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ARTICLE

Potential application of lightweight foamed concrete in the Malaysian construction industry



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Abstract

The significant contribution of the construction sector to Malaysia's economic development is widely recognized. However, the organization continues to maintain its adherence to conventional methodologies, thereby hindering its advancement toward achieving excellence on a global scale. The significance of incorporating innovative construction techniques, advanced building materials, and digital technologies cannot be underestimated in fostering the development of Malaysian enterprises operating within the construction industry. The application of lightweight foamed concrete (LFC) offers a potentially advantageous substitute for conventional concrete owing to its ability to reduce the weight of structures and foundations, enhance energy efficiency, and lower costs related to manufacturing, labor, and transportation. This study provides a thorough examination of LFC, including its constituent components, manufacturing procedure, and material properties. Considering the importance of LFC in the field of engineering construction, this study incorporates a comprehensive evaluation of the current practices of LFC implementation in construction and civil engineering works. This article addresses certain deficiencies and technical constraints of LFC, while also exploring potential avenues for improving its utilization in the Malaysian construction industry.

1.0 Introduction

The construction sector is widely recognized as a significant contributor to the economic development of Malaysia. Nevertheless, the industrial players persist in adhering to traditional methods, impeding their progress toward achieving global excellence. The adoption of innovative construction techniques, advanced building materials, and digital technologies is imperative for the growth of Malaysian industry players. It is anticipated that the construction sector in Malaysia will experience a robust resurgence in 2023, owing to the revitalization of significant infrastructure construction undertakings. The predicted rise in the approval of residential projects, coupled with this factor, is projected to facilitate the expansion of this sector. The Malaysian construction industry looks set to experience an upsurge and conclude on a positive note. However, consultants and contractors are currently grappling with pressing challenges pertaining to building materials and labor. In the short term, it seems likely that the escalating material costs will have a negative impact on the profit margins of construction industry participants within the nation. The Twelfth Malaysia Plan (12MP), which encompasses the period from 2021 to 2025, is a strategic blueprint designed to facilitate the realization of a prosperous, inclusive, and sustainable Malaysia. The plan signifies the initiation of a novel stage in Malaysia's developmental trajectory, commonly referred to as the Shared Prosperity Vision 2030 (WKB 2030). The construction industry is a substantial subcategory that is expected to make a significant contribution towards the attainment of the overarching objectives stated in the 12th Malaysia Plan (12MP). Given the increasing prevalence of global warming, the construction sector is actively seeking an alternative to conventional normal-weight concrete owing to its substantial dead weight and thermal conductivity. Employment of ground-breaking building materials like

Lightweight Foamed Concrete (LFC) would contribute positively towards the enhancement of the construction sector in Malaysia in line with the request of 12MP. The utilization of LFC in construction has been widespread in various nations including Libya, Turkey, Brazil, Germany, the United Kingdom, and Singapore. Figure 1 shows the employment of LFC blocks in Malaysian projects and Figure 2 demonstrates a LFC wall panel being constructed in a project in Surabaya, Indonesia.

Figure 1. Using LFC blocks in a Malaysia

Figure 2. LFC panel used in Indonesia

Considerable advancements have been achieved in the field of LFC application in recent decades. The utilization of LFC for tunnel grouting has been extensively adopted in Canada [1]. The present manuscript provides a succinct overview of the historical evolution and progress of LFC, along with an analysis of prospective viewpoints. This paper provides an elaboration on the engineering properties and benefits of LFC in construction engineering. The aim of this assessment is to emphasize the engineering characteristics, material properties, and practical uses in the construction and infrastructure sectors.

2.0 Constituent materials

2.1 Binders

The preeminent binding agent utilized in LFC is cement. The various types of cement utilized in LFC include OPC, rapid-hardening cement, calcium sulfoaluminate cement, and high alumina cement. These types of cement can be incorporated in the binder content within a range of 20% to 100%, [2]. Nonetheless, various additional elements, including but not limited to silica fume, lime and fly ash, have the potential to serve as substitutes for cement, with replacement percentages ranging from 5% to 70%. Supplementary materials are utilized to improve the uniformity of mix design, support long-term strength, and mitigate cracks propagation. Various supplementary materials can impact the characteristics of LFC in distinct ways.

2.2 Fillers

In the production of LFC, it was generally observed that coarse aggregate was not utilized. Numerous studies have been documented regarding the substitution of conventional sand with alternative finer materials as a binding agent in LFC. A variety of additives, such as silica fume, fly ash, limestone powder, granulated blast furnace slag, and fly ash, have been widely employed to improve the mechanical characteristics of LFC [3]. The utilization of these additives confers several benefits, including the improvement of the mix proportion design, enhancement of long-term strength, and reduction of costs. In addition, the manufacturing process of high-density LFC commonly involves the incorporation of different types of fine aggregates, such as fine sand, recycled glass powder, and surface-modified chips.

2.3 Water

The essential amount of water for the constituent material is contingent upon the mortar body's composition, consistency, and stability. Lower water content in the mixture can lead to increased hardness, resulting in a higher likelihood of bubble bursting [4]. Conversely, the higher water content can cause the LFC mix to become too thin to effectively contain bubbles, leading to their separation from the mixture. As per the guidelines of the ACI, it is suggested that the water used for mixing should be potable and recently sourced. In certain instances, the substitution of mixed water may be feasible with water of comparable efficacy procured from urban sectors, provided that the strength of LFC attains 90% during a designated period of curing.

2.4 Surfactant

The production of LFC involves a significant constituent known as surfactant, which serves as a foaming agent. This component can be manufactured through either a pre-foaming process or a mixed-foaming process. The pre-foaming process involves the individual preparation of the base mix and stable aqueous foam, which are subsequently combined and mixed thoroughly. Conversely, the mixed foaming process entails the direct addition of the foaming agent to the base mix. The incorporation of foam during the mixing process induces the formation of a cellular microstructure within the concrete. Various surfactants such as aluminium powder, CaH_2 and TiH_2, were employed in the mixed foaming procedure [5].

2.5 Additives

Frequently employed additives comprise water-reducing agents, waterproofing agents, retarders, coagulation accelerators, and others. Plasticizers are commonly acknowledged to improve compatibility. Water reducers are commonly utilized in fresh concrete to enhance its performance by decreasing fluidity and plasticity. Research has indicated that the use of water reducers does not significantly affect concrete segregation [6].

2.6 Fibers

Various types of fibers are incorporated into LFC to enhance their strength and minimize their shrinkage. The materials commonly utilized in the mixture include polypropylene, a combination of glass and polypropylene, ramie, palm oil, steel, coir, wastepaper cellulose, carbon, and polypropylene. The typical range for the introduction of these materials is between 0.1% and 2.0% of the volume fraction of the mixture.

3.0 Properties of LFC

3.1 Fresh state properties

The rheological properties of the LFC mix in its initial state are characterized by a self-compacting and flowing behaviour. Hence, it is crucial to take into account multiple parameters, including stability, consistency, rheology, workability, and compatibility. The parameters of the system are primarily influenced by various factors, including the water-to-cement ratio, supplementary materials, fine and coarse aggregate, plasticizers, as well as the quantity and characteristics of foam agents incorporated into the base mix.

3.1.1 Rheology and Consistency

The preliminary assessments of fresh LFC properties usually include the assessment of their consistency and rheology. Rheology and consistency of LFC mixes are commonly evaluated through the utilization of a flow cone test, which serves to analyze the performance of the mix [7]. The adequacy of the consistency and rheological properties of LFC is deemed satisfactory when the spreadability of fresh concrete blends is confined within the range of 40% to 60% of the duration of flow. For a mix to be adequately prepared for placement into moulds and achieve self-compaction without external assistance, the duration of the flowing time should not exceed 20 seconds. Allegedly, various factors exert an impact on the uniformity and flow properties of the mixture, which are primarily linked with the constituents of the LFC mix design.

3.1.2 Stability

The connection between the constituents of an LFC mix design and their overall consistency as a system are critical factors in assessing the level of stability. LFC can be categorized as a homogeneous type when its mixture exhibits a close fluid consistency that is creamy and easily pourable. This consistency is indicative of a fresh mix that is free from bleeding and segregation. The preparation of the mixture composition is deemed appropriate when a precise mix design method and accurate calculation procedure are employed, resulting in a discrepancy no greater than 2-7% between the attained plastic density and the intended plastic density [8]. A spread flow of 45% was determined to be a suitable indicator of workability, indicating favorable stability for any LFC mixture produced. To date, various assessments have been suggested by scholars to evaluate the consistency of the mixture. Various factors can impact the stability of a mixture, including the incorporation of mineral admixtures.

