using special procedures. In a typical assembly of precast wall panels during cold weather, the entire floor is typically blanketed and heated for one day, while the grout is poured into the connection. The wall connection is then exposed to the subfreezing temperatures before the grout is fully cured; this can significantly affect the bond strength between the grout and reinforcing bar, possible compromising the overall integrity of the connection.

Research has shown that concrete can gain strength when cured in cold weather temperatures (up to -5°C) (Nassif & Petrou, 2013). This was largely attributed to the heat of hydration raising the internal temperature above freezing temperatures for approximately three days. However, Nassif & Petrou (2013) cast relatively larger volumes of concrete (750x750x300 mm slabs) than what is used for grouted dowel connections. Hence, it is unlikely that the heat of hydration would allow for strength gain in grouted connections for up to three days. Karagol et al. (2015) cured concrete cylinders in subfreezing temperatures from -5°C to -20°C and showed that very little strength gain was achieved for normal concrete. At 90 days, the concrete compressive strength was 8.5 MPa and 10.8 MPa for -20°C, and -10°C, respectively (Karagol, Demirboga, & Khushefati, 2015). Based on this research it is expected that some strength gain for the grouted dowel connection will be achieved once heating is removed. Yet, this needs dedicated testing and quantitative assessment.

The present study examines the effects of early-age exposure to subfreezing temperatures of grouted precast wall dowel connections using 25M deformed steel bars. The connections were exposed to subfreezing temperatures (-10°C) after one day of ambient curing, which is similar to winter construction conditions. Varying embedment lengths and grout strength development were examined and compared from ambient to -10°C curing conditions. Grout strength development was also monitored for a more severe temperature of -20°C.

2. EXPERIMENTAL PROGRAM

2.1 Test Specimens

Pullout test specimens were designed to represent a bar grouted into a section of a precast concrete wall panel. Each specimen consisted of a precast concrete block having 203.2 mm x 203.2 mm in cross-sectional area, and a height of 406.4 mm. Thin-walled steel corrugated ducts were cast into the center of the block and had a diameter of 76.2 mm. The top and bottom of the test bars were wrapped with plastic to debond sections of the bar so that the embedment length, L_d, would lie in the middle of the specimen. Such a debonding was done to avoid compression stresses induced from the pullout testing, which is known to affect the bond behaviour of the bar (ACI Committee 408, 2003). The test bars were 25M deformed steel, placed concentrically in the duct and grouted from the top (passive) end. The bar extended 25 mm above the concrete block to measure slip during testing.

All specimens were first cured for 24 hours at an ambient lab temperature of 23°C. The specimens exposed to subfreezing temperatures were then moved to an environmental chamber preconditioned at the specified temperature (-10°C or -20°C) and cured for an additional six days (for a total curing time of seven days) then tested. Control specimens were left at ambient temperature in the lab for seven days and then tested. The internal temperature of the grout was monitored for both temperatures, with readings taken every 10 minutes for 7 days. The specimens used for monitoring the temperature were not used for strength testing to ensure the temperature probe did not affect the strength results.

2.2 Material Properties

The specimens were cast using self-consolidating concrete having a 28 days compressive and tensile strengths of 50.6 MPa, and 4.9 MPa, respectively. Commercially available non-shrink grout used in precast wall connections with a specified compressive strength of 30 MPa at 7 days was used. One 25 kg bag was mixed with 3.75 L of water to achieve a fluid consistency. The tested reinforcing bars had a specified yield strength of 400 MPa.

2.3 Test Setup

After 7 days of curing, the specimens were placed atop the active pulling end of an open loop Tinius Olsen testing machine with a maximum capacity of 530 kN. A 215.9 mm x 215.9 mm steel bearing plate with a square internal void was used for distributing the load to a 25.4 mm wide outer edge of the concrete block to further help avoid compression stresses induced from the pullout testing as previously mentioned. One strain based linear variable displacement transducer (LVDT) with a 25 mm gauge length was placed on the unloaded end of the bar to measure its slip. The crosshead movement, which represents the elongation of the bar, was measured by another LVDT with a gauge length of 150 mm. The data acquisition system recorded measurements at a rate of 1 reading per second. A depiction of the test setup is shown in Figure 2. The test bar was loaded monotonically in tension, at a rate of approximately 70MPa/min. The test was stopped once either a bar fracture occurred, or the LVDT measuring slip reached its gauge length.

