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Joint Venture Team

Mission Rock Lightweight Cellular Concrete Technical Advisory Panel

Technical Review Report (Addendum)

Volume 1 – Main TAP Report

Lightweight Cellular Concrete Technical Advisory Panel Members:

Steven Bartlett, Ph.D., P.E.

Stan Peters, P.E.

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May 07, 2020

1.1 Foreword (Authored By Port of San Francisco)

Current and future conditions at the Mission Rock development site, which encompasses about 16 acres underlain by both fill material and San Francisco Bay mud, present engineering challenges to mitigation of settlement and protection against Sea Level Rise (SLR).

To protect the site against SLR, the Mission Rock development plan proposes to raise the site by up to five and one-half feet, which is consistent with the recommended elevations of the ResilientSF plan (66" of SLR in year 2100). If the site were raised by placing soil fill, the weight of the fill would cause settlement of the underlying soils that is predicted to be up to 22 inches. The new roads and parks would settle with the soil while other structures, such as pile supported buildings, would not. This amount of future settlement would result in non-functioning infrastructure and would be unacceptable to both the City and the project sponsor.

On the surface, the Mission Rock site appears to be a large, ordinary parking lot used by San Francisco Giants fans. Below the surface are several layers of soil that consist of heterogeneous fill, rubble, and Bay Mud. The Bay Mud layer is largely responsible for the potential future settlement described above and it is relatively thick at this location, which increases the potential for settlement. At this point in time the Bay Mud has adjusted to the weight of the overlying fill and parking lot, which have been in place for a long time, and settlement has largely ceased. If additional weight were placed over the Bay Mud, it would respond by compressing and settlement would recommence.

Engineering the site to address future SLR combined with the presence of underlying Bay Mud requires a different approach than has previously been used to raise grade elevations for roadways in San Francisco. The Mission Rock Partners team has proposed a unique system to solve these challenges. The system consists of embedding stone columns throughout the site to improve soil stability, and raising the grade elevation of the site with lightweight cellular concrete (LCC), a material much lighter than soil. The raised site would be topped with typical City streets, pavements, landscaping, and utilities. This system is intended to provide a site resilient to SLR up to year 2100 and prevent street settlement that could break public infrastructure and allow streets to pull away from adjacent, pile-supported buildings. It is up to the City, with the advice from a group of technical and industry experts, to independently examine the Developer's proposal and set out the performance criteria needed for a viable solution that the City can support and permit for construction.

1.2 Executive Summary (Authored By TAP)

The Mission Rock developer team proposes to use lightweight cellular concrete (LCC) fill to limit settlement in the streets within the Mission Rock project. The Port of San Francisco (SFPort), The San Francisco Public Utilities Commission (SFPUC), and the San Francisco Department of Public Works (SFDPW) have convened a Technical Advisory Panel (TAP) to review the proposed use of LCC.

A workshop was held on September 13, 2019 where the Developer Team presented the proposed use of LCC. The TAP has engaged in an interactive process with the Developer Team, SFDPW, and SFPUC since that time to review the proposed use of LCC, proposed design documents and specifications, and other resource documents. Many issues and questions have been raised and resolved in that process. As of March 12, 2020 all comments and concerns raised by the TAP were resolved and documented in the Draft Technical Review Report dated March 12, 2020, which is included as Volume 5 of this report.

Based upon that report the SFDPW and the SFPUC have continued their own review and the TAP and the Developer Team have responded to their comments. As of April 27, 2020 the Developer has responded to all SFDPW and SFPUC comments and the TAP has reviewed those responses and found them satisfactory.

The TAP has determined that the EOR (Langan) has adequately demonstrated and supported the BOD by its evaluations, calculations, and field and laboratory test programs. The BOD is appropriate and sufficient to serve as design criteria for the use of LCC on the project. Further, the BOD has been sufficiently communicated via design and construction documents to the TAP. Therefore, it is the TAP's opinion that the LCC alternative is a reasonable, equivalent, and a safe alternative for use as engineered fill, backfill and pavement subgrade for the Mission Rock Project. We also conclude that the LCC can be functionally and safely integrated with the planned improvements, which are primarily subsurface public utilities planned for the project.

This Technical Review Report documents the issues raised and the resolution of those issues relative to the use of LCC on the Mission Rock Project. It also collects the important reference documents utilized during the review process. This Final Addendum to the Technical Review Report concludes the Technical Advisory Panel's tasks consistent with Public Works Order No. 202368, Public Works – Bureau of Streets and Mapping - Permits Division, conditions 8(b) and 8(c). The Preliminary Performance Criteria has been established per condition 8(b)(iv) recommending certain changes to the LCC Infrastructure Designs that when incorporated into the design will meet the standard of practice for reasonableness and technical merit.

1.3. Table of Contents

Volume 1: Main TAP Report

Volume 2: Comments and Responses

Volume 3: LCC Pilot Report

Volume 4: SIP LCC Design

Volume 5: Original TAP Report (March 12, 2020)

1.4 Introduction (Authored By TAP)

1.4.1 Background and Purpose

The Mission Rock developer team proposes to use lightweight cellular concrete (LCC) fill to raise the grade elevation of the streets within the Mission Rock project while at the same time limiting consolidation settlement. Lightweight cellular concrete fill has been used for similar purposes on other projects; however, it has not been used beneath San Francisco city streets, and several city agencies have raised questions and concerns about such use. The Port of San Francisco (SFPort), The San Francisco Public Utilities Commission (SFPUC), and the San Francisco Department of Public Works (SFDPW) have convened a Technical Advisory Panel (TAP) to review the proposed use of LCC for the Mission Rock project. The TAP panel members are:

Steven Bartlett, Ph.D., P.E.
University of Utah, Asia Campus
Incheon, Korea

Stan Peters, P.E.
Castle Rock Consulting
Denver, CO

Arul Arulmoli, Ph.D., P.E., G.E.
Earth Mechanics, Inc.
Fountain Valley, CA

As described in Section 8 of Public Works Order No. 202368, the TAP evaluates the technical merit, design assumptions, engineering studies, and engineering conclusions of the LCC Infrastructure Design provided by the project developer team as it relates to the use of LCC fill beneath the streets within the Mission Rock project.

The TAP evaluates the technical performance and safety of the LCC Infrastructure Design based on criteria and variables identified by its members and the affected City Departments, including:

1. the anticipated use of the public improvements located in and above LCC by property owners and users;
2. anticipated routine maintenance and repair of, and excavation in streets containing LCC for roadway repair, utility services, and other purposes;
3. the geologic, soils, and hydrology conditions of the Project site; and
4. the anticipated infrastructure changes, variances, and performance at the Project boundaries.

The scope of the Technical Review also includes the following:

1. Developing objective technical performance and safety criteria for the LCC Infrastructure, including but not limited to addressing the effects of settlement, uplift, and the rupture of a pipe embedded in LCC on the LCC Infrastructure ("Preliminary Performance Criteria"), based on well-established engineering principles, standards, and practices.
2. Analyzing the LCC Infrastructure Designs for consistency with the Preliminary Performance Criteria and analyzing how the LCC Infrastructure will interact with building foundations and sub-structures such as stone columns and superstructures, including review of calculations and mathematical modeling for the seismic response.
3. Identifying additional data, design specifications, or design changes required to decrease the likelihood of subsidence, uplift, or failure of the LCC Infrastructure
4. Developing the parameters for and supervising the LCC Pilot (described below).
5. Attending meetings with City or Subdivider meetings as needed and directed.
6. Providing Technical Review letter(s) to the City per requirements identified by the Affected City Departments.
7. Preparing required reports in connection with the items listed above. The Technical Review includes a report ("Technical Review Report") that summarizes the Technical Advisory Panel's findings, which shall establish the Preliminary Performance Criteria, and i) demonstrates to the City Engineer's satisfaction the reasonableness and technical merit of the LCC Infrastructure Designs; ii) recommends changes to the LCC Infrastructure Designs, if required; or iii) states that the LCC Infrastructure Designs, or any components of such designs, are unsafe or infeasible for the intended purpose and use. If the Technical Review concludes that the LCC Infrastructure Designs is unsafe or infeasible for the intended use, the LCC Infrastructure Designs will be disapproved.

1.4.2 LCC Infrastructure Description

The Mission Rock site exhibits heterogeneous geological conditions which vary across the site. Below the surface, there is a 10-30' upper layer of uncontrolled fill, which may be subject to liquefaction and lateral spreading during a seismic event. The uncontrolled filled layer is underlain by 40-80' of Young Bay Mud, which is very compressible and prone to settlement when subjected to even a small amount of new fill.

The developer team plans to utilize pile foundations for the Mission Rock buildings, which will be subject to negligible settlement over time. Since the adjacent roadways and parks will not be pile supported, those areas could experience settlement due to the additional weight placed on them to raise the grade for sea level rise as well as the weight of paving, sidewalks, etc. (See Figure 1). Future differential settlement between the buildings and the roadways and parks could result in various undesirable conditions including offsets or unacceptable slopes in paving, poor drainage, and damage to underground utilities.