3.1.3 Compatibility

LFC compatibility is characterized by a robust interplay between the constituent elements of the mix design, particularly the chemical admixtures and foaming agents. Hence, in cases where there was a lack of association among the constituents of the mixture, the LFC compatibility would be diminished. The issue of segregation commonly arises as a result of the lack of interaction between the surfactant and plasticizers, which can be attributed to the incompatibility of their design admixtures.

3.1.4 Workability

The workability of LFC is demonstrated by its remarkable performance resulting from the incorporation of a stable foam agent, which creates air voids in the fresh mixture. The workability assessment, typically executed through a slump test for standard concrete, is unsuitable for fresh low-density concrete. The workability performance of LFC is assessed through visual means, with the objective of attaining a suitable viscosity for the mixture. In addition, Brewer [9] assessed the workability of LFC through the use of a spreadability technique.

3.2 Mechanical properties of LFC

The evaluation of LFC's suitability in its hardened state primarily revolves around the assessment of its mechanical properties. This section provides a thorough evaluation of the compressive, flexural, and tensile strengths, along with the modulus of elasticity, with the aim of providing the reader with current insight into the recent progressions in LFC.

3.2.1 Axial compressive strength

The available empirical information supports the notion that there is a direct association between density and axial compressive strength. Specifically, a decrease in density has a negative and exponential effect on the compressive strength of LFC. The available literature reveals a considerable variation in dry densities, spanning from 350 to 1850 kg/m^3, as documented by previous research. The compressive strength of the material is subject to the influence of multiple factors, such as the application rate of the foam agent, the water-to-cement ratio, the characteristics of the sand particles utilized, the curing method employed, the ratio of cement to sand, as well as the properties and distribution of any supplementary ingredients. The presence of air voids in hardened LFC is influenced by this particular factor, which subsequently results in variations in its compressive strength. Besides, the compressive strength of LFC is influenced by the water-to-cement ratio, which serves as an additional controlling factor. The inclusion of water in the mixture improves its consistency and stability, while also reducing the size of foam bubbles, ultimately leading to an increase in compressive strength [10]. The study documented the successful production of high-strength LFC using water-to-cement or binder ratios of 0.19 and 0.17, respectively.

3.2.2 Flexural and tensile strengths

The tensile strength of LFC exhibits a relatively lower magnitude in comparison to that of conventional normal-strength concrete. Historically, scholarly literature has recorded that the ratio between tensile strength and compressive strength in LFC typically ranges from 0.22 to 0.42. This ratio is notably higher when compared to the corresponding ratio observed in conventional normal-strength concrete.

The splitting tensile strength to compressive strength ratio of normal-strength concrete typically falls within the range of 0.07 to 0.12 [11]. The flexural and tensile strength of LFC has been observed to demonstrate a range of values between 15% and 35% in relation to its compressive strength. Additionally, previous studies have demonstrated that the relationship between flexural strength and compressive strength of LFC becomes negligible when the concrete density falls below 300 kg/m^3. The inclusion of mineral admixtures and fibers in the mixture composition has the capacity to augment the tensile strength of LFC. The observed increase in performance can be attributed to the increased shear capacity between the fine particles of sand and foam agent, as supported by multiple sources of evidence.

3.3 Physical properties

Some physical properties of LFC include density, drying shrinkage, permeable porosity, and sorptivity.

3.3.1 Density

The determination of mix density can be categorized into two phases specifically fresh density and dry density. It is recommended to restrict the disparity between fresh and dry density values within the range of 90-110 kg/m^3 [12]. The determination of the actual density of the fresh mixture is typically conducted by filling a pre-weighed standard container with a known volume of the LFC that has been produced and subsequently weighing it. Next, it is important to evaluate the divergence between the target design densities and the densities that were achieved. The acceptable tolerance for dry density is typically restricted to a range of ±55 kg/m^3, although for high-density LFC mixes (such as those with a density of 1600 kg/m^3), this difference may extend up to ±95 kg/m^3. The objective of determining the fresh density is to establish the precise volume required for the design mix and the control of casting, while the dry density plays a critical role in regulating the mechanical, physical, and durability characteristics of hardened LFC.

3.3.2 Drying shrinkage

The occurrence of drying shrinkage is considered a notable constraint of LFC, commonly observed during the first 20 days after the casting process. The drying shrinkage of LFC generally ranges from 0.1% to 0.35% of the total volume of the hardened LFC matrix. The compressive strength of this material exhibits a substantial increase compared to conventional concrete, ranging from four to ten times higher. This is primarily attributed to factors such as the type of aggregate used in the mix design, increased cement and water contents, and the presence of mineral admixtures in LFC. Currently, there exists a need for understanding the impact of cement content on the drying shrinkage of LFC. This issue can be mitigated by partially replacing Portland cement with alternative supplementary materials like fly ash, silica fume, and lime. The utilization of these materials is advantageous due to their lower heat of hydration [13].

3.3.3 Permeable porosity

The inclusion of porosity in the analysis of LFC is of utmost importance owing to its substantial influence on a range of characteristics, encompassing compressive and flexural strengths, as well as durability. The infiltration of water into concrete is not exclusively governed by its porosity, but rather influenced by various factors including pore diameter, distribution, continuity, and tortuosity. The determination of porosity in LFC is performed through the utilization of various techniques, including apparent porosity, total vacuum saturation, and mercury intrusion porosimetry (MIP). However, the vacuum saturation method is widely regarded as the primary approach for assessing the porosity of LFC. It has been documented that this method provides results that are 66% and 13% more precise compared to the apparent and MIP methods, respectively [5].

3.3.4 Sorptivity

The term sorptivity refers to the quantitative assessment of a medium's capacity to absorb a liquid via capillary action. The sorptivity of LFC is affected by various factors, including the type of foaming agent, mineral admixtures, density, permeability characteristics, and curing conditions. The parameters mentioned above influence the propensity of water transmission with respect to bubble size (pores), tortuosity, and the uniformity and continuity of distribution. The determination of sorptivity can be accomplished by employing the principles of unsaturated flow and quantifying the rate of capillary rise absorption in concrete that possesses a relatively uniform composition, such as LFC [14].

4.0 Employment of LFC

The use of LFC has become increasingly prevalent in diverse civil and structural engineering fields owing to its distinctive attributes, including decreased density, diminished thermal conductivity, enhanced flowability, and inherent self-compacting capabilities. Furthermore, the ease with which it can be produced, and its relatively cost-effective nature have also played a significant role in its extensive adoption. For example, the use of low-density LFC has been observed in cavity filling and insulation applications, while high densities have been employed in structural contexts. LFC has various additional applications, which encompass the manufacturing of lightweight blocks and pre-cast panels, fire insulation, thermal and acoustic insulation, road sub-base construction, trench reinstatement, soil stabilization, and the creation of shock-absorbing barriers for airports and regular traffic. The utilization of LFC has gained significant traction on a global scale, particularly in areas that are grappling with housing deficiencies or are prone to inclement weather conditions such as hurricanes and earthquakes. LFC has been extensively utilized in Malaysia for various purposes such as tunnel annulus grouting, flowable fills, and geotechnical applications. The surge in interest can be attributed, at least in part, to the notable rise in expenses associated with alternative lightweight construction materials like plasterboard and wood, as well as the growing concerns surrounding environmental factors. Moreover, LFC possesses an additional characteristic that makes it suitable for incorporating large quantities of supplementary cementing admixtures. This is due to the high manufacturing and environmental costs associated with cement production. The annual market size of LFC in the UK is estimated to be approximately 250,000-300,000 m^3, which includes a significant mine stabilization project. The market size of Western Canada is estimated to be approximately 50,000 m^3 [15] per year. Moreover, in the context of Korea, an annual construction volume of approximately 250,000 m^3 of LFC is utilized as a crucial element in the implementation of floor heating systems [16].