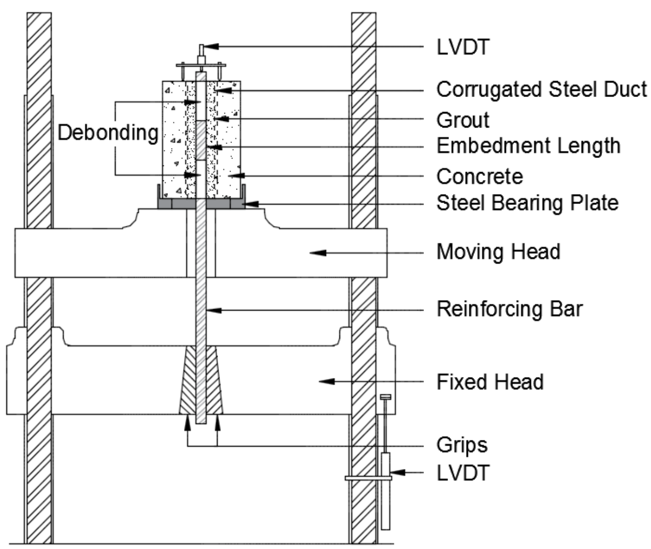

Figure 2: Test Setup.

3. RESULTS AND DISCUSSION

3.1 Grout Properties

The static modulus of elasticity, tensile strength, and compressive strength were evaluated and compared at ambient and -10°C; with the compressive strength also tested at -20°C. The test results from an average of two specimens are presented in Table 1 and discussed below.

Table 1: Grout Strength Development

Temperature	Age (days)	Compressive Strength (MPa)	Tensile Strength (MPa)	Young's Modulus (MPa)	Poisson's Ratio
Ambient	1	19.2	3.0	N/A	N/A
	3	33.7	3.3	N/A	N/A
	7	38.4	4.5	20712	0.2285
	28	39.3	6.3	22713	0.2346
-10°C	1	20.1	3.0	N/A	N/A
	3	26.9	3.2	N/A	N/A
	5	32.4	N/A	N/A	N/A
	7	32.4	3.2	19971	0.2315
	28	32.5	3.7	20563	0.2322
-20°C	1	22.0	N/A	N/A	N/A
	3	30.1	N/A	N/A	N/A
	5	32.3	N/A	N/A	N/A
	7	32.3	N/A	N/A	N/A

3.1.1 Compressive Strength

The compressive strength of the grout was monitored for up to 28 days at ambient and -10°C, and up to 7 days for a more severe temperature of -20°C. The grout had a compressive strength of approximately 20 MPa after one day of ambient curing, and continued to gain strength even after being exposed to subfreezing temperatures, as seen in Figure 3. It is apparent that there is little to no long-term strength gain between 7 and 28 days for both the ambient and subfreezing conditions. The severity of the subfreezing temperature does not have an effect on the compressive strength of the grout; similar trends in strength gain were observed for both the -10°C and -20°C conditions with a final strength of about 32 MPa reached at 5 days, achieving 83% of the strength of the ambient specimens. Although the grout is weaker when cured in subfreezing temperatures, Karagol et al. (2015) and Nassif & Petrou (2013) have shown that the compressive strength of concrete cured at freezing temperatures, then brought to room temperature for long periods (275 days and 240 days respectively) converges, and can slightly increase compared to concrete constantly cured at ambient temperatures.

Other researchers have observed similar compressive strength gain when cured at subfreezing temperatures, and have largely attributed it to the internal temperature of concrete staying above freezing for an extended period of time due to the heat of hydration (Çullu & Arslan, 2013; Karagol et al., 2015; Nassif & Petrou, 2013). Figures 4(a) and 4(b) show the internal grout temperature at -10°C, and -20°C, respectively; data is only shown up to 3 days for clarity, but the temperature remained constant up to 7 days. It can be observed that the heat of hydration did not significantly influence the internal temperature once placed in the temperature chamber. It is believed that once the specimens were exposed to subfreezing temperatures, the remaining water is in small enough pores that it does not freeze, thus allowing for some strength gain to still occur.