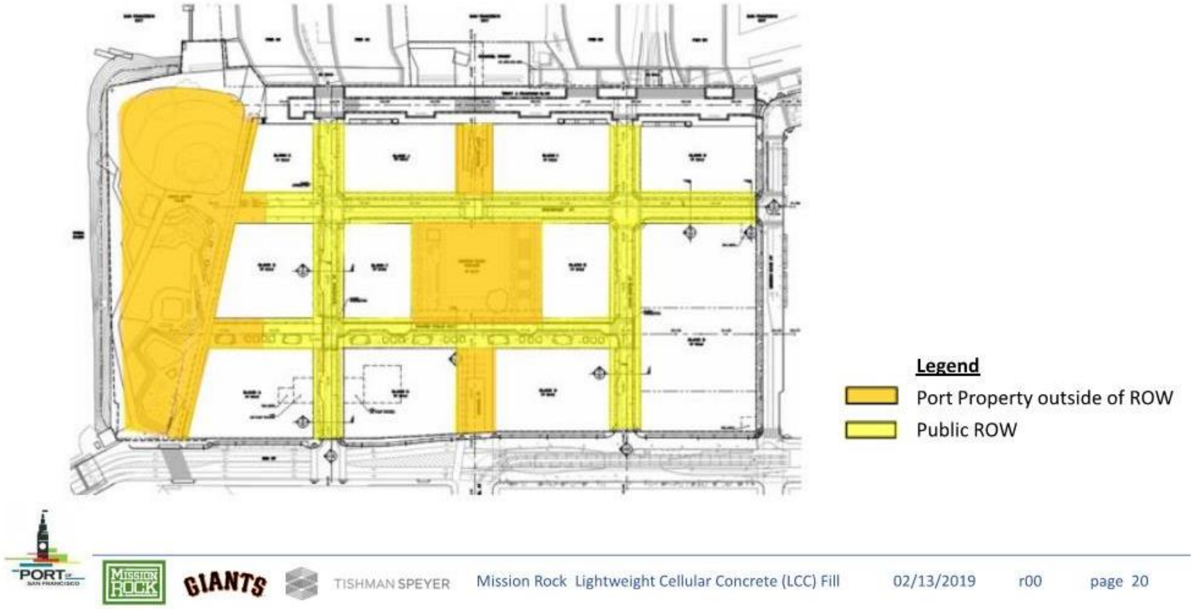

Figure 1: Locations of LCC Fill at Roadways and Parks

The developer team proposes to address this concern by limiting the potential for future settlement of the roadways and critical portions of parks by removing an upper layer of soil and replacing it with lightweight cellular concrete (LCC), which weighs significantly less than soil (See Figure 2). By choosing the appropriate depth of soil removal and replacement, the existing load on the underlying soil can be reduced such that the additional load due to raising the grade and building the streets can be accommodated without increasing the total load on the underlying soil. This approach endeavors to reduce future settlement of the roadways to small and acceptable levels.

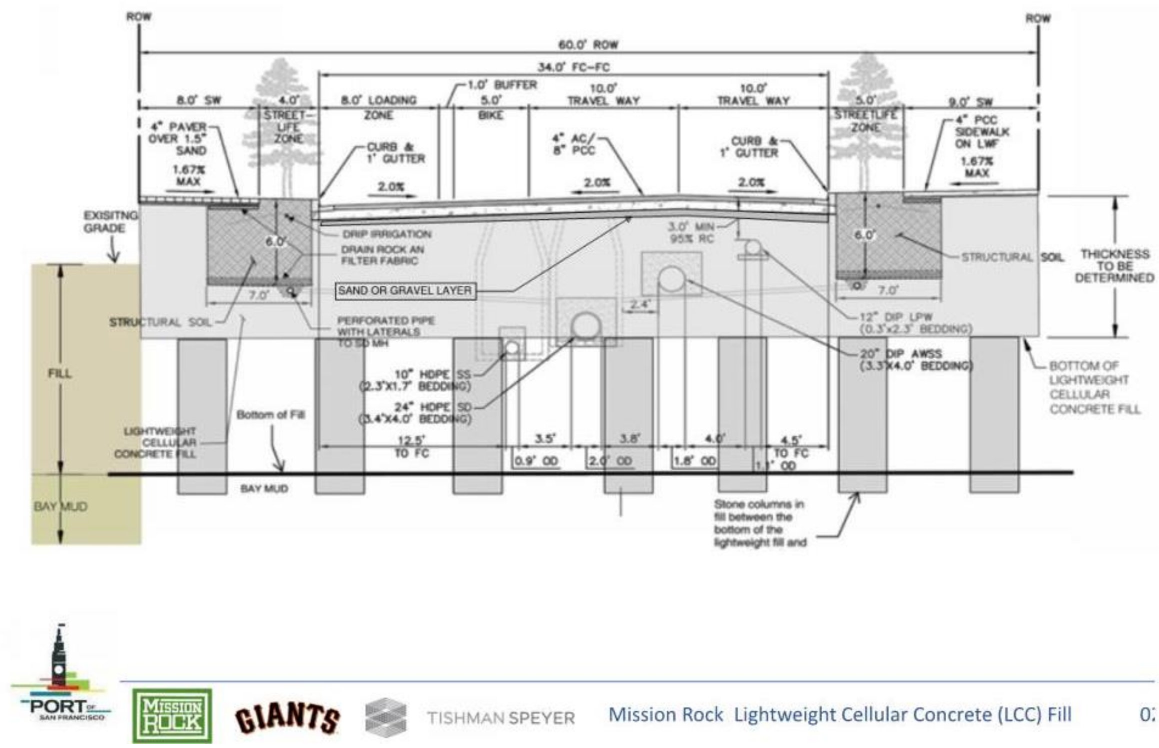

Figure 2: Typical Roadway Section with LCC Fill

1.4.3 TAP Process

A workshop was held on September 13, 2019 where the developer presented the proposed use of LCC. The TAP members as well as representatives of the three City agencies discussed the use of LCC and raised several questions. The developer team provided drawings, specifications, reports, and other documents useful for the TAP review. Written comments and responses as well as conference calls were utilized to raise and answer additional questions.

On October 11, 2019, the Preliminary Technical Review Report was distributed for review and comment by the City Agencies, Developer, and its consultants. The Discussion section of that report documented the significant issues that had been raised and discussed during the course of the TAP review of the initial documents. That Discussion section is reproduced below. The Recommendations section of that report established sixteen specific recommendations from the TAP panel regarding issues that required additional effort by the developer team to resolve.

All recommendations were initially addressed in a response provided by the Developer team on November 6, 2019. The TAP reviewed these responses and subsequently asked follow up questions and requested additional documentation. Over the ensuing months the Developer submitted revised documents and additional information and engaged in conference calls with the TAP where issues were discussed and resolved and requests for additional calculations and documents were made. Each of the sixteen recommendations were summarized in a comment log, which has tracked the progress towards resolution of each recommendation over the course of these discussions. The comment log is included in Volume 2 of this report. The Developer conducted a Pilot Test program and documented the results in a Pilot Test Report dated February 10, 2020, which is included in Volume 3 of this report. The TAP reviewed both the plan for the pilot test and the resulting report and generated additional comments specific to the Pilot Test. These comments have been resolved in a similar fashion to the original sixteen recommendations and that back and forth process is documented in the comment log included in Volume 3 of this report. At the point in these discussions that the issues were resolved, the TAP requested that the Developer Team formulate a Basis of Design (BOD) document that would satisfy the requirement in Section 8 of Public Works Order 202368 for a technical performance and safety criteria. The Developer Team submitted the BOD for TAP review. A similar process of discussions and revisions ensued until all issues were resolved. The final version of this BOD is included in Section 1.8 below.

As of March 12, 2020 all of the TAP's concerns with all of its comments have been resolved. SFPW and SFPUC have not necessarily concluded their own reviews. The Comment Log is included in Section 2.2.3 of this report.

Based upon that report the SFDPW and the SFPUC have continued their own review and the TAP and the Developer Team have responded to their comments. As of April 27, 2020 the Developer has responded to all SFDPW and SFPUC comments and the TAP has reviewed those responses and found them satisfactory.

This Technical Review Report documents the issues raised and the resolution of those issues relative to the use of LCC on the Mission Rock Project. It also collects the important reference documents utilized during the review process.

1.4.4 Organization of Report

This Technical Review Report is organized into five volumes, with the most central information located in Volume 1.

Volume 1 includes introductory material, a discussion of important issues raised by the TAP relative to the use of LCC, A summary of the original sixteen TAP recommendations as well as a brief description of how each was resolved. It also includes the TAP Conclusions and several reference documents including: the Geotechnical Basis of Design and Performance Criteria, the project LCC specification, an Excavation and Backfill Procedure, a discussion of Excavatability, a discussion of Durability, A discussion of Saturation Density Testing, A discussion of Factor of Safety for Buoyancy, and a recent Technical Memo Authored by Langan in response to several SFDPW and SFPUC concerns.

Volume 2 contains the comment logs utilized throughout the review process that document questions and concerns raised by the TAP and SFDPW, and SFPUC as well as the back and forth discussion and final resolution of each comment. It also contains several exhibits that were provided by the Developer Team to help resolve various issues.