5.0 Conclusions

This manuscript primarily aimed to assess the present characteristics of LFC for potential utilization in the Malaysian construction industry. It subsequently delved into enhancing the design proportions of LFC and selecting constituent materials. The primary focus of most investigations was on the assessment of the LFC properties, with limited attention given to the intrinsic characteristics of the foam material or its impact on the strength of the LFC matrix. The foam stabilization mechanism in concrete is a critical aspect that plays a significant role in achieving consistent geometric properties and uniform progression of hardening stages. The determination of material mix proportions for LFC lacks standardized methods, as it is contingent upon various factors such as desired density, required strength, and the specific approaches used for material proportioning, guidelines, and trial and error methods. The production of stable LFC is influenced by various factors, including the type of foam agent used, the method of preparing the foaming agent to ensure a consistent distribution of air voids, the process of foaming materials, the accuracy of design calculations for the mixture, and the production of LFC. These factors play a crucial role in improving the performance of LFC in both its fresh and hardened states. The addition of foam agents results in the creation of a consistent distribution of pores, which effectively mitigates the issue of segregation at an early stage. Furthermore, foam agents serve to impede the penetration of chloride, prevent sulfate-induced deterioration, and extend the duration of fire resistance.

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TECHNICAL REPORT

Reprint from CI Magazine, Volume 37, No 1, Page 51-58

Lap Splices in Unconfined Boundary Elements

Tests indicate that a currently allowed detail provides insufficient toughness

by John N. Hardisty, Enrique Villalobos, Brian P. Richter, and Santiago Pujol

Because lap splices are believed to limit frame toughness, they are not used near critical sections of frames that are required to resist earthquake demands. Nevertheless, the current building code1 still allows lap splices at the bases of structural walls, where large inelastic deformations are expected to take place during strong ground motions. In the most critical cases, these splices are not located within confined boundary elements.

Certainly, the tensile strength of lap splices has been studied extensively.2 However, failures of unconfined deformed-bar lap splices observed after recent seismic events in Turkey,3 Japan,4 and Chile5 indicate a need to revisit the topic.

In buildings with structural walls, wall toughness is critical to seismic response. While most structural walls are currently constructed with lap splices at their bases, information on the deformation capacity of such walls is scarce.6,7 The work described herein was aimed at generating data to help fill that gap.

Experimental Program

Six beams and two structural walls were tested. For the beams, test variables included the type of hook on the transverse reinforcement, spacing of the longitudinal and transverse reinforcement, and whether the reinforcing steel had a well-defined yield plateau. For the walls, test variables included the type of hook on the transverse reinforcement and the spacing of the longitudinal reinforcement. This article comprises a summary of the work. Detailed information can be accessed at http://nees.org/warehouse/project/1130 and http://nees.org/warehouse/project/1050. Beams T-60-8-A, B, and C were tested by Richter.8 Walls were tested by Villalobos.9 Beams T-60-8-D, E, and F are not reported anywhere else.

Beams

Table 1 summarizes specimen properties for the test beams. Test beams (Fig. 1) were designed with stems (webs) propor-tioned to replicate the boundary elements of relatively thin structural walls. Longitudinal tension reinforcement comprised four No. 8 bars located in the web (Fig. 1(a)). The clear cover to the transverse reinforcement was 3/4 in. (19 mm), which is typical in structural walls today. The minimum clear cover over the longitudinal bars was 1-1/8 bar diameters. The effective depth d, the distance between extreme compression fiber and the centroid of longitudinal reinforcement, was 25-7/8 in. (657 mm) for all beams except Beam T-60-8-B, which had a d of 26-3/8 in. (670 mm). Beam T-60-80-B was designed to mimic structural walls constructed in Chile, with close longitudinal bar clear spacing sc in lap splices in wall boundary elements.5

Longitudinal reinforcement ratios ρ were 0.63% and 0.61% outside the spliced region and 1.26% and 1.22% inside the spliced region. Stirrup types, center-to-center spacing s, and the resulting transverse reinforcement ratios ρv are provided in Table 1 and Fig. 2 (note that Beam T-60-8-C had no transverse reinforcement).

The beams were cast with the No. 8 longitudinal bars at the bottom of the formwork to avoid top-casting effects. Before testing, the beams were rotated 180 degrees about the longitudinal axis so that the longitudinal bars were at the top of the beam during testing.

Walls

Two wall specimens, W-60-N and W-60-N2, were tested. Each specimen consisted of a footing and a wall (Fig. 3). The footing was fastened to the laboratory floor, and lateral load was applied near the top of the wall. Each footing was 8 ft (2.44 m) long, 4 ft (1.22 m) wide, and 3 ft (0.91 m) thick. Each wall was centered on its footing and was 8 in. (203 mm) thick (bw), 5 ft (1.52 m) long (Lw), and 12 ft (3.66 m) tall.

Details of the reinforcement layout and dimensions for the walls are shown in Fig. 4. Longitudinal reinforcement comprised four No. 8 bars in each boundary element (providing tension reinforcement As of 3.16 in.2 [2039 mm2]) and six No. 4 bars distributed in two layers in the web and spaced at 12 in. (305 mm) on center. The longitudinal bars were lap spliced at the base of the walls, with splice lengths of 60 bar diameters. The minimum clear cover to the longitudinal bars was 1.1 bar diameters. Table 2 lists the main properties of the walls. For both walls, the effective depth to the boundary- element longitudinal bars was approximately 4 ft 8 in. (1.42 m).

Transverse reinforcement in both walls consisted of two No. 3 ties spaced at 5 in. (127 mm) on center, resulting in a transverse reinforcement ratio of 0.55%. The clear cover to the transverse reinforcement was 3/4 in. (19 mm). Wall W-60-N had ties similar to the Type II stirrups of Beam T-60-8-A, while Wall W-60-N2 had ties similar to the Type I stirrups of Beam T-60-8-B (refer to Fig. 2 and Fig. 4).

Table 1:
Summary of properties for beam test specimens

Beam label	Clear spacing between spliced bars s_c	Stirrup type*	Center-to-center spacing of transverse reinforcement s, in. (mm)	Transverse reinforcement ratio ρ_v, %	Well-defined yield plateau	Yield stress f_y, ksi (MPa)		Compressive strength f_c', psi (MPa)
						No. 8 bars†	No. 3 bars‡	
T-60-8-A	1.5 d_b	II	5 (127)	0.55	Y	65 (448)	63 (434)	4300 (29.6)
T-60-8-B	0.5 d_b	I	5 (127)	0.55	Y	66 (455)	63 (434)	4100 (28.3)
T-60-8-C	1.5 d_b	—	—	0.0	Y	66 (455)	63 (434)	4100 (38.3)
T-60-8-D	1.5 d_b	III	8 (203)	0.34	N	63§ (434)	63 (434)	5900 (40.7)
T-60-8-E	1.5 d_b	I	8 (203)	0.34	N	63§ (434)	63 (434)	5200 (35.9)
T-60-8-F	1.5 d_b	III	11 (279)	0.25	N	63§ (434)	63 (434)	6300 (43.4)

***Refer to Fig. 2**
†Boundary-element longitudinal bars
‡Bars used for stirrups
§Taken as fs at 0.2% offset
Note: db is longitudinal bar diameter

The wall specimens were cast vertically in three lifts—one for the footing and two for the wall. The resulting cold joints were cleaned, roughened, and moistened before casting the subsequent lift.

Test Setup, Instrumentation, and Procedure Beams

The beam test setup is shown in Fig. 1(b). The beams were loaded in four-point bending with their webs in tension. Lap splices, all with lap lengths of 60 bar diameters, were located in the constant moment region between the supports. Load was applied by four 30 ton (300 kN) center-hole hydraulic rams (two rams at each load point) that reacted against high-strength threaded rods anchored to the laboratory strong floor.