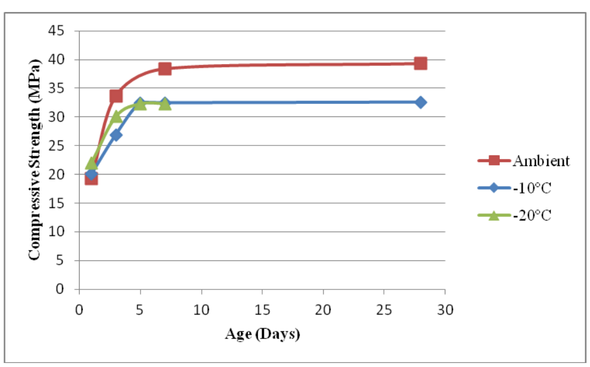

Figure 3: Grout Compressive Strength Development

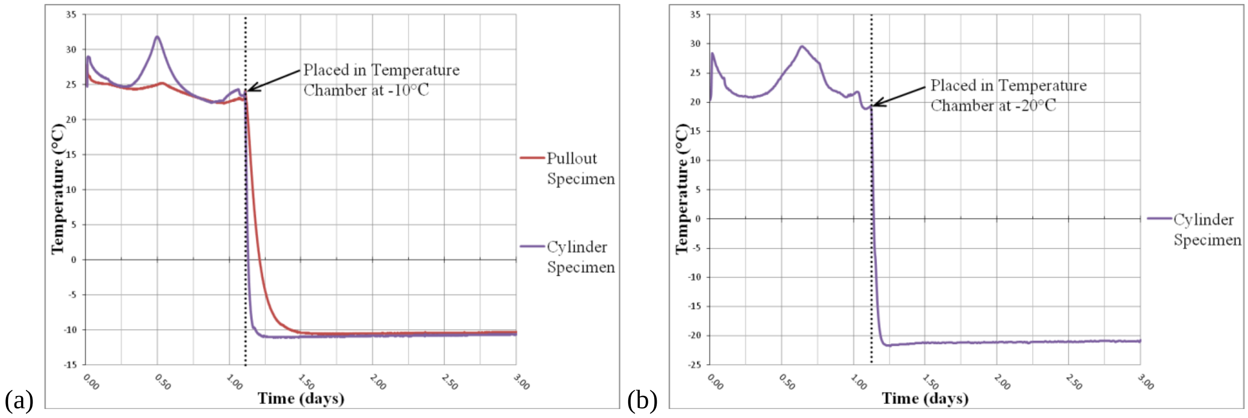

Figure 4: Internal Grout Temperatures for different curing temperatures: (a) -10°C; (b) -20°C.

<u>3.1.2 Tensile Strength</u>

There was no significant tensile strength gain of the grout cylinders once the grout was exposed to subfreezing temperatures. The specimens cured in subfreezing temperatures reached tensile strengths 41% lower than that of the control specimens stored at ambient temperatures, when comparing 28 day strengths. At ambient conditions, the tensile strength continued to increase up to a maximum of 6.3 MPa at 28 days. It is known that the tensile strength is related to the compressive strength; this suggests that if the grout is later exposed to ambient temperatures and gains further compressive strength, matching that of the control specimen (as previously mentioned), the tensile strength will also increase. The lack of tensile strength gain can be attributed to the lower compressive strength rather than to a change in microstructure.

<u>3.1.3 Modulus of Elasticity</u>

The modulus of elasticity was recorded at 7 and 28 days. It can be observed that there was greater modulus increase at ambient conditions between 7 and 28 days than that at subfreezing conditions. A similar trend was noted by Çullu & Arslan (2013). As discussed above, the lower value of the modulus of elasticity can be attributed to the lower value of the compressive strength of the grout.