Volume 3 contains the LCC Pilot Test Report produced by the Developer Team to document the Pilot Test and its results. Several videos taken during the pilot test are included in this volume. Due to the large file size of these videos they have been placed in a separate pdf file labeled Volume 3A.

Volume 4 contains Site Improvement Package drawings and calculations provided by the Developer Team during the review process.

Volume 5 contains the complete March 12, 2020 Draft TAP report. The present report is an addendum to the March 12 report and the previous report is therefore included in its entirety. Much of the information in the March 12 report is repeated in the present Addendum report.

Various materials included in this report have been authored by either the TAP, the Port of San Francisco, or the Developer Team. In Volume 1 the author of each section is indicated in parenthesis at the beginning of the section. In Volume 2 the comment logs have been passed back and forth with both the TAP and the Developer Team contributing and the authorship of comments and responses is generally attributed in a column heading or a color legend. The Exhibits A-K have been authored by the Developer Team. The documents in Volumes 3 and 4 have been authored by the Developer Team. Volume 5 largely replicates Volumes 1-4 and authorship is as described above.

1.5 Discussion Section from Oct. 11, 2019, Preliminary Technical Review Report (Authored By TAP)

A discussion of each significant issue reviewed by the TAP is as follows:

Applicable Codes

Changes to codes that may be applicable to the project may impact the earthquake ground motions used in the liquefaction analysis. The California Building Code changes will become effective on January 1, 2020, and the Caltrans Seismic Design Criteria changes became effective on September 1, 2019. The developer team should confirm which code is applicable to the project and evaluate the liquefaction potential and develop mitigation measures per that code.

Long-Term Settlement

The developer team estimates the anticipated differential settlement between pile-supported buildings and streets would be on the order of 1.5 inches, or less (Geotechnical Investigation Report, p. 17). However, the source of the differential settlement is not explained (i.e., recompression, primary or secondary consolidation settlement?). Generally, pile foundation groups are evaluated for recompression and primary consolidation settlement resulting from an increase in vertical stress in the layers under the pile tips.

However, secondary compression settlement has been included in this differential settlement evaluation. The developer team believes this long-term settlement will be negligible; however this may not be the case. See the additional discussion about this issue in the Construction Dewatering and Potential Disturbance of Sensitive Clay sections of this report.

Construction Dewatering

Pilot Test Area

Dewatering is proposed during the pilot test to allow placement of LCC in the dry. Dewatering increases the vertical stresses in the soil, which can trigger settlement of the Bay Mud. If dewatering is expected to lower the water levels in the areas surrounding the pilot test location, it could impact the infrastructure in the surrounding area. The developer team should evaluate this possibility and develop appropriate mitigation measures. A settlement monitoring program for the adjacent infrastructure before, during, and after the pilot test should be implemented and appropriate mitigation measures should be developed.in the event measured settlement exceeds the acceptable limit. This situation also needs to be addressed during construction of LCC for the entire project. The settlement monitoring program should be included in the dewatering specifications and plans.

The developer team has not fully explained the construction cut-off and dewatering methods (e.g., use of coffer dams, sheet piling, well points, pumping, etc.). The selected method(s) is also an important design consideration because it affects the depth and extent of the zone of influence of the dewatering.

The Proposed LCC Pilot drawings dated September 11, 2019 (R04.3) include a construction and testing sequence on sheet 4 of 5. The dewatering step should be included in the construction sequence. It is assumed that dewatering occurs after excavation (Step 1) and before the test liner is installed (Step 2). The developer should describe contingency measures that will be in place to mitigate the possibility of dewatering pump malfunction or failure during LCC placement (such as having spare pumps on site.)

Roadway Areas

The project team has suggested plausible counter-measures to be taken during construction to ensure that no significant recompression of the Young Bay Mud occurs in the soils below the proposed roadway areas; hence future creep settlement in this area resulting from construction dewatering activities is expected to be small (i.e., of no engineering importance).

Building Areas

The project team has suggested: "there may be some drawdown of groundwater within the building sites. *We have performed preliminary analysis and conclude that because the upper part of the Bay Mud is overconsolidated (by an average of 200 psf), an additional 3 feet of dewatering could occur without putting the Bay Mud into virgin compression. Durations of dewatering are anticipated to be on the order of 3-5 months for the deepest depth of dewatering; the depth of dewatering will be incrementally reduced as loads are applied to the LCC street sections. For the short durations that the dewatering will occur, there may be some localized slight recompression of the Bay Mud, which is then expected to rebound after the dewatering is stopped and the groundwater level comes back up. Furthermore, it appears, based on a comparison of recent topographic survey data to data from 9 years ago, that secondary compression not occurring. The temporary recompression of the Bay Mud would not cause new secondary compression."*

Currently, secondary compression (albeit at a very low rate) is probably still occurring at the project site, but the amount of settlement in a 9-year period may be below the resolution or accuracy of the topographical survey put forth by the applicant. However, the rate of secondary compression initiated by the placement of the fill many decades ago is not the issue. What is relevant is the evaluation of the new rate of secondary settlement caused by any potential recompression of the clay due to the proposed construction dewatering in the building areas.

Furthermore, the basis for the average 200 psf overconsolidation of the upper Young Bay Mud needs to be given to the TAP for review and documented in the project design documents. The geotechnical report provided to the TAP does not have any consolidation tests from specimens of the Young Bay Mud. All tested specimens were obtained from the Old Bay Mud, which is known to be slightly to moderately overconsolidated, but the properties of this deposit are not germane.

In addition, the applicant's response to comment 2 states: *"The highest observed groundwater level in the boring to date was 94'."* Therefore, the amount of drawdown for the "worst-case" evaluation has been incorrectly calculated. Instead, the lowering of the groundwater table would be from 94 to 88 feet, which is a 6 x 62.4 increase in vertical effective stress (i.e., 374.4 psf increase) in the building areas. This "worst-case" drawdown value will exceed the average 200 psf overconsolidation of the upper Young Bay Mud and potentially cause virgin compression, and the associated normally consolidate rate of secondary compression.

Nonetheless, it is not necessary for the Young Bay Mud to reach virgin compression to initiate an increase in the rate of creep (i.e., secondary) settlement in this deposit (see Section 2.3 of the attached thesis from MIT, Ng (1998).) Sections 2.3 and 7 of this report particularly Fig. 7.3 shows a creep ratio of about 0.8 or 80 percent of the normally consolidated creep value when the soil has been loaded in recompression to a value that approaches the "aged" preconsolidation stress. Furthermore, the proposed dewatering for 3 to 5 months is certainly sufficient time to allow for the occurrence of recompression, thus potentially initiating unanticipated long-term creep settlement of the Young Bay Mud.

The TAP agrees that upon ceasing dewatering and with some modest rise in the water table minor heave will occur, which will temporarily delay the initiation of the new round of creep settlement caused by the recompression of the clay during watering, but secondary settlement at a new and increased rate will most likely occur in these sediments after this delay. The amount of heave (rebound) is inconsequential to the project because it occurs during construction; however, the potential for any post-construction secondary settlement is important, because it can cause long-term differential settlement between the ground and the pile-supported structures.

Lastly, in its conclusion, the project team emphasizes: "*The temporary recompression of the Bay Mud would not cause new secondary compression.*" This statement is incorrect. There are theoretical work and laboratory testing referenced by the TAP in this section clearly shows this is not the case. Furthermore, inherent in the applicant's argument appears to be the assumption that the soil in recompression is behaving as a quasi linear-elastic material. While this may be relatively true for heavily to moderately overconsolidated deposits, it is not true for the recompression behavior of the Young Bay Mud. As the state of effective stress in this deposit increases to near a value near the preconsolidation stress, the recompression loading caused by dewatering causes irreversible strain in the soil fabric which cannot be fully recovered by an unloading event (i.e., allowing the water table to rise again after dewatering), and this permanent irrecoverable strain will produce an increase in the rate of secondary compression, albeit delayed, beyond that currently being experienced at the project site.

Therefore, this issue needs more careful evaluation by the applicant for the building areas.

Future Utility Installation and Repair

The project team has taken reasonable care in evaluating and designing for a "zero net load increase" on the Young Bay Mud resulting from the placement of LCC during initial construction. However, there is a concern that, despite the best efforts to prevent it, future construction or utility repair will be made by a third party and such construction/repair trench will utilize a backfill heavier than the LCC. The design team needs to develop contract provisions for the future placement or repair of utilities in the LCC treated areas (see recommendations).

Stone Column Installation

All members of the TAP agreed that this is the first project we have encountered where a light-weight fill solution is combined with a vibroreplacement method atop soft, sensitive clay deposits. This combination of the technologies requires careful evaluation regarding the construction and installation of the planned geosystem(s). In terms of the design of the ground improvement, there are two competing goals: (1) achieve sufficient densification in the overlying fill to obtain adequate earthquake performance, and (2) minimize any potential disturbance of the upper portion of the Young Bay Mud that might produce an additional amount of unanticipated, long-term settlement. The planned construction activities might reactivate the normally consolidated rate of secondary compression (i.e., $C\alpha$ normally consolidated) in a zone in the upper Young Bay Mud. The consequences of this may be an unanticipated increase in the rate of secondary compression and the associated long-term settlement.