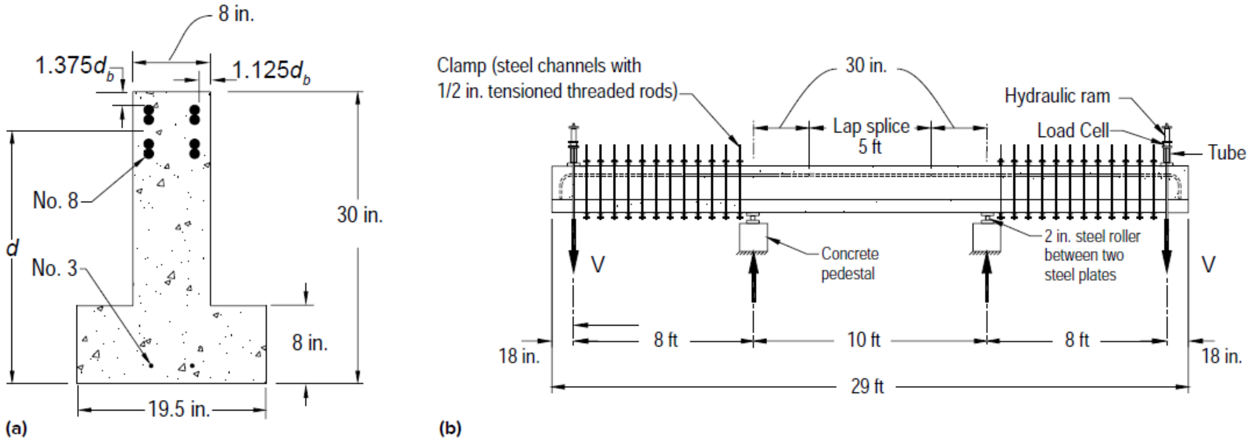

Fig. 1: Geometry of test beams: (a) section; and (b) elevation and test setup (Note: 1 in. = 25.4 mm; 1 ft = 0.3 m)

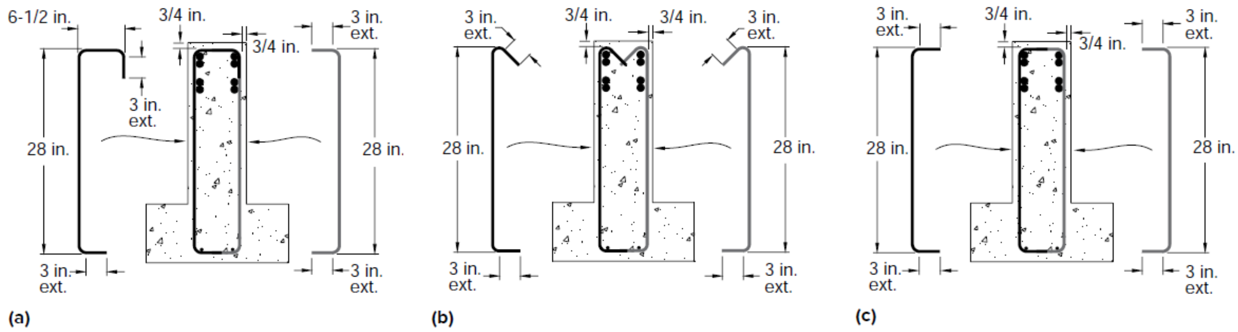

Fig. 2: Stirrup types: (a) Type I, used in Beams T-60-8-B and E; (b) Type II, used in Beam T-60-8-A; and (c) Type III, used in Beams T-60-8-D and F (Note: 1 in. = 25.4 mm; stirrup types shown for Beams T-60-8-A and B (in Fig. 2-7 and 2-8) in Reference 8 were accidentally interchanged).
All stirrups were fabricated using No. 3 bars

Table 2:
Summary of properties for wall test specimens

Wall label	Clear spacing between spliced bars s_c	Tie type*	Yield stress f_y, ksi (MPa)			Compressive strength f_c', psi (MPa)
			No. 8 bars†	No. 4 bars‡	No. 3 bars§	
W-60-N	1.75d_b	II	67 (462)	63 (434)	77 (531)	4300 to 4900 (39.6 to 33.8)
W-60-N2	0.5d_b	I	68 (469)	63 (434)	66 (455)	4600 to 4700 (31.7 to 32.4)

*Refer to Fig. 4
†Boundary-element longitudinal bars
‡Web longitudinal bars
§Bars used for ties
Note: d_b is longitudinal bar diameter

During testing, load was increased monotonically until the longitudinal reinforcement yielded or the specimen failed (whichever occurred first). Loading was increased at 8 kip (35.6 kN) increments (at each end). If yield was reached, loading was stopped and the specimen was unloaded. The specimen was reloaded to the same load seven times (Beams T-60-8-A and B) or 10 times (Beams T-60-8-D, E, and F). Beam T-60-8-C failed before yield. If failure did not take place in the reloading at the yield force, the load was increased to reach a displacement equal to twice the displacement at first yield. This procedure was repeated until failure. Load and vertical displacements were monitored continuously through-out the test. Vertical displacement was measured at the load points, midspan, and splice ends. Figures 5 and 6 provide examples of measured load-deflection curves.

Walls

The wall test setup is shown in Fig. 3. Eight post-tensioning bars (four 1-3/8 in. [35 mm] diameter bars at the corners of the footing and four 1-1/4-in. [32 mm] diameter bars near the center of the footing) fastened the footing to the laboratory reaction floor with a total force of 960 kip (4270 kN). The north and south faces of the footing were butted against steel reaction blocks to prevent slip. Each reaction block was post-tensioned to the laboratory floor with a force of 600 kip (2670 kN).

An axial load of 200 kip (890 kN) was applied at the top of each wall by four 1-1/4 in. (32 mm) in diameter post-ten-sioning bars tensioned by hydraulic rams reacting on steel tubes that rested on top of the wall. The rams were connected to a manifold and were continuously controlled by a single manual pump during the tests to maintain the axial load within 1% of the target load (±2 kip [8.9 kN]).

Lateral load was applied by two hydraulic actuators with swivels at both ends and attached to the test walls through load tubes perpendicular to the in-plane direction of the wall on either side. The load rig was attached to the wall by two, 1-1/4 in. (32 mm) in diameter, post-tensioning bars reacting on either tube and tensioned by hydraulic rams. The distance from the top of the footing to the point of load application (that is, the clear wall height) was 10 ft, 10-1/2 in. (3.32 m). Lateral bracing was provided by square steel tubes on either side of the wall at 10 ft (3.05 m) from the top of the footing.

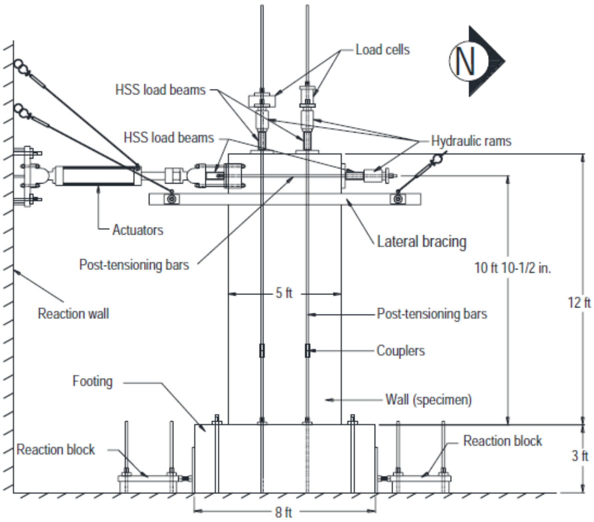

Fig. 3: Test wall east elevation and setup (Note: 1 ft = 0.3 m)

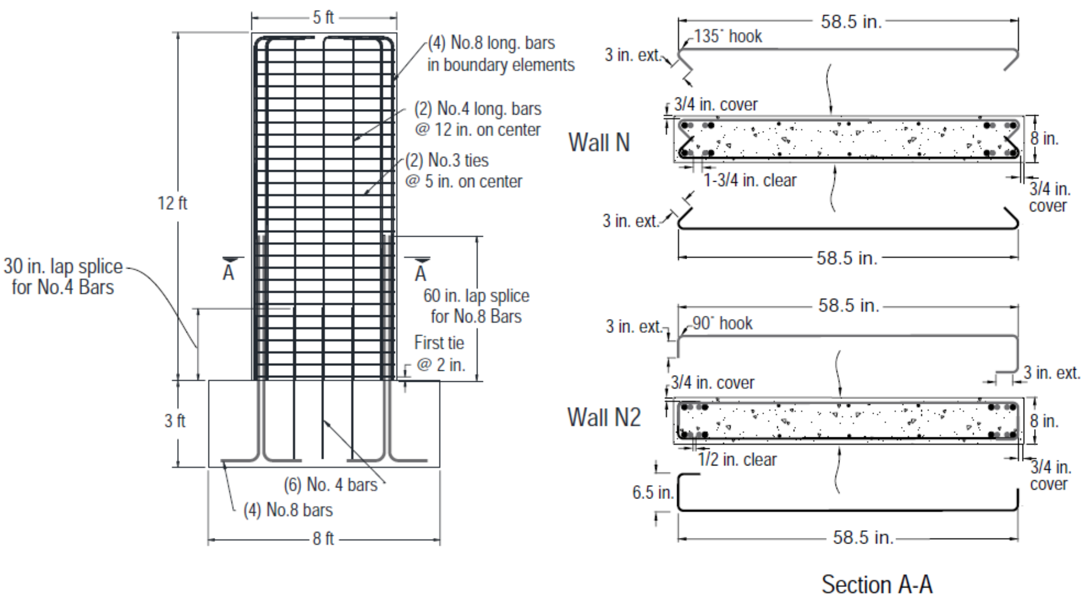

Fig. 4: Wall reinforcement layout and dimensions (Note: 1 in. = 25.4 mm; 1 ft = 0.3 m)