3.2 Pullout Specimens

The test results of the 10 pullout specimens are shown in Table 2. For each set of parameters, two specimens were tested. Specimens were labeled as follows: the first digit refers to its embedment length relative to its bar diameter (d_b) (6, 12, and 16d_b), the second character refers to curing condition (A for ambient and 10 for -10°C), with the final digit referring to specimen number. For example, 6-10-2 refers to a specimen with an embedment length of 6d_b cured at -10°C and is the second of its set.

The yielding load, P_y, and displacement, δ_y, are taken as the point where the specimens' stiffness degraded suddenly after the elastic response. The ultimate displacement, δ_u, and maximum slip, s_{max}, correspond to the point where failure occurs and the ultimate load, P_u, is taken as the maximum recorded load. The maximum bond stress, u_{max}, of the bar equals the ultimate load divided by the total surface area embedded in the grout (Eq.1).

[1] $$u_{max} = \frac{P_u \times 10^3}{\pi d_b l_b}$$

Table 2: Pullout Test Results

Specimen	P_y (kN)	P_u (kN)	f_{bu} (MPa)	δ_y (mm)	δ_u (mm)	s_{max} (mm)	u_{max} (MPa)	R_s	μ_Δ	Failure Mode
6-A-1	198	250.8	510.9	16.3	54.8	0.96	21.0	1.28	3.4	Pullout
6-A-2	192.2	243.6	496.3	13.5	43.8	1.02	20.4	1.24	3.2	Pullout
6-10-1	215.8	235.9	480.6	16.2	22.8	0.92	19.7	1.20	1.4	Pullout
6-10-2	211.5	217.6	443.3	14.0	21.7	0.60	18.2	1.10	1.6	Pullout
12-A-1	194.4	273.1	556.4	12.4	110.4	0.05	11.4	1.39	8.9	Fracture
12-A-2	196.2	273.4	557.0	12.7	104.0	0.05	11.4	1.39	8.2	Fracture
12-10-1	205.9	277.6	565.5	11.3	43.8	0.82	11.6	1.41	3.9	Pullout
12-10-2	200.9	271.1	552.3	12.1	72.8	0.97	11.3	1.38	6.0	Pullout
16-10-1	210.2	293.1	597.1	13.2	113.6	0.05	9.2	1.49	8.6	Fracture
16-10-2	204.6	290.7	592.2	12.5	100.5	0.11	9.1	1.48	8.1	Fracture

3.2.1 Failure Mode

No cracking of the concrete was observed for all tested specimens. Since the bars were wrapped, no conical grout failure at the active end occurred, as observed by other researchers investigating similar connections (Steuck, Eberhard, & Stanton, 2009). Bar fracture is the preferred failure mode since it provides a higher tensile capacity, along with a more ductile response, this was achieved by an embedment length of $12d_b$ at ambient conditions, and by a longer embedment length of $16d_b$ when cured at -10°C. A typical bar fracture failure can be observed in Figure 5(b). All other specimens failed in bar pullout due to a bar-grout bond failure, which can be observed in Figure 5(a).

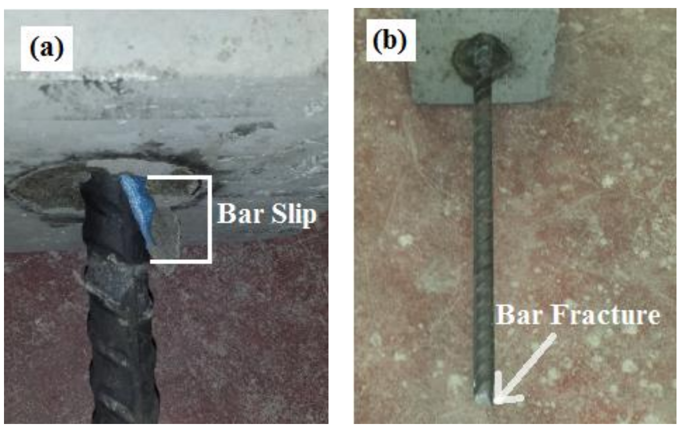

Figure 5: Typical Failure Modes: (a) bar pullout (specimen 12-10-1); (b) bar fracture (specimen 16-10-1).