The primary reason for the planned stone column treatment is to reduce liquefaction-induced settlement and horizontal movement (i.e., lateral spread) in the uncontrolled fill placed atop the Young Bay Mud.

However, the planned installation of stone columns has the possibility of disturbing the Young Bay from two primary mechanisms: (1) potential disturbance of the Young Bay Mud from vibration as the vibroflot approaches or penetrates the top of the Young Bay Mud, (2) potential changes in the in situ vertical and horizontal stress distribution resulting from the injection of a significantly stiffer and denser material into or near the top of the upper Young Bay Mud.

The design team has offered the solution to stop the stone column installation about 2 feet above the top of the upper Young Bay Mud. The team plans to use data obtained from the proposed prefabricated vertical drain (PVD) (i.e., wick drain) installation to obtain better information regarding the elevation of the top of the Young Bay Mud in the project extents.

The use of the PVD information to better define the depth of the Young Bay Mud seems to be a reasonable approach. However, the extent, degree, and potential consequences of potential disturbance need further evaluation as the design of the ground improvement progresses and is finalized (see the recommendation section).

Transition to Existing Streets

The planned LCC thickness tapers in the vicinity of existing streets outside of Mission Rock, which are expected to settle with time (Geotechnical Investigation, pp. 19-20). Two to three inches of settlement is expected at Mission Rock Street and up to one inch of settlement at

Third Street. The developer team has suggested that a tapered LCC fill will be used at such locales to provide a smooth transition between the existing settling streets and the new streets that will be designed to have minimum settlement. The developer team should evaluate the proposed tapered LCC transitions to confirm their effectiveness.

Ease of Excavation of LCC

Two values for the proposed design densities of non-pervious LCC appear in various documents and presentations (30pcf & 33pcf) but neither is shown on the Pilot drawings. Establishment of these values is important for construction& testing, and to ensure that the LCC can be readily excavated in the future when repairs or new utilities are required. Stan Peters began a discussion of Removability Modulus (RE) at the 9-13-2019 meeting but didn't have the opportunity to finish the discussion. See explanation on the attached spreadsheet on RE in Appendix D. Excavatability of LCC should not be a problem for future utility repairs, but a RE of 1.5 maximum should be specified for non-LCC backfill materials that may be allowed in small quantities, to ensure reasonable excavatability in the future.

Placement of LCC Fill

Several lift heights were discussed. The currently specified 36" miaximum lifts in the Specification for P-LCC is acceptable as normal industry-practice The previously placed lift setup time is currently specified as a 12hour minimum, but also requires 5psi before subsequent lifts can be placed. The specifications for LCC placement that ensure these lift heights will be successful, including QA/QC requirements should be defined by the developer team. Appropriate special inspections need to be defined; un-foamed slurry and final foamed density testing needs to be performed and held within tolerances of the submitted, approved mix designs to ensure proper w/c ratio for quality LCC. Suggested language is included in the Specification, which may be found in Section 1.9 below.

A silt-barrier geotextile fabric should be included on the excavation sides and below the LCC zone, and also in the landscape pockets above and below the structural soil. Mirafi 140NC is shown on the pilot project drawings.

Full Depth Permeable LCC

Cell-Crete has suggested using pervious LCC throughout the entire depth and would use Aerix foam solution throughout the LCC fill. That is preferred since it would alleviate some of the concerns of non-pervious (E-6 cm/sec) trapping air from escaping the pervious LCC layer below (1xE-1) as water levels rise. Having pervious all the way to the top would readily "vent" to atmospheric pressures, through tree planters, cracks, gaps, etc. By using pervious throughout, but at 30 for higher load-bearing, the air venting concerns go away.

Aerix Industries normally recommends using a silt-barrier, geotextile fabric between soft clays and their pervious LCC, to prevent migration of fines into the LCC, and clogging its pore structure. This is now shown on the pilot project plans, but could not be found in the SIP drawing details. Sheet 257 noted a filter fabric between the structural soil for tree plantings and the clean drain rock below it. This filter fabric should be also extended up along the vertical faces of the excavations, to prevent fines migration laterally into the pervious LCC. The Mikrafi 140 NC fabric is shown in the most recent Pilot Project drawings.

Absorption Testing

We had recommended including saturation testing as a function of hydrostatic head (1'-12'), to get effective UWs for Mission Rock. Aerix Industries (the pervious foam solution manufacturer agreed to fund saturation testing at 25pcf and 30pcf (bracketing the project's 27pcf) at hydrostatic heads of 1', 2', 3', 4', 5', 6', 8', 10' and 12'to determine percent saturation of the air voids, unit weights, and also a final "drained-out" density. Compressive strength tests and permeability tests were also included. Predicted values for the 27pcf pervious mixture are as follows ; naturally saturated from the bottom, 59pcf, with 68% of the voids filled; Drained-down density of 40pcf; 56psi strength at 7days, and permeability of 2E-1 cm/sec. The summary report is attached in Appendix F.

Buried Utility Construction Details

The developer team intends to place the LCC first and then install utilities in trenches that are either excavated into the LCC or were formed into the LCC when it was placed. Bedding should be wrapped in filter fabric, which was not consistently shown on all the details presented. Standard plans and specifications for water lines show imported dune sand for bedding and backfill, without a mirafi wrap. The standard Mirifai wrap was shown in a few drawings; Its use needs to be confirmed throughout.

Burst Water Lines

When a water line breaks in traditional backfill, the water first travels through the bedding, then eventually migrates to the surface and serves as a key indicator that a repair is needed, and where. This is unlikely to occur with pervious LCC fill even if pervious 30pcf is provided up to pavement levels because the water will follow the path of least resistance and travel laterally great distances through the LCC and probably not reach the surface.

Two other options of leak detection were discussed. Dr. Bartlett suggested a pressure-sensing system to help isolate leaks by low-pressure drops; the water utility doesn't have the staff or budgeting to oversee this methodology, and would prefer something "low-tech". the Developer suggested installing special rubber "boots" at valves and couplings, that would readily direct water leaks to the surface for normal detection in the event of a leak; They will search the availability of such devices and submit information for review.

Future Sourcing of LCC

A separate specification should be provided for a small batch LCC for emergency repairs. Require the Developer to establish a list of approved LCC providers available in the SF area, and make this available to permittees desiring to perform cuts in the street and LCC. This special district of LCC trench requirements should be GIS-located when obtaining permits.

Protection of Exposed LCC

The LCC fill will be cast above existing grade, and both the top surface and the vertical sides will be exposed for a significant period during construction. There was some discussion about protecting some of the vertical faces temporarily with soil berms (with vegetation, for visual aesthetics and erosion control). There was some discussion of limiting traffic on the unprotected top surface to keep it away from the unreinforced edges of the LCC fill. A detailed plan should be submitted for protection of the LCC. The effect on settlement of the surcharge loading from any berms should be investigated. The integrity of unprotected vertical faces should be investigated, and the face should be designed, including reinforcing steel/geo-textiles/integral fibers if it is required.

Where an adjacent building crawl space wall will adjoin the exposed vertical face of the LCC and provide protection for it in the permanent condition, the building will be designed without reliance on passive pressure from the LCC for seismic loading.

Long Term Strength Gain

Charts have been presented showing that strength gain of the LCC is negligible after 28 days. Strength gain charts for concrete frequently show significant strength gain after 28 days. A 25% increase (or more) after several years relative to the 28-day strength would not be uncommon. A more detailed analysis of the upper bound expected strength of the LCC is needed to ensure it can be easily excavated in the future. Estimated higher strengths could be used in the RE equation to verify it will remain excavatable.

The current specification indicates a maximum strength of 200 psi. The SFPUC water department is under the impression that a maximum of 100 psi is required to ensure it can be excavated in the future. Peters does not expect that 200psi will be exceeded in 28days with the 30pcf pervious mix, and even if it did, it will be readily excavatable for normal repairs. . See the attached file on Removability Modulus (RE) in Appendix D.

Long Term Durability

There could be long term durability concerns associated with the LCC material given that the LCC will be used below the water table and in close proximity to saltwater? Some discussion of this occurred on 9-13. While long term effects could be tested, the effects could take longer than the time allowed for decisions. Some discussion of accelerated salt-water testing occurred, about accelerated testing with increased salt content & elevated temperatures, similar to ASTM C1260 for alkali-silica reactivity in concrete, but false positives could occur with higher than normal chemical concentrations of the site brackish water. Discussions with Aerix Industries have shown some strength testing versus saturation has occurred with pervious LCC, with strength losses at different densities; see data and graphs in Appendix F. However, this was performed with potable water. No testing has been performed with salt water, to determine what the strength loss may be.