The test procedure consisted of slowly applied displacement reversals, increasing in magnitude up to failure. Three cycles were applied at each of the following target drift ratios: 1/8, 1/4, 1/2, 3/4, 1, 1-1/2, 2, 2-1/2, and 3% of the clear wall height.
The applied lateral load, axial load, and displacements of the footing and along the height of the wall were measured continuously throughout the test.

Material Properties

Concrete compressive strength was obtained from compression tests of 6 x 12 in. (152 x 305 mm) cylinders. Stress-strain relationships for the reinforcing steel were obtained from standard tensile tests. Three coupons from each heat of steel were tested.

Beams

For the beams, concrete compressive strength at the time of testing ranged from 4100 to 6300 psi (28.3 to 43.4 MPa). The reinforcing bars were certified as meeting ASTM A615/A615M, "Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement." Our tests showed, however, that the bars met the tensile yield stress requirement described in ASTM A706/A706M, "Standard Specification for Deformed and Plain Low-Alloy Steel Bars for Concrete Reinforcement."
For Beams T-60-8-A, B, and C, the longitudinal steel had a well-defined yield plateau. For Beams T-60-8-D, E, and F, the longitudinal reinforcing steel did not have a well-defined yield plateau. In all beams, the yield stress of the transverse reinforcement was 63 ksi (434 MPa). Material properties for each beam are listed in Table 1.

Table 3:
Summary of test results for beam test specimens

Beam label	Peak load reached during reloading, kip (kN)			Maximum applied load V_u, kip (kN)	Midspan deflection at maximum applied load Δ_u, in. (mm)	Midspan deflection at yield Δ_y, in. (mm)	Δ_u/Δ_y	Δ_u/L_{CL}, %	Δ_y/L_{CL}, %
	V_1	V_2	V_3						
T-60-8-A	55.0 (244.7)	57.0 (253.5)	59.0 (262.4)	59.4 (264.2)	0.78 (19.8)	0.20 (5.1)	3.9	1.30	0.33
T-60-8-B	56.5 (251.3)	—	—	56.7 (252.2)	0.33 (8.4)	0.21 (5.3)	1.6	0.55	0.35
T-60-8-C*	—	—	—	42.2 (187.7)	0.17 (4.3)	—	—	0.28	—
T-60-8-D	56.0 (249.1)	65.0 (289.1)	—	64.8 (288.2)	0.58 (14.7)	0.26 (6.6)	2.2	0.97	0.43
T-60-8-E	58.0 (258.0)	66.0 (293.6)	—	66.1 (294.0)	0.62 (15.7)	0.28 (7.1)	2.2	1.03	0.47
T-60-8-F	58.0 (258.0)	65.0 (289.1)	—	64.8 (288.2)	0.56 (14.2)	0.29 (7.4)	1.9	0.93	0.48

*Specimen did not reach yield

Note: L_{CL} is distance from support to centerline of beam span

Walls

For the walls, the compressive strength of concrete at the time of testing ranged from 4300 to 4900 psi (29.6 to 33.8 MPa) for Wall W-60-N and from 4600 to 4700 psi (31.7 to 32.4 MPa) for Wall W-60-N2. The No. 4 longitudinal bars in both wall specimens had a yield stress of 63 ksi (434 MPa) and the No. 3 ties had a yield stress of 77 ksi (531 MPa) in Wall W-60-N and 66 ksi (455 MPa) in Wall W-60-N2. The reinforcing bars were certified as ASTM A706/A706M Grade 60. Material properties for each wall are listed in Table 2.

Test Results and Data Analyses

Table 3 provides a summary of the beam test results. Sample load-deflection curves for two specimens are presented in Fig. 5 and 6. The loads reported are the means of the loads applied at each beam end. The reported midspan deflection at yield is the midspan deflection corresponding to the intersection of a tangent to the applied load versus midspan deflection curve in the linear range of response after cracking, and a tangent to the same curve in the inelastic range of response.

Table 4 shows the measured lateral loads and drift ratios. Based on a definition of limiting drift ratio as the maximum drift ratio at which the lateral load was at least 80% of the maximum load (in each direction), the limiting drift ratio was 2% for Wall W-60-N and 1.5% for Wall W-60-N2.

Failure mechanisms

In spite of their similar tension reinforcement layouts, the wall specimens were observed to fail in compression while the beam specimens failed in bond along the splices (Fig. 7). The moment gradient over the splice length appears to have been be a major factor. While the lap splices in the walls were

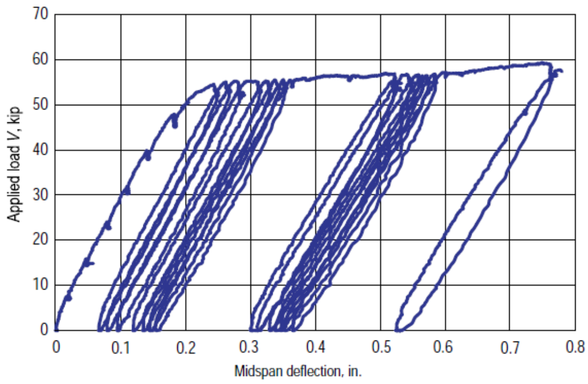

Fig. 5: Applied load *V* versus midspan deflection for Beam T-60-8-A (Note: 1 kip = 4.45 kN; 1 in. = 25.4 mm)

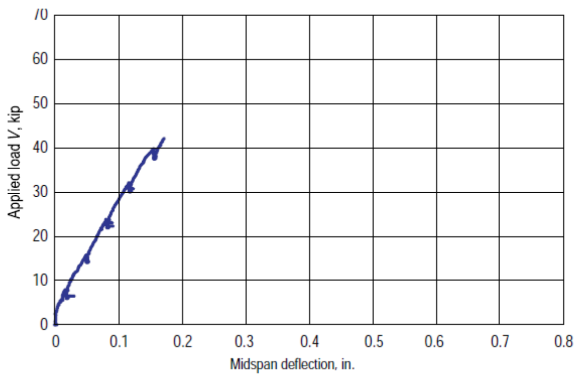

Fig. 6: Applied load *V* versus midspan deflection for Beam T-60-8-C with visible failure before yield (Note: 1 kip = 4.45 kN; 1 in. = 25.4 mm)

Table 4:
Summary of results of wall test specimens

Wall label	Loading direction	Maximum applied load V_u, kip (kN)	Δ_u/L, %	Applied load at limiting drift ratio V_Δ, kip (kN)	Δ_Δ/L, %
W-60-N	North	159 (707)	1.5	143 (636)	2.0
	South	−155 (−689)	−1.5	−127 (−565)	−2.0
W-60-N2	North	170 (756)	1.5	142 (632)	2.0
	South	−167 (−743)	−1.5	−167 (−743)	−1.5

Note: Δ_u is deflection at maximum applied load, L is distance from top of footing to point of load application (10 ft 10-1/2 in. [3.32 m]), and Δ_Δ is limiting drift

subjected to large moment gradients, the lap splices in the beams were subjected to a nearly constant moment over their lengths. The moment imposed on the test walls varied from zero, at the point of load application, to a maximum at the base of the wall. The moment arm for this load was 130.5 in. (3315 mm), while the length of the lap splice was 60 in. (1525 mm). The moment developed along the lap splices therefore varied from 100% of maximum at base of the wall to approximately 54% of maximum at the opposite end of the splice. In contrast, four-point bending of the test beams placed the splices in the beams in nearly constant moment.