3.2.2 Bond Behaviour

The load-displacement responses for all specimens are shown in Figure 6. The reported displacement represents the elongation of the bar tested. The peak bar stresses, f_{bu}, are reported in Table 2. The specimen failing at the lowest load, 6-10-2, achieved f_{bu}= 443 MPa which exceeds the 400 MPa yield strength of the bar. This indicates that the response for all specimens was dominated by bar yielding, as opposed to solely the bond properties governing pullout. Inelastic elongation due to bar yielding reduces the bar diameter, partially disengaging the bar from the surrounding grout, and therefore reducing the bond capacity. Therefore, further testing should be done on shorter lengths that remain elastic throughout the duration of the test to obtain a better understanding of this connection behaviour.

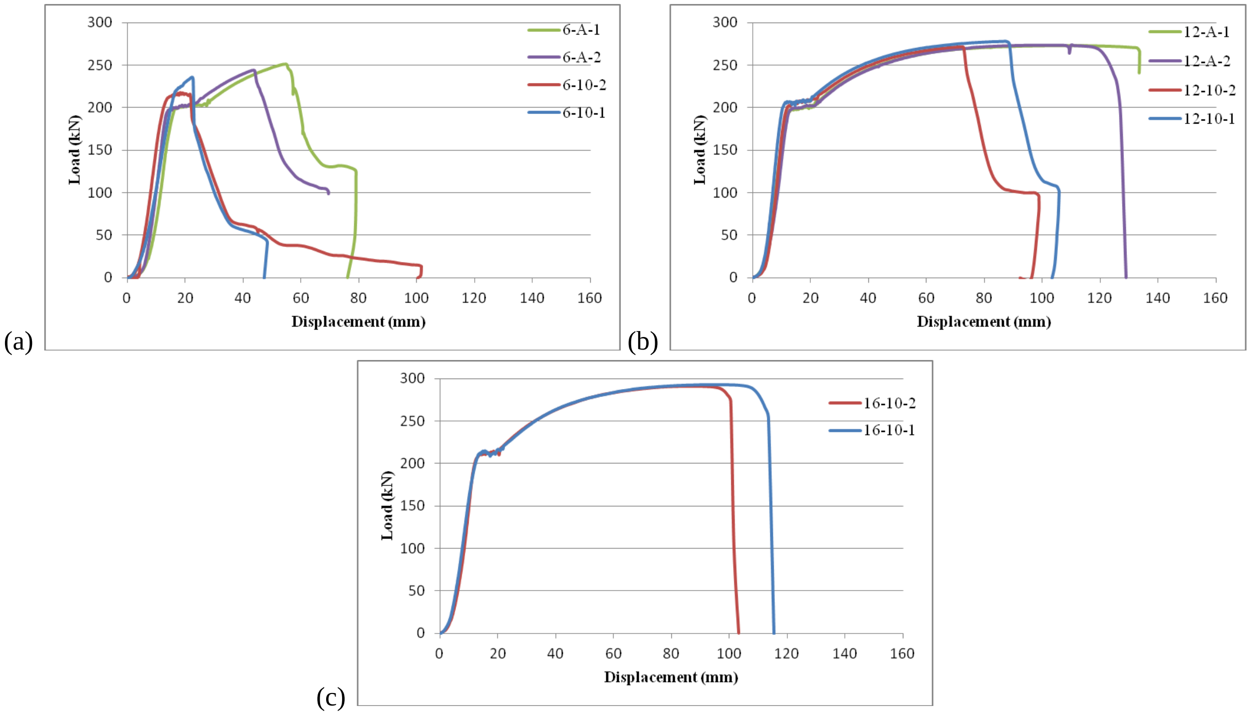

Figure 6: Load-displacement curves comparing connection specimens cured at ambient and -10°C: (a) $L_d = 6d_b$; (b) $L_d = 12d_b$; $L_d = 16d_b$.