Subsequent to the October 4, 2019 TAP conference call, Aerix Industries has agreed to conduct such testing for strength loss, with potable water and on-site salt water samples. Details of the testing are underway to work with CRC and Aerix, to start this testing quickly so that strength result on the 7day and 28day results will be available for the final TAP report. Additional specimens will be fabricated for extended testing through 360days; should significant strength issues arise during early testing, there would be time to revise and improve initial strength of the mixture, priot to final project construction. Three normal C495 strength baseline, three saturated in potable and three in site water

Also, LCC does not have a high tensile strength capacity and undergoes shrinkage and thermal cracking. However, it is believed that such cracking will not greatly affect is load-carrying capacity and ability to adequately support the roadway pavement section.

Pilot Test

The pilot test is intended to demonstrate both the construction of the LCC roadway section, verify buoyancy and the compensating-load design performance, as well as a repair of a pipe buried in the LCC. If the data from the load cells are important to the demonstration of the design, then their purpose and arrangement should be described in more detail. Since the LCC will behave to some extent as a rigid body after initial set it is unclear whether the desired data will, in fact, be measured unless the LCC is constructed on void forms that would eliminate any bearing pressure on the soil directly below the LCC The proposed stress-gathering geo-pressure plates should perform acceptably, without the suggested collapsible, void-form. However the horizontal drainage geotextile panel beneath all LCC is needed to assure uniform upward water pressure is applied during GW testing. The target density of the upper LCC needs to be shown on the pilot drawings, as either 30pcf of 33pcf. When will the cellular contractor selected for the pilot submit mix designs for review?

During construction of the pilot project, extra cylinders of LCC material should be made by the testing lab for various tests that the TAP may deem appropriate. Compressive strength at 1, 7 & 28 days, and some additional specimens for testing at 56, 90 and 120days, to address the potential long-term strength gain question. Also, permeability tests, with the permeability samples first tested for absorption and effective unit weight, based on the existing groundwater elevations at the pilot project locations, should be performed on site-batched LCC.

A detail has been presented for a settlement isolation joint in some utilities where it exits the LCC. The Developer Team should submit manufacturer's product information demonstrating that the isolation joint can accommodate the anticipated differential settlements.

1.6 Summary of Original Recommendations and Final Resolution (Authored BY TAP and Developer Team)

Following is a summary of the 16 original recommendations developed by the TAP after their review of the project and the documentation submitted by the Developer. These recommendations served as a basis for the ensuing review and discussions with the Developer Team. The path to resolution of each recommendation is documented in the comment log in Section 2.2.2 and a summary of the resolution is included here.

Recommendation 1 – Applicable Codes

The project team should confirm which code is applicable to the project and evaluate the liquefaction potential and develop mitigation measures per that code.

> Response and Resolution: Langan responded that the Phase 1 *"project will be permitted now under the 2016 San Francisco Building Code. The applicable code for future phases will be updated with the current code at the time the phase is designed."*

Tap concerns with this recommendation have been resolved.

Recommendation 2 – Long Term Settlement in Building Area

The amount of long-term secondary settlement of the Young Bay mud resulting from dewatering activities outside the footprint of the roadway has not been fully addressed by the project team. The potential amount of long-term settlement between the pile-supported structures and the surrounding ground has not been evaluated. Furthermore, the current estimate of 1.5 inches of differential settlement offered by the project team does not appear to include any consideration of secondary compression settlement.

Therefore, the TAP recommends a zero net load be achieved for all project areas. The implications of this are that any significant loading event (e.g., placement of soil and LCC, stone column installation, construction dewatering, etc.) be proceeded by sufficient sub-excavation and removal to offset the potential increases in vertical stress caused by the proposed loading event(s). Based on our understanding of the project and its constraints, it appears that the applicant has the means and methods to accomplish this during construction.

> Response and Resolution: Langan detailed the issue of total and differential settlement in their updated *report dated 31 October 2019, stating: "The results of consolidation testing in the Phase 1 Development site indicate the Bay Mud is generally slightly overconsolidated, but may be normally consolidated in some areas. Accordingly, we judge consolidation is complete under the existing fill loads that were placed in the late 1800s to early 1900s. These results are consistent with the thickness of the Bay Mud, the length of time the fill has been in place, and the history of site use. Based on consolidation theory, after primary consolidation is compete, soils that are subjected to a sustained load at their maximum past pressure (i.e. normally consolidated) will undergo strain-related movements associated with clay particle deformation (a phenomenon called secondary compression), leading to a small amount of future settlement over time. If secondary compression were ongoing at the site, we would calculate about ¼ to ½ inch of settlement in the last 8 years using published coefficients ($C\alpha$) for estimating secondary compression. However, the Bay Mud is overconsolidated, and a comparison of survey data between July of 2011 and 2019 shows no measurable settlement has occurred; therefore, the amount of secondary compression at the site is likely very small. In addition, the planned construction across the Phase 1 Development site will result in a net unload on the Bay Mud."*

Tap concerns with this recommendation have been resolved.

Recommendation 3 – Construction Dewatering

The developer team should develop construction provisions or construction monitoring methods that define the extent, depth, and length of time that dewatering activities are allowed to maintain an operational "safe envelope." These construction provisions should include a reasonably detailed construction sequencing plan that maintains this "safe envelope." The project team should develop construction monitoring documentation that verifies key design assumptions and inputs have been maintained within the "safe envelope" and have met the requirements of contract provisions.

As the construction activities proceed and information is gained from the initial monitoring data, these can be used to inform or confirm the design (i.e., confirm the "safe envelope"). Subsequently, monitoring and installation requirements may need to be revised using the observational method championed by Ralph Peck (1969).

> <u>Response and Resolution</u>: The project *team provided data and discussion regarding the low groundwater level at the site and that the compressible Bay Mud has been subjected to repeated cycles of groundwater fluctuation over more than 100 years. Therefore, where groundwater will be required to be lowered below the average typical low groundwater level (Elevation 90 feet), mitigation measures will be taken to offset the potential stress increase associated with the planned dewatering. Langan also discussed that the planned excavations for the placement of the LCC are generally above Elevation 92 feet; therefore lowering the water 2 feet below the excavation depth will not lower the groundwater in the surrounding areas more than Elevation 90 feet. However, in isolated areas, the excavation for the LCC will likely range from Elevation 88 to 92 feet, and the required dewatering will extend 0 to 4 feet below the average typical low groundwater level of 90 feet. In these areas the project team indicated that site grades will be temporarily lowered to offset the potential for increasing load to the underlying Bay Mud.*

TAP concerns with this recommendation have been resolved.

Recommendation 4 – Backfilling for Future Utilities and Emergency Repair

The design team should develop construction provisions/recommend practices regarding backfill placement for 1) emergency repair situations, 2) future, new construction of larger trenches that required adherence to LCC placement provisions, and 3) smaller trenches or situations that do not require any special limitations or consideration in terms of construction and placement of backfill.

> <u>Response and Resolution</u>: The MRP provided a proposed Excavation and Backfill Procedure for LCC in Mission Rock Streets, which may be found in Section 1.11 of this report.

Tap concerns with this recommendation have been resolved.

Recommendation 5 – Stone Column Design and Installation

Additional evaluations are needed during the design of the stone column treatment to understand better how stone column installation may potentially affect the long-term settlement behavior of the upper part of the Young Bay Mud. These evaluations should present additional information about the strength and consolidation properties of the Young Bay Mud, including (1) detailed evaluation of elevations to the top of the Young Bay Mud throughout the project site, (2) estimates of the thickness and degree of possible disturbance or increases in stress resulting from stone column installation, (3) estimates of C_α for the Young Bay Mud (Mesri and Castro (1987).)

The additional data needed for these evaluations might be obtained using in situ measurement (CPT, vane shear, dilatomer (DMT), CPTU pore pressure dissipation, etc.) or other testing and/or additional laboratory evaluations of undisturbed soil specimens. Such information, if required, can be gained during the planned stone column installation test program.

Also, detailed requirements should be developed to serve as a basis for provisions in the stone column installation contract(s).

These provisions should provide requirements about the maximum allowed depth of stone column installation and other construction control practices to be implemented to minimize potential disturbance. The adherence to these provisions should be documented during construction.

> Response and Resolution: As discussed in *Section 8.1 of Langan's 31 October 2020 report: "To minimize the disturbance in the underlying Bay Mud, we recommend stone columns terminate at the bottom of the liquefiable fill, or one to two feet above the underlying Bay Mud, whichever is shallower." Further, additional language was added to the stone column specification stating that wick drains shall be installed prior to stone column installation and that detailed records of the wick drain depths and load-cell pressures will be kept by the contractor and relayed to the geotechnical engineer, who will in turn recommend the final depths of the required stone columns. Stone columns will be required to be terminated 2 feet above the top of Bay Mud. Based on the work performed during stone column test sections this method is achievable and works well in the field.*
>
> TAP concerns with this recommendation have been resolved.

Recommendation 6 - Earthquake Considerations for LCC

The project team should include in its report(s) more information about the design basis earthquake. This should include the discussion of expected site/soil amplification effects, the design peak ground acceleration, and the expected level of ground motion within the LCC backfill. This information is needed by the TAP and others (e.g., utility and pipeline designers) to complete their engineering evaluations.

Central to these evaluations is the question "What is the magnitude of seismic demand placed on the LCC backfill in terms of the peak cyclic shear stress caused by the earthquake?"