In a multi-story building with splices at the base, the moment that would be developed in the splices would not vary much along the splice length. These observations suggest that test results for scaled structural walls with large-scale lap splices cannot be reliably projected to full-scale walls. The results from beam tests could be expected to better represent the performance of full-scale structural walls.

From tests of cantilever beams with the same length but different lap splice lengths, Ferguson and Briceno10 inferred that splice strength is sensitive to the difference between the bar stresses at each end of the splice. It follows that splice strength is sensitive to moment gradient. The tests reported herein confirm this inference.

Splice Strength

Table 5 shows the maximum total moments, peak steel stresses, and mean unit bond strengths computed for each layer of reinforcement. The maximum total moments and steel stresses were computed at the ends of each splice. Maximum total moment results from the maximum applied load, the weight of the test hardware, and the self-weight of each beam. The steel stresses in each reinforcement layer (maximum and at each load increment) were computed using the method outlined by Richter.8 The data in Tables 3 and 5 show that the presence of transverse reinforcement resulted in increases in mean unit bond strength and deformation capacity.

Figure 8 shows drift ratio at failure versus transverse reinforcement ratio. The most striking feature of this plot is the magnitude of the drift ratios reached. No beam specimen went beyond a drift ratio of 1.3%. If we consider that 1) no load reversals were applied; and 2) in a strong earthquake, cover spalling is likely to occur at wall bases—where lap splices are often located—the measured drifts suggest that structural walls with details similar to those tested do not have the toughness required to survive earthquake demands.

Figure 8 also shows that at the same transverse reinforcement ratio, decreasing the clear spacing between spliced bars lowered the drift capacity at failure. This is evident in beams T-60-8-A and B.

At the same clear vertical bar spacing and hook configuration, increasing the transverse reinforcement ratio led to an increase in drift capacity. Provided that the transverse reinforcement ratio stayed the same and that minimum hook dimensions were met, transverse reinforcement detailing (hook configuration) appears to have had no appreciable effect on peak drift ratio.

In Fig. 9, it can be seen that peak reinforcement stress varied inearly with transverse reinforcement ratio. While vertical bar clear spacing and transverse reinforcement detailing appear to have had little effect on maximum stress, variations in bar clear spacing did appear to affect drift ratios at failure (Fig. 8).

Fig. 7: Splice failure in test beam

Table 5:
Estimated maximum steel stress and mean bond stress in test beams

Beam label	Maximum total moment at slice end M_u, kip-ft (kNm)	Steel layer	Maximum steel stress at splice end f_{su}, ksi (MPa)		Mean unit bond strength μ_u, psi (MPa)		Transverse reinforcement ratio ρ_v, %
T-60-8-A	506 (686)	Top Bottom	81 (559) 80 (552)	$1235\sqrt{f_c'}$ $1220\sqrt{f_c'}$	338 (2.33) 333 (2.30)	$5.1\sqrt{f_c'}$ $5.1\sqrt{f_c'}$	0.55
T-60-8-B	485 (658)	Top Bottom	78 (538) 78 (538)	$1218\sqrt{f_c'}$ $1218\sqrt{f_c'}$	325 (2.24) 325 (2.24)	$5.1\sqrt{f_c'}$ $5.1\sqrt{f_c'}$	0.55
T-60-8-C	369 (500)	Top Bottom	65 (448) 54 (372)	$1015\sqrt{f_c'}$ $843\sqrt{f_c'}$	271 (1.87) 225 (1.55)	$4.2\sqrt{f_c'}$ $3.5\sqrt{f_c'}$	0.00
T-60-8-D	550 (746)	Top Bottom	86 (593) 84 (579)	$1120\sqrt{f_c'}$ $1094\sqrt{f_c'}$	358 (2.47) 350 (2.41)	$4.7\sqrt{f_c'}$ $4.6\sqrt{f_c'}$	0.34
T-60-8-E	560 (759)	Top Bottom	88 (607) 86 (593)	$1220\sqrt{f_c'}$ $1193\sqrt{f_c'}$	367 (2.53) 358 (2.47	$5.1\sqrt{f_c'}$ $5.0\sqrt{f_c'}$	0.34
T-60-8-F	550 (746)	Top Bottom	86 (593) 84 (579)	$1083\sqrt{f_c'}$ $1058\sqrt{f_c'}$	358 (2.47) 350 (2.41)	$4.5\sqrt{f_c'}$ $4.4\sqrt{f_c'}$	0.25

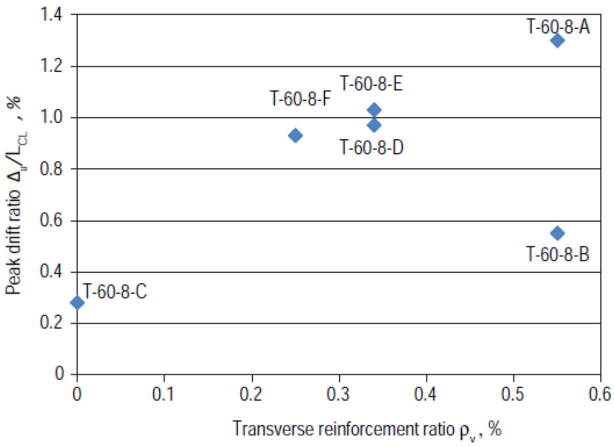

Fig. 8: Peak drift ratio versus transverse reinforcement ratio for test beams

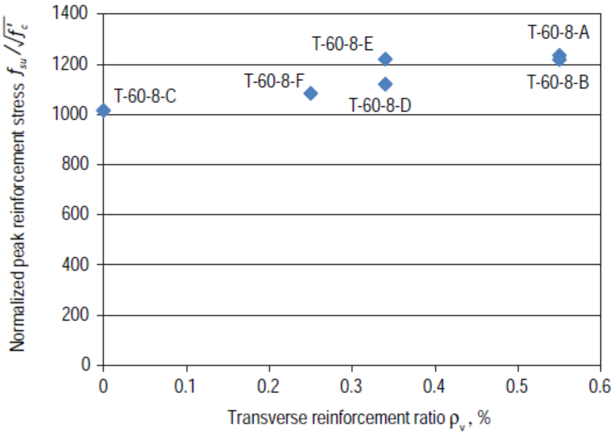

Fig. 9: Peak mean reinforcement stress versus transverse reinforcement ratio for test beams

Summary and Conclusions

Tests were conducted on six test beams and two scaled walls that were designed to represent 8 in. (200 mm) thick structural walls with lap splices in unconfined boundary elements. The subject is relevant because unconfined lap splices at bases of structural walls are allowed by the current building code. The test beams were evaluated in constant moment, with the lap splices in tension. Lap splice length was 60db and met current design requirements, while concrete strength and transverse reinforcement ratio varied from approximately 4100 to 6300 psi (28.3 to 43.4 MPa) and 0 to 0.55%, respectively. Lap splices in all test beams failed at drift ratios ranging from approximately 0.3 to 1.3%. Boundary reinforcement in the two test walls was similar to the tensile reinforcement in the test beams. The walls were loaded as cantilevers with a horizontal in-plane load near the free end and subjected to displacement reversals. Lap splices in walls did not fail. Nevertheless, the test walls also failed at modest drift ratios (as low as 1.5%).

The most salient conclusions from the data obtained in our tests are:

- The beam test results were not affected by the shape of the stress-strain curve (the presence or absence of a well-defined yield plateau) in the longitudinal bars, suggesting that bar stress is a more critical factor affecting splices than bar strain;
- Comparisons of the responses of the beam and wall tests confirm that the strength of a lap splice is sensitive to the moment gradient over the length of the splice. Because the bases of most structural walls will be subjected to moments with small gradients, results from tests of lap splices in scaled walls cannot be projected directly to full-scale walls. A more conservative approach is to base performance projections on results from tests in which lap splices are subject to constant moment over the splice length;
- Drift capacity was modest (no more than 1% in five out of six beam tests) in beam specimens with unconfined splices. While peak bar stress and drift capacity increased with increases in transverse reinforcement ratio, these increases were offset almost completely by reductions in the clear spacing between spliced bars, which can occur easily during construction; and
- Because factors such as spalling and displacement reversals are bound to reduce further the drift capacity of walls, the observed drift capacities suggest that structural walls with unconfined lap splices at their bases do not have the toughness required to survive the demands associated with strong ground motion caused by earthquakes.