The bond stress and slip responses of all specimens are shown in Figure 7. The bar slip in specimen 6-A-1 was not fully recorded due to an equipment malfunction. It can be observed in Figure 7(a) that the bond-slip curves of all bar pullout failure specimens have comparable shape but reach different bond stress peaks, u_{max}. Each bond-slip curve can be characterized by three regions: 1) the ascending branch, which is from the start of the test until u_{max} is reached; 2) a region of sudden drop to the residual bond stress; and 3) a final region in which the stress dropped gradually until the test is ended. The residual bond stress indicates the additional resistance of the bar to the pullout load after the initial failure has occurred. The response post-peak of the bond-slip curve is predominantly governed by the frictional resistance of the bar, as well as the additional resistance provided by the undamaged section of the bar that enters the embedded zone when the bar slips. It is interesting to note that for the longer embedment length ($L_d = 12d_b$) there was almost zero slip until near peak bond stress was reached, whereas for $L_d = 6d_b$ the slip gradually increased until failure as can be observed in Figure 7(b). This may be caused by the major yielding that occurred in those bars before failure; they elongated on average an extra 10 mm compared to 6-A-1 and 6-A2. This could lead to a significant reduction in bar-cross section, which then disengaged from the grout and failed suddenly, experiencing little to no slip beforehand.

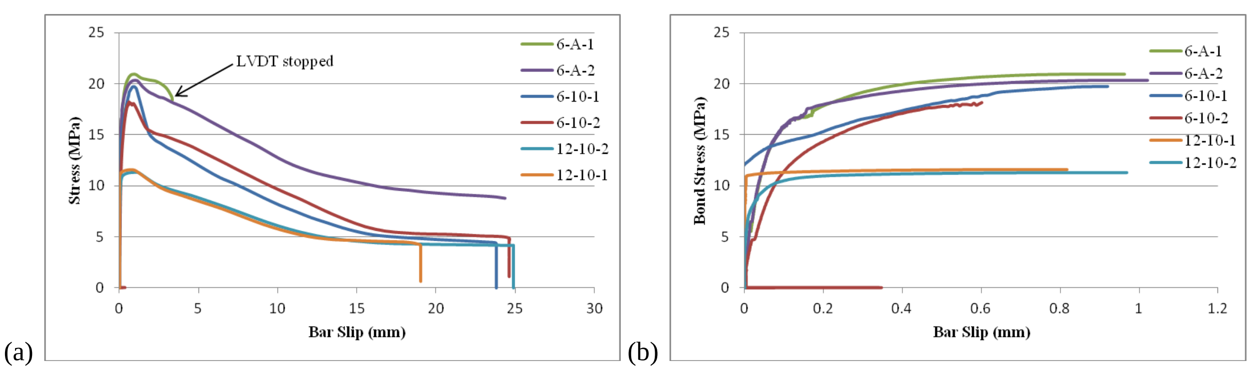

Figure 7: Bond stress-slip curves: (a) Full response; (b) Ascending branch.

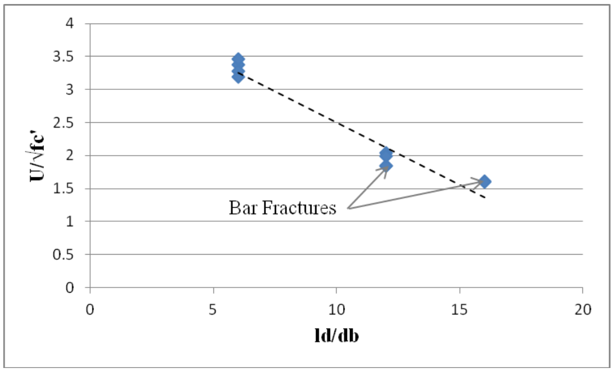

Figure 8: Normalized bond stress vs. embedment length.