The design team needs to evaluate the potential for fracturing of the LCC due to the seismic loading in the case where the peak cyclic shear stress may exceed the peak shear strength of the LCC. If so, the design team should evaluate whether or not the stiffness of the LCC would be sufficiently degraded so as to impact its long-term function and performance. Tiwari (2018) has presented dynamic properties and shear behavior of LCC undering cyclic loading which are deemed sufficient for preliminary evaluations. These can be used in conjuction with other evaluations to determine the dynamic loads induced in the LCC by earthquake shaking. Additional laboratory testing may be required in the event that the LCC does not have sufficient strength to prevent shear fracturing from the peak cyclic shear stress.

Because the LCC is a relatively weak and brittle material, its strength will not be sufficient to provide significant resistance to earthquake shaking when placed as an “apron” around pile-supported buildings. Therefore, in terms of the design of the deep foundation systems buildings, the foundation designers should not use any passive resistance from the LCC in their design calculations. Also, any consequences of cracking of the LCC apron should also be evaluated, as appropriate.

In addition, the planned bedding or wrapping materials placed around utilities placed in the LCC should be clearly identified in all project drawings and documents. Furthermore, their interface properties (i.e., material stiffness, coefficient of interface friction, adhesion, cohesion, etc.) are often required by utilities to complete their seismic and other pipeline evaluations. These interface properties may vary according to the type of pipe (rigid vs. flexible), material used for the pipe (concrete, steel, plastic etc.) and the diameter of the pipe size. The specific design information needs of the various utilities are difficult to pre-determined, but the project team needs to identify any special embedment or bedding requirements.

> Response and Resolution: Langan has provided a calculation showing the estimates of shear stress are lower than the shear strength of the LCC. In addition, a detailed discussion regarding the likely seismic performance of the LCC was provided in the Lightweight Cellular Concrete Geotechnical Performance Goals and Design Criteria document dated 4 March 2020 provided by Langan.

TAP concerns with this recommendation have been resolved.

Recommendation 7 - Buoyancy During Construction

It is unclear whether or not LCC may undergo buoyancy uplift during construction. The buoyancy calculations performed by the design team need revisions in light of the recent testing done by Castle Rock Consulting. In addition, these calculations need to evaluate the potential for buoyancy uplift for temporary/interim conditions where dewatering may have been discontinued or interrupted. These should be done using laboratory tested values of LCC and with soil unit weights having realistic degree of saturation. Also, it is anticipated that the pilot LCC installation will produce key information for these evaluations.

> Response and Resolution: It will be the responsibility of the contractor to ensure the LCC does not become buoyant during construction. Additional language was included about protecting the LCC in Section 3.5 of the LCC specification. If the LCC does become

buoyant and is damaged, the damaged LCC will be removed and replaced by the contractor.

TAP concerns with this recommendation have been resolved.

Recommendation 8 – Long-Term Durability in Brackish Water

Some testing should be performed to determine what the compressive strength losses will be when saturated with the brackish water on-site, at least through 28days. Details for testing can be given.

> Response and Resolution: Testing has been performed comparing the compressive strength of cylinders cured in moist conditions, fresh water, and brackish water obtained from the site. The initial test results show no negative impact of brackish water and no impact is anticipated.

TAP concerns with this recommendation have been resolved.

Recommendation 9 – Protection of the Pervious LCC from Fines Infiltration

A suitable silt-barrier geotextile filter fabric should be installed before placing pervious LCC in any excavation, to prevent migration of clay fines and clogging the pores. This fabric should also be used in the bottom and sides of any excavations before placing structural or horticultural soils for tree plantings, etc.

> Response and Resolution: A filter fabric, such as Mirfi 140NC, will be placed between the LCC and any exposed soil surface, see Section 3.7 of the LCC specification.

TAP concerns with this recommendation have been resolved.

Recommendation 10 – Waterline Leak Detection

The developer team should propose a method to identify and locate leaks in pipes that are embedded in LCC since the porosity of the LCC will prevent water from rising to the surface where it is visible. The developer team prefers a system where the pipes are contained within waterproof plastic sleeves with boots at valves and other risers to direct leaking water to the surface. The developer team should document this system with manufacturer information and submit it for review.

> Response and Resolution: MRP demonstrated during the Pilot Test that a leak can be successfully seen through a valve riser to allow for appropriate future repair.

TAP concerns with this recommendation have been resolved.

Recommendation 11 - Pavement Design

The LCC acts as a base/subbase material for the planned pavement section. However, it is unclear how its stiffness was incorporated into the pavement design calculations. The geotechnical report requires a minimum compressive strength of 80 psi and 40 psi for the closed cell and open cell LCC, respectively (Section 8.2, par. 1). However, no requirements or data are given for what is the corresponding CBR value, modulus of subgrade reaction, or resilient modulus for the LCC materials? The long-term performance of the pavement (serviceability) is dependent upon the stiffness of the material which is not directly related to its unconfined compressive strength.

Furthermore, the testing of the LCC presented by the design team consists of unconfined compression testing on unsaturated specimens. The influences of saturation and mild confinement on the properties of the LCC should also be explored to verify that there is no significant loss in strength or stiffness of the material as it becomes saturated or buried and subjected to low-strain repetitive loading. This evaluation should be done for all LCC materials that are proposed to be placed directly under the pavement.

Also, the design vehicle load and the design service life of the pavement are not stated in the technical reports. Because the LCC plays a role in pavement support and affects the pavement service life, these calculations and the assumed properties for the LCC should be provided for review.

In addition, The SFPUC has requested to observe the performance of the LCC when a maintenance vehicle is parked atop a sidewalk zone in the LCC treated area with a gross vehicle with of 26,000 lbs. Also, municipal engineers sometimes request that a fire-truck loading be considered to verify that it will not significantly damage the LCC treated area in the event the limits of the roadway.

Currently, it is unclear what would be the consequences of such an event. If this field test is carried out, the test should be planned to gain information useful to verify the pavement design parameters or assumptions.

Before performing the test(s), it is recommended that the pavement designer evaluate this extreme loading case to see if potential cracking might occur from the truck loading. Also, it is recommended that plate load tests be conducted prior and after the vehicle loading to evaluate potential changes in vertical stiffness. Lastly, careful documentation should be made of any deflection or distress caused by the loading. It may be possible for the planned pilot LCC testing to incorporate these evaluations and tests.

The design team should evaluate the consequences of unplanned vehicles trafficking on LCC areas that are unprotected by pavement. If damage is possible, the team should provide guidelines regarding the possible repair or replacement of the LCC, when deemed necessary.

> Response and Resolution: Langan has provided a calculation showing the pavement section with LCC as a subbase, even with a reduced modulus is still adequate to support vehicular loads.

TAP concerns with this recommendation have been resolved.

Recommendation 12 – Compressive Strength of Saturated LCC

The developer should perform testing of compressive strength of LCC cylinders when saturated with both brackish (saltwater) and on site ground water, The effects of saturation of both types of water on the compressive strength should be determined from these tests and submitted to the City for review.

> Response and Resolution: Testing has been performed comparing the compressive strength of cylinders cured in moist conditions, fresh water, and brackish water obtained from the site. The initial test results show no negative impact of brackish water and no impact is anticipated.
>
> TAP concerns with this recommendation have been resolved.

Recommendation 13 – Tapered LCC Transitions

The developer team should evaluate the proposed tapered LCC transitions to confirm their effectiveness.

> Response and Resolution: Langan responded *"The overall engineering design approach is to unload the Bay Mud by 10 percent at locations beneath the LCC. Therefore, once the weight of the pavement thickness, improvements are accounted for, in addition to unloading by 10%, the tapered section of LCC is still on the order of 5 to 7 feet thick. Therefore it may not look significantly tapered at locations where the LCC meets the adjacent roadways.*
>
> *Additionally, the LCC section includes unloading of the underlying Bay Mud. The stress decrease from the LCC decreases stress in the area beyond the footprint of the LCC. Therefore, if there is ongoing settlement in 3rd Street, the use of LCC will allow for a more gradual differential settlement from this unloading."*

TAP concerns with this recommendation are resolved.

Recommendation 14 – Placement of LCC Fill

The specifications for LCC placement that ensure 36" lift heights will be successful, including QA/QC requirements, should be defined by the developer team. Appropriate special inspections need to be defined; un-foamed slurry and final foamed density testing needs to be performed and held within tolerances of the submitted, approved mix designs to ensure proper w/c ratio for quality LCC. Field testing of porosity during construction is also needed to ensure the LCC is within the specified limits and consistent with the assumptions used for buoyancy and load balancing calculations.

A silt-barrier geotextile fabric should be included on the excavation sides and below the LCC zone, and also in the landscape pockets above and below the structural soil

> Response and Resolution: The specification has been updated.

TAP concerns with this recommendation have been resolved.

Recommendation 15 – Future Sourcing of LCC

A separate specification should be provided for small batch LCC for emergency repairs. The developer team should establish a list of approved LCC providers available in the SF area, and submit it to the City so it can be available to permittees desiring to perform cuts in the street and the LCC.