Acknowledgments
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Note: Additional information on the ASTM standards discussed in this article can be found at www.astm.org.

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CASE STUDY

Reprint from CI Magazine, Volume 37, No 2 , Page 47-51

Glen Elder Dam and Fort Randall Dam Spillway Repairs

Work included the use of hydrodemolition and low-shrinkage concrete mixtures

by Clinton L. Powell and Bob Schieffer

In 1944, the U.S. government instituted a comprehensive plan for the conservation, control, and use of water resources in the entire Missouri River Basin in the central United States. The legislation resulted in the construction of numerous dams and reservoirs on the Missouri River and its tributaries. This article highlights unique concrete repairs recently made on two of the associated spillway structures.

Completed in 1969, the Glen Elder Dam is located at the confluence of the North and South Forks of the Solomon River in north-central Kansas, near Glen Elder. These streams are part of the Missouri River tributary system, as the Solomon flows into the Kansas River, which in turn feeds into the Missouri River near downtown Kansas City, MO. The Glen Elder Dam comprises an earth embankment and a concrete spillway. The spillway inlet apron slab underwent significant renovation in early 2010.

Completed in 1952, the Fort Randall Dam is located on the Missouri River in southeast South Dakota, near Pickstown. Also comprising an earth embankment with concrete spillway and apron systems, the dam suffered major, nondam safety critical damage during historic flooding in the spring and summer of 2011. The spillway outlet apron slab underwent significant renovation in 2013 and 2014.

Glen Elder Dam Spillway Structure

Glen Elder Dam's spillway includes twelve 50 ft (15.2 m) wide by 22 ft (6.7 m) high radial gates seated on top of a concrete ogee crest (Fig. 1). The spillway is founded on layers of fragmented limestone, shale, and the occasional clay seam. Due to concern of lateral shifting across the clay seams when the spillway is under full reservoir conditions, about 2000 foundation anchors were grouted into the foundation to provide sliding stability.

A series of reinforced concrete inlet apron slabs extends 50 ft (15.2 m) upstream of the gate structure and across the full 644 ft (196.2 m) width of the spillway. The inlet apron slabs connect roughly 760 of the aforementioned foundation anchors to the spillway structure. The concrete slabs were slowly deteriorating and thereby reducing the effectiveness of the foundation anchors and the overall sliding stability of the spillway.

Fig. 1: View of repair work for the inlet apron slabs on the upstream side of Glen Elder Dam spillway. There is no water in contact with the spillway unless the reservoir is in the flood pool

Damage evaluation

Petrographic evaluations of concrete cores indicated that cracking induced by cycles of freezing and thawing in saturated conditions was the primary source of deterioration. The cracks allowed water to to flow more easily through the concrete, which led to further damage by alkali-silica reaction. Depths of deterioration ranged from 0 to 14 in. (0 to 350 mm). As the concrete began to deteriorate, especially near construction joints, the deteriorated areas were capable of holding water, leading to an exponential rate of deterioration during cycles of freezing and thawing. Some of the deteriorated areas could be excavated with a shovel.

Repair

At first glance, a complete replacement of the inlet slabs seemed to be the appropriate method for restoring the sliding stability of the structure. However, after looking into the difficulty of not deforming or damaging the roughly 760 foundation anchors in the demolition process, it was determined that other alternatives should be evaluated. Life-cycle costs were developed for several alternatives: installing new foundation anchors, making only partial repairs, and executing full replacement. It was decided the most economical option was to selectively remove deteriorated concrete using hydrodemolition and place concrete patches back to the original lines and grades.

Hydrodemolition is commonly used by road departments to remove and roughen the top inch or two of concrete bridge decks to facilitate good bonding for overlays. In the case of Glen Elder Dam, the excavation depths were going to be highly variable, from a minimum of 6 up to 18 in. (150 to 450 mm). The concept behind hydrodemolition is that a high-pressure water jet (about 20,000 psi [138 MPa]) will continue to take out deteriorated and cracked concrete until it runs into good quality concrete. Benefits include:

- Fast removal of unsound material;
- Production of a three-dimensional concrete surface with maximum potential for bonding;
- Cleaning of rust from reinforcing;
- Avoiding concrete microcracking typically associated with chipping hammers; and
- Fast removal (worth mentioning again).The drawbacks are nonexistent unless the project isn't large enough to overcome the higher mobilization cost.

Using hydrodemolition with such a large variation in depth of removal is not a common application. This resulted in some apprehension prior to moving forward. At the preconstruction meeting, the hydrodemolition foreman indicated he had never seen a project like this before in the 17 years he had been demolishing concrete, which didn't lift the project delivery team's spirits. After several iterations of equipment calibrations, involving adjustments to the number of nozzle revolutions as well as revolution speed to facilitate the removal of poor- quality concrete while leaving high-quality concrete in place, the hydrodemolition process was dialed in and proved successful. After hydrodemolition, crews performed minor sawcutting and chipping hammer work to prepare the slabs for placement (Fig. 2).

Fig. 2: Inlet slabs and spillway structure prior to placement of overlay concrete. The hooked bars that extend over the top mat are foundation anchors (No. 11 bars grouted into the underlying rock)

The initial placements exhibited restrained shrinkage cracking, which proved to be a challenge for the relatively large and constrained slab sections (Fig. 3 and 4). The mixture design was adjusted to include more coarse aggregate along with water-reducing admixtures, but the measures resulted in minimal success.

An experimental shrinkage-reducing/compensating admixture, a blend of magnesium oxide and glycol ether which is now marketed as PREVent-C®, was then tried at different dosages. The tests eventually resulted in a mixture with restrained shrinkage cracking reduced by 90% relative to control mixtures.1

The completed repairs restored structural sliding stability for the Glen Elder Dam spillway. This project demonstrated that hydrodemolition could be an economical means to selectively demolish deteriorated concrete at variable depths. Additionally, admixture and design criteria were developed to help minimize restrained shrinkage cracking.

Fig. 3: Placing concrete back to the original lines and grades

Fig. 4: Typical restrained shrinkage cracking observed in the Glen Elder Dam spillway repair areas (based on Reference 1). This cracking prompted a review and adjustment of the concrete mixture used for subsequent repairs

Fort Randall Dam

During the flooding of 2011, the Fort Randall Dam spillway was subjected to a record flow of 143,000 ft3/s (4049 m3/s). To put this into historical context, the average discharge from the Fort Randall Dam control structure is 29,000 ft3/s (821 m3/s).2 The extensive water flow caused damage to many of the dam's structures, including the 1000 ft (305 m) wide by 1805 ft (550 m) long spillway slab.

Damage evaluation

During the initial assessment of the spillway damage, ground-penetrating radar was used to estimate the required scope of repairs. While the radar results showed extensive anomalies indicative of delamination along both spillway walls and the expansion joint between the slab and the gate structure, the designers chose to discount a majority of the anomalies as false returns and identified only 40,000 ft2 (3716 m2) of the slab areas as requiring repair. This decision was ill-founded, however, as nearly 130,000 ft2 (12,077 m2) of delamination repairs were ultimately required.

Repair

Hydrodemolition was specified for rehabilitating the spillway slab (Fig. 5). The average calibration required water delivered at 20,000 psi (138 MPa) at a rate of 90 gal./minute (341 L/minute) for five machine revolutions to achieve a 6 in. (150 mm) minimum removal depth in one pass. Total removal depth varied from 6 to 18 in. (150 to 460 mm) to remove delamination (Fig. 6). In addition to the concrete removal, the hydrodemolition process did an exceptional job of removing rust, scale, and concrete from the bars in the slabs.

While it was observed that No. 4 reinforcing bars present in the slabs were deformed during the hydrodemolition process, the more prevalent No. 6 and No. 9 bars were not affected by the impact of the water jets at this calibration.

After the hydrodemolition was completed, the contractor manually removed shadowing (unsound concrete shielded by the reinforcement) with a 15 lb (7 kg) chipping hammer. Demolition also occurred in small areas with a 30 lb (14 kg) hammer. However, the larger hammer resulted in significantly greater microfracturing as seen in tests conducted per ASTM C1583/C1583M, “Standard Test Method for Tensile Strength of Concrete Surfaces and the Bond Strength or Tensile Strength of Concrete Repair and Overlay Materials by Direct Tension (Pull-off Method).”