Previous research reported a linear relationship between the bond stress, u_{max}, and the square root of the compressive strength of concrete/grout, $\sqrt{f_c'}$, (Einea, Yamane, & Tadros, 1995; Untrauer & Henry, 1965). The average change in bond stress from ambient to -10°C for $L_d = 6d_b$ is 8.3%, and the change in the square root of compressive strength from ambient to -10°C is 8.1%, this shows there is a clear nearly linear relationship present. This can also be observed in Figure 8, which shows the relationship between $u_{max}/\sqrt{f_c'}$ and embedment length in terms of d_b, for all specimens.

Two ratios were used to determine the suitability of these connections for the given embedment lengths and grout strengths. The first is the strength ratio, R_s, which is the ratio of the ultimate bar stress, f_{bu}, to the specified yield strength of the bar (f_y = 400 MPa) as observed in Equation 2.

[2] $$R_s = \frac{f_{bu}}{f_y}$$

This connection is used in place of mechanical connectors which require a tensile capacity of 1.25 the specified yield strength of the bar (ACI Committee 318, 2014), therefore the same strength criteria was used to deem it suitable. All of the tested specimens reached a strength ratio greater than 1.25, except both specimens with $L_d = 6d_b$ cured at subfreezing conditions, and one with $L_d = 6d_b$ cured at ambient conditions, which achieved $R_s = 1.24$. This demonstrates that even with some pullout failures this connection is sufficiently strong to fully develop the bar, reaching strength ratios as high as 1.49 when bar fracture was achieved.

The second is the ductility ratio, μ_Δ, which is the ratio between the ultimate displacement, δ_u, and the yielding displacement, δ_y (Eq. 3).

[3] $$\mu_\Delta = \frac{\delta_u}{\delta_y}$$

Research has shown that precast concrete connections in low to moderate seismic regions require a ductility ratio of 4.0 (Soudki, Rizkalla, & Leblanc, 1995). For all specimens with $L_d = 6d_b$ the ductility was too low. Specimens 12-10-1, and 12-10-2, demonstrated that a high level of ductility can be achieved when a pullout failure occurs with ductility ratios of 3.9, and 6.0, respectively. When the bar fully developed and fractured, a very high ductility ratio greater than 8.0 is reached. Using section 25.4.2.2 in ACI 318-14 a development length of $40d_b$ is required. However, through testing it is shown that this connection can yield the bar at $6d_b$ and fracture at $12d_b$, and $16d_b$, for ambient and subfreezing conditions, respectively.

The ratios in Eqs. 2 and 3 demonstrate that specimens with embedment lengths equal to or greater than $12d_b$ are suitable for precast concrete connections, regardless of the curing conditions.

4. CONCLUSUIONS

The objective of this study was to examine the effect of early-age exposure of grouted precast wall connections to subfreezing temperatures. Grout strength development was monitored at three different conditions: ambient up to 28 days, one day at ambient then at -10°C up to 28 days, and one day at ambient then at -20°C up to 7 days. The bond strength of this connection was compared for specimens cured at ambient and -10°C exposure for 7 days.

The severity of the subfreezing temperature did not have a significant effect on the grout strength, with a final compressive strength of 32 MPa reached at 5 days for both -10°C and -20°C exposure. There was a reduction in strength (17%) between ambient and subfreezing curing conditions. The subfreezing conditions appear to have the largest impact on the tensile strength of the grout. There was a 41% reduction in tensile strength when cured at subfreezing conditions: achieving only 3.7 MPa at -10°C, compared to 6.3 MPa at ambient temperature. The effect of long-term strength gain after re-exposure to ambient temperatures was not examined in this study, but should be tested in future work. The reduction in bond strength for $L_d = 6d_b$ between ambient and -10°C was found to be approximately equal to the reduction in the square root of the grout compressive strength, by 8.3%, and 8.1%, respectively. A near linear relationship was found between the embedment length and $u_{max}/\sqrt{f_c'}$.

The results of the 10 pullout tests performed show that this connection can develop a bar with a much shorter length than what is suggested by current code equations. Experimental testing showed that yielding can be achieved by as little as $6d_b$ and bar fracture can be achieved by $12d_b$, and $16d_b$, at ambient, and subfreezing conditions, respectively, for monotonic loading. Further testing is required to determine the appropriate development length of this connection under cyclic loading.

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