Response and Resolution: MRP provided the response, *"LCC Repair and Backfill Procedure has been revised to allow non-permeable LCC backfill in limited areas. This would allow LCC to be made with foaming agents from different manufactures rather than just Aerix, which has the sole patent for permeable foaming agent. In these repairs non-permeable LCC can be placed above Elevation 95 feet or in localized trenches that with a volume less than 10 cubic yards."*

TAP concerns with this recommendation have been resolved.

Recommendation 16 – Pilot Test

The Developer should submit a written narrative description of the Pilot Test including objectives, construction sequence, and testing methodology.

A detail has been presented for a settlement isolation joint in some utilities where it exits the LCC. The Developer Team should submit manufacturer's product information demonstrating that the isolation joint can accommodate the anticipated differential settlements, which will not be tested as part of the Pilot Test.

The developer should perform a test as part of the Pilot where a typical maintenance vehicle is driven on the bare unprotected LCC and also parked. Any damage to the LCC surface should be noted and the depth of damage determined. This will inform any future repairs that must be made due to damage that may occur during construction.

Response and Resolution: MRP provided a Pilot Test plan and narrative, and final report after the Pilot Test was completed.

TAP concerns with this recommendation have been resolved.

1.7 TAP Conclusions and Recommendations (Authored By TAP)

A formal engineering design review process requires that the Engineer of Record (EOR)[1] demonstrates that the basis of design (BOD)[2] meets the standard of engineering practice[3]. Specifically, the technical advisory panel (TAP) in its scope of work was requested to "evaluate the technical viability, performance, safety, maintainability and operability of light-weight cellular concrete (LCC) and the integrated improvements, as specifically proposed for the Mission Rock Project."

[1] Engineer of Record means a professional engineer who seals drawings, reports, or documents for a project. The seal shall acknowledge that the professional engineer prepared, coordinated, or had subordinates prepare under the direct supervision of the professional engineer, drawings, reports, or documents for a project. https://www.lawinsider.com/dictionary/engineer-of-record

[2] Basis of Design constitutes the performance goals, design criteria, and other requirements, considerations, components, features, and primary assumptions evaluated by the Engineer of Record to implement the design and construction of the engineered feature or work. The basis of design must be communicated and documented, and its requirements presented in construction documents via calculations, typical drawings, and details, specifications, special provisions, etc.

[3] Standard of practice in terms of code enforcement means that the engineer of record designs or makes calculations and specifications accord to acceptable engineering practice, standards or methods and does so with the stamp of a registered design professional as required by the law of the particular jurisdiction.

The TAP was required to make "recommendations to the City on whether the proposed design is reasonable, equivalent to alternative designs, and safe given the site conditions." In terms of the BOD, "reasonable" design is one that meets the standard of engineering practice. Likewise, an "equivalent" design is one that provides equivalent or similar performance when compared to other alternatives. Lastly, "safe" design is one that demonstrates that the BOD meets the required factors of safety or performance goals as embodied in the standard of engineering practice.

In summary, the TAP has determined that EOR (Langan) has adequately demonstrated and supported the BOD by its evaluations, calculations, and field and laboratory test programs. The BOD is appropriate and sufficient to serve as design criteria for the use of LCC on the project. Further, the BOD has been sufficiently communicated via design and construction documents to the TAP. Therefore, it is the TAP's opinion that the LCC alternative is a reasonable, equivalent, and a safe alternative for use as engineered fill, backfill and pavement subgrade for the Mission Rock Project. We also conclude that the LCC can be functionally and safely integrated with the planned improvements, which are primarily subsurface public utilities planned for the project.

A few issues generated substantial discussion during the review process. The question of excavatability of LCC, particularly after long term strength gain, was carefully considered. An emphasis on the removability modulus rather than compressive strength for this purpose is preferred and it is the TAP's opinion that LCC meeting the project specifications will be sufficiently excavatable. The TAP has included a discussion of this issue in Section 1.12. Durability of the LCC material was another subject of much discussion. It is the TAP's opinion that LCC meeting the project specifications will be sufficiently durable and the TAP has included a discussion of this issue in Section 1.13. Field test data during the Pilot Test showed surprising scatter in saturated density of the LCC. The TAP concluded that the scatter was a result of the field test procedure and not indicative of variability in the LCC material itself. It is the TAP's opinion that LCC meeting the project specifications will exhibit sufficiently narrow scatter in saturated density to justify the average values used in design calculations when tested correctly. The TAP has included a discussion of this issue, including recommended modifications to the testing procedure, in Section 1.14. The question of appropriate factors of safety was considered carefully. It is the TAP's opinion that the factors of safety used in the design are appropriate and the TAP has included a discussion of this issue in Section 1.15.

Appendices included in this report consist of documents reviewed by the TAP during the course of their evaluation of LCC for the project and their discussions with the Developer Team. Many of these were provided by the Developer in support of the project design and some were provided by TAP members. Appendices that consist of design documents or calculations by the Developer Team that pertain to the use of LCC on the project have been found acceptable to the TAP. Appendices that consist of academic papers or other references have been used for reference and are included in the report to document the materials that were reviewed by the TAP.

1.8 LCC Geotechnical Performance Goals & Design Criteria (Authored By Developer Team)

4 March 2020

Mr. Steve Minden
Mission Rock Partners, LLC
c/o Tishman Speyer
One Bush Street, Suite 450
San Francisco, California 94104

SUBJECT: Lightweight Cellular Concrete Geotechnical Performance Goals and Design Criteria
Mission Rock – Phase 1 Horizontal Development
San Francisco, California
Langan Project No. 750604203

Dear Mr. Minden:

This letter presents our recommended geotechnical design criteria for raising street grades with compensating lightweight cellular concrete for the Mission Rock Phase 1 Horizontal Development project in San Francisco, California. The results of our geotechnical investigation for the horizontal components of the Mission Rock Phase 1 project were presented in a report dated 31 October 2019. Information provided here is based on the subsurface conditions documented in the 31 October 2019 report and on the conclusions and recommendations provided in that report. Anyone relying on the recommendations here should be familiar the subsurface conditions, assumptions, and conclusions provided in that 31 October 2019 report.

Background

Existing site grades within the Phase 1 Development area are from about Elevation 97 feet to about 101.5 feet[1]. Site grades for the future streets and sidewalks will be raised to accommodate future sea level rise, with planned street grades up to about Elevation 104 to 104.5 feet and sloping down to meet the existing street grades at 3rd Street. If conventional soil fill is placed to raise grades, the load from this new fill would result in consolidation settlement in the underlying compressible clay (known locally as Bay Mud). Consolidation would be unacceptable for the project requirements. Therefore, the project team has elected to raise grades using permeable lightweight cellular concrete (LLC). However, because the LCC and street improvements will apply some load, some existing soil at the will need to be overexcavated and replaced with LCC to offset all new loads.

Additionally, the remaining fill below the LCC section is potentially liquefiable and can cause erratic settlement. Therefore, the project team has elected to improve the fill below the LCC to mitigate liquefaction.

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1 Elevations based on topographic survey by Martin Ron, dated 2 July 2019, Mission Bay Datum (Old San Francisco City Datum plus 100 feet).

Performance Goals and Design Criteria for LCC

The geotechnical aspects of the LCC performance goals and design criteria for meeting those goals includes the following:

- Constructing streets and rights of way (ROW) founded upon LCC to limit future settlement and heave to acceptable levels and prevent hydrostatic uplift caused by sea level rise.
    - The new loads should be offset by at least 10 percent by overexcavating existing fill to a sufficient depth and replacing it with LCC.
    - The new LCC and overlying street sections should be designed to resist hydrostatic uplift with a factor of safety of at least 1.2 and 1.1 against future groundwater rise to the potential future (year 2100) mid-range and high-range groundwater level of 97.0 and 99.5 feet, respectively.
- Providing a suitable pavement substrate for the anticipated traffic loading.
    - LCC should be sufficiently strong to resist crushing under anticipated loading, including self-weight and from overlying improvements and temporary loads, such as heavy vehicle wheel loading.
    - LCC should be sufficiently stiff to provide an adequate substrate for the San Francisco standard pavement design.
- Allowing for future utility installation or repair using standard equipment, tools and methods.
    - LCC should be excavatable to allow for underground utility installation, repair, or other maintenance.
- Providing earthquake performance consistent with, or better than, traditional roadway construction in San Francisco.
    - LCC should perform adequately to provide vertical support of the roadway after a major earthquake.
    - Cracking of the LCC resulting from a major earthquake should not result in a significant decrease in the pavement lifecycle.
    - Pavement repair following a major earthquake should be equal to or less severe than traditional construction in San Francisco.

Geotechnical Evaluation and Engineering

Details regarding each of these criteria and the engineering background for each are provided in the sections below.

Load Compensation

To reduce the potential for new primary and secondary consolidation settlement caused by raising site grades and installing street improvements, the existing fill should be removed to a specified depth, and the resulting overexcavation should be backfilled using permeable LCC. The bottom elevation of the lightweight fill section should be determined such that the effective stress on the top of the Bay Mud following placement of the improvements is at least 10 percent less than the existing effective stresses. This reduction in effective stress will result in a "factor of safety" for net unloading (removed load/new load) of at least 1.1.