The 18 in. (460 mm) thick spillway slab panels were 25 x 25 ft (7.6 x 7.6 m) in plan and were constructed with only a single mat of reinforcement at a depth of 9 in. (230 mm). The panels exhibited significant thermal cracking, and some of the cracks extended to middepth of the slabs. Because the panels were highly restrained by the spillway wall and gate structures, the repair mixture was designed to have minimal shrinkage.

Fig. 5: Hydrodemolition of the upper expansion joint at the Fort Randall Dam spillway (U.S. Army Corps of Engineers, Oct. 1, 2013)

Fig. 6: Delamination at a depth of 6 to 10 in. (150 to 250 mm) of the Fort Randall Dam spillway visible after initial removal (U.S. Army Corps of Engineers, June 16, 2014)

The selected mixture had a 4500 psi (31.0 MPa) compressive strength and comprised 1-1/2 in. (37.5 mm) maximum size aggregate (MSA), with at least 2% exceeding 1 in. (25.4 mm) size; polypropylene macro fiber; and a shrinkage-reducing/compensating admixture.

The large aggregate size was selected over a more typical 3/4 in. (19.0 mm) MSA because it would reduce the paste content in the mixture and thus provide about a 40% reduction in shrinkage cracking.3 Forta-Ferro® was used as the polypro-pylene macro fiber because it combines fibrillated polypropylene fibers with twisted bundle monofilament fibers. The fiber was anticipated to reduce shrinkage cracking, increase residual strengths, and enhance bond to the existing concrete. The designers also strove to choose a dosage that would provide significant residual strength per ASTM C1399/C1399M, “Standard Test Method for Obtaining Average Residual-Strength of Fiber-Reinforced Concrete,” yet not be detrimental to the compressive strength. Table 1 summarizes compressive and residual flexural strength data for mixtures with three different dosages of fiber.4 A dosage rate of 5 lb/yd3 (2.97 kg/m3) was selected for the repair mixture.

Based on data previously obtained for the Glen Elder Dam repair project, PREVent-C was approved as the shrinkage-

PREVent-C was approved as the shrinkage reducing/compensating admixture. The designers strongly believed that prevention of shrinkage cracking was a key to long-term durability of the repair given the temperature extremes as well as the consistent presence of seepage water subject to freezing and thawing. Using the shrinkage- reducing/compensating admixture in conjunction with the macro polypropylene fibers and large aggregate has resulted in negligible visible shrinkage cracking (Fig. 7) in a harsh environment.

Table 1:
Compressive strength and average residual flexural strength for Forta-Ferro fiber reinforcement[4]

Dosage rate, lb/yd^3 (kg/m^3)	Compressive strength, psi (MPa)	Average residual strength, psi (MPa)
3.0 (1.78)	6500 (44.8)	126 (0.87)
5.0 (2.97)	6110 (42.1)	185 (1.28)
7.5 (4.45)	5860 (40.4)	260 (1.79)

Fig. 7: A concrete overlay shown at 29 days after placement exhibits no shrinkage cracking. This overlay comprised a 25 x 25 ft (7.6 x 7.6 m) monolithic placement (U.S. Army Corps of Engineers, Oct. 11, 2013)

After placement of the overlay, tensile strength of the repairs and underlying substrate were determined per ASTM C1583/C1583M. Based on testing in undisturbed, undamaged locations of the spillway prior to construction, it was determined that the average tensile strength of the slab was approximately 258 psi (1.78 MPa). The specifications required that all repairs exceed this average tensile strength to ensure a quality bond between the repair and the substrate concrete. The average tensile strength of the repairs conducted was 262 psi (1.81 MPa).
Following the completion of the initial repair contract, the critical areas of spillway delamination at Fort Randall Dam were repaired. This project further demonstrated the economic viability of large-scale variable depth removal of concrete with hydrodemolition. Again, through the use of a high-performance mixture, shrinkage cracking was minimized while also meeting the high tensile strengths required by such a unique structure.

Ongoing Work

A shrinkage-reducing/compensating admixture—a blend of magnesium oxide and glycol ether—proved to be a good solution for crack mitigation in concrete placed in spillway slab repairs on the Glen Elder Dam and the Fort Randall Dam. The U.S. Army Corps of Engineers, in conjunction with the Bureau of Reclamation, is currently engaged in additional lab testing and performance field trials of shrinkage-reducing admixtures to evaluate their effectiveness. These activities include both laboratory and field research on innovative shrinkage-reducing/compensating admixtures used for concrete repair in combination with other means of improving performance, such as fiber reinforcement, internal curing, bonding conditions, and optimized aggregate gradations.
Laboratory-based studies will focus on length change and restrained shrinkage testing as well as the influence of shrinkage-reducing/compensating admixtures on the early-age microstructure of the concrete and long-term durability. Full-scale simulated repairs will also be performed in a laboratory setting with test slabs exposed to an aggressive environment (high temperature, low relative humidity, and wind) to accelerate shrinkage-induced crack generation. Best- performing materials and material combinations identified in the laboratory studies will be selected for future field demonstration projects within the U.S. Army Corps of Engineers.

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Note: Additional information on the ASTM standards discussed in this article can be found at www.astm.org.

Selected for reader interest by the editors.

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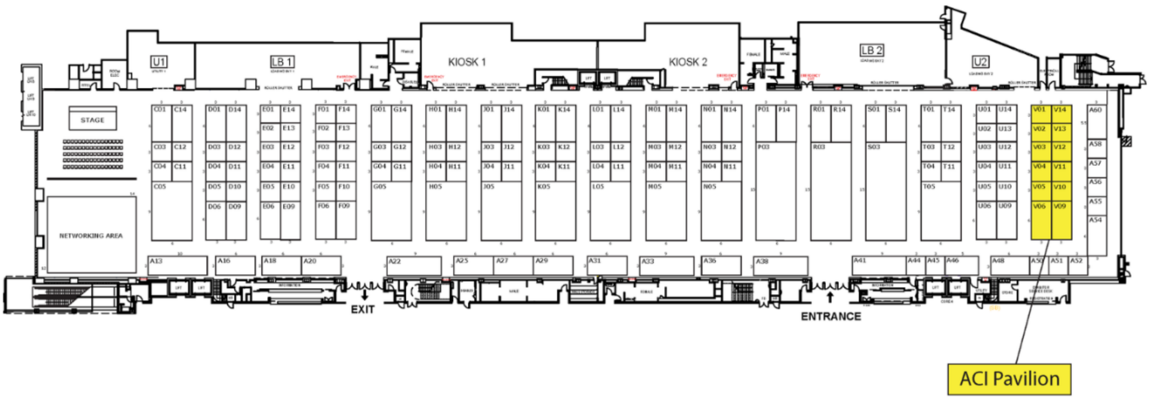

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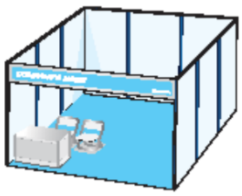

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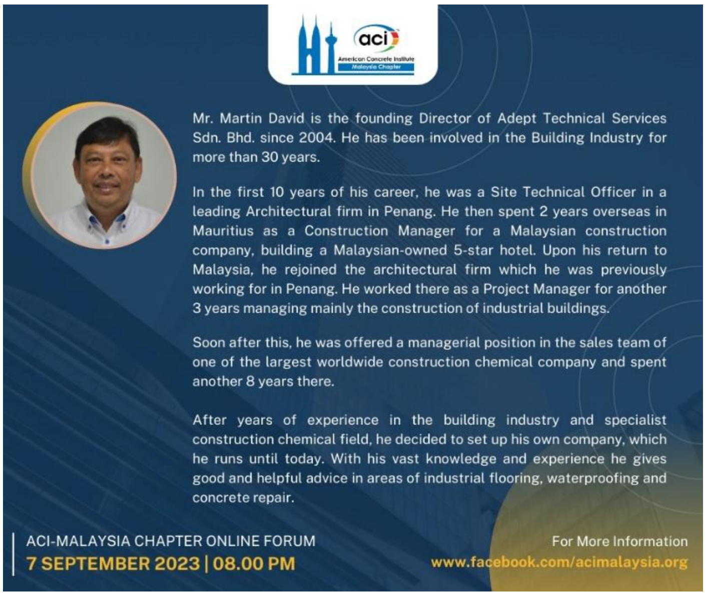

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