Within the new 60- to 70-foot-wide ROW, there will be new utilities, streets, sidewalks, light poles, and tree-planting areas between the building parcels. The evaluation for the required depth of overexcavation includes the weight of these new improvements, including the loads from new utilities, utility bedding and shading, the street and sidewalk pavement sections, trees, light poles, structural soil, and the increased density of improved fill that remains below the LCC. The following assumptions are included in calculating the required depth of overexcavation and placement of the load-compensating open-cell (permeable) LCC:

- Existing observed average high groundwater level is at Elevation 93 feet.
- Unit weight of brackish groundwater is 63 pounds per cubic foot (pcf).
- Target cast unit weight of the open-cell (permeable) LCC is 26+/- 2 pounds per cubic foot (pcf) with a minimum compressive strength of 50 pounds per square inch (psi) at 28 days.
- Target cast unit weight of the upper 2 feet of LCC is 30+/- 2 pcf with a minimum compressive strength of 80 psi at 28 days.
- Long-term (potentially fully saturated) unit weight of permeable LCC below groundwater is 68 pcf, resulting in a new buoyant (effective) unit weight of 5 pcf (68 pcf minus 63 pcf). This number is based on vacuum-pressure laboratory saturation testing, which indicates a potentially fully saturated unit weight of 63 pcf with an additional 5 pcf to account for potential variability.
- Unit weight of the existing fill varies from 110 (very loose sand) to 140 pcf (concrete and brick debris), with an average of approximately 125 to 130 pcf. A unit weight of 125 pcf is used for load offset calculations. Improved fill (beneath the new LCC section) is estimated to have a unit weight of 131 pcf—an increase of 6 pcf above the existing conditions.
- Pavement section is comprised of 8 inches of Portland cement concrete (PCC) overlain by 4 inches of asphalt concrete (AC), both with a unit weight of approximately 150 pcf. The pavement is underlain by 4 inches of aggregate base with a unit weight of approximately 130 pcf.
- Structural soil placed in the planter strips has a unit weight of 110 pcf. The width and length of the planting strips is different for each street section. The width of the structural

soil is approximately 6.5 to 13 feet and have been accounted for in the calculations at each section.

Using these values, the overexcavation and elevation of the bottom of LCC has been calculated such that the effective stress on the top of the Bay Mud after placement of the improvements will be at least 10 percent less than the existing effective stresses at the top of Bay Mud. We judge that, in using this approach, there will be a net unloading of the Bay Mud across the site, and the potential for a new cycle of primary consolidation will be low. In addition, the net unloading should significantly slow or retard any ongoing secondary compression settlement of the Bay Mud under existing loading within the street sections.

There may be a need for temporary backfill in localized excavations in LCC. Provided that the extent and duration are limited, these excavations can temporarily be backfilled with soil without causing new settlement. For these cases, a volume of up to 24 square feet of soil per linear foot of right of way can be placed for a duration of no more than 3 months. If the extent is larger or the duration is longer than these recommended values above, the use of temporary backfill should be evaluated case by case.

Prevention of Hydrostatic Uplift

To prevent hydrostatic uplift, open-cell (permeable) LCC will be used. The open-cell LCC will allow water to flow through the material, preventing excessive hydrostatic pressure from building at the bottom of the LCC section. The critical condition for hydrostatic uplift occurs when the LCC is only partially saturated. The assumptions used as the design criteria for the hydrostatic uplift check include:

- Existing observed average high groundwater level is at Elevation 93 feet.
- Unit weight of brackish groundwater is 63 pounds per cubic foot (pcf).
- Future (year 2100) mid-range groundwater level of Elevation 97 feet and high-range groundwater level is 99.5 feet[2].
- Target cast unit weight of the permeable LCC is 26+/- 2 pcf.
- Target cast unit weight of the upper two feet of LCC is 30+/- 2 pcf.
- Partially saturated unit weight of permeable LCC below groundwater is 50 pcf, resulting in a net buoyant unit weight of -13 pcf (50 pcf minus 63 pcf). This value is only used to check for hydrostatic uplift calculations.

The check for hydrostatic uplift compares the total stress at the base of the LCC against the theoretical hydrostatic pressure based on the future high groundwater levels. Each section of LCC should be considered adequate to resist hydrostatic uplift provided the factor of safety

[2] Groundwater levels have been taken from potential sea level rise levels provided in FEMA Guidelines.

against hydrostatic uplift is at least 1.1 when checking the high-range groundwater level of Elevation 99.5 feet and at least 1.2 when checking the mid-range groundwater level of Elevation 97 feet.

The LCC should be sufficiently permeable to prevent the buildup of excessive hydrostatic uplift pressure during fluctuations in the groundwater table. The tides in the San Francisco Bay generally change 5 feet or less over a period of 6 hours or longer (approximately 0.007 cm/sec). The water level measured in piezometers within the site fluctuates less than 1 foot when the tides change. Considering the likely rate of tidal fluctuations and the groundwater level fluctuations observed within in the site, we conclude that the material should have a minimum permeability of 0.005 cm/sec. The minimum permeability should be sufficient to prevent excessive hydrostatic uplift pressure on the LCC as the tides change. To mitigate the likelihood of the permeable LCC from becoming clogged with migrating fines from the surrounding soil and reducing the permeability, filter fabric should be placed at all interfaces where LCC is in contact with soil.

During construction, dewatering should be maintained until a sufficient thickness of LCC has been placed to prevent hydrostatic uplift using a using the observed high groundwater level currently encountered within the site of Elevation 94 feet.

<u>Crushing Resistance</u>

LCC should be considered adequate for support of the improvements in the new ROW provided the LCC has adequate compressive strength to resist crushing under anticipated loading, including self-weight, the load from overlying improvements, and temporary loads, including the heaviest anticipated fire truck, which represents the critical case for LCC crushing.

The LCC should have sufficient strength to resist crushing with a factor of safety of at least 2. Based on our calculations, we conclude LCC with a minimum submerged strength of 40 psi has a factor of safety greater than 2 for crushing under a tiller ladder truck tire or outrigger loads from an American LaFrance truck with a 105-foot-long ladder). Studies indicate the compressive strength of LCC reduces when saturated in brackish water. Based on test results, LCC saturated in brackish groundwater had a 28 days compressive strength as low as 80 percent that of LCC cured in a nonsaturated environment. Therefore, a target minimum compressive strength of 50 psi should be specified to allow for a 20 percent reduction in strength and still maintain a factor of safety of at least 2 under crushing.

<u>Pavement Design</u>

As described in the geotechnical report for the project, the standard City and County of San Francisco pavement section is being used. This pavement section consists of 4 inches of AC over 8 inches of PCC with an unconfined compressive strength of 4,500 psi. Although it is not part of the standard pavement section, a 4-inch-thick layer of aggregate base is detailed beneath the PCC. This composite section is not consistent with either rigid or flexible pavement design methodologies. However, we evaluated the pavement section using the design methodology per AASHTO Guide for Design of Pavement Structures. The results of our analysis indicate that the concrete section over a substrate with the strength and modulus of intact LCC is capable of supporting more than 11 million equivalent single-axle loads (ESAL). This ESAL value suggests that for a typical 20-year pavement design life, the pavement could support either 395 heavy trucks per day, including the fire truck or other trucks with three axles with the maximum legal weight at rear and a combined weight of 54,000 pounds (examples include dump trucks, garbage trucks, fire trucks, or full concrete trucks) or 500,000 light trucks per day what have two axles with a combined weight of 8,500 pounds (examples include box vans, utility trucks, or a pickup truck with a trailer).

Provided this number of ESAL's meet or exceed the expected performance for San Francisco City streets, we conclude that the LCC provides an acceptable substrate for the San Francisco City street pavement section.

<u>LCC Excavatability</u>

LCC should be excavatable to allow for underground utility installation, repair, or other maintenance. It can be excavated using standard tools, equipment, and methods, provided it is not too strong. LCC can be excavated in vertical cuts, allowing for smaller and more precise excavations for utility repair, without the need for shoring. Therefore, using LCC is beneficial for future work in the streets.

LCC with a maximum compressive strength of 300 psi is likely the upper limit for which LCC can still be excavated. The specification for the LCC specifies a maximum 28-day compressive strength of 200 psi. Because strength can continue to increase beyond 28 days, it is appropriate to specify 200 psi so that the LCC strength does not ultimately exceed 300 psi over time and is still excavatable.

<u>Seismic Design and Performance</u>

The Phase 1 project will be granted a permit under the 2016 San Francisco Building Code (SFBC). Strong shaking is expected during a major seismic event. The LCC will be subjected to several types of earthquake-induced loading, including (1) vertically propagating shear waves, (2) surface waves (e.g., Rayleigh waves), and (3) potentially differential ground movements caused by variation in depth to bedrock, thickness of Old Bay Clay, and thickness of Young Bay Mud.

One potential sources of damage to the LCC would be the horizontal cyclic shear stresses induced from vertically propagating horizontal shear waves. We have analyzed this condition, and our calculations show that LCC with a target unconfined compressive strength of 50 psi at 28 days (degraded to 40 psi) has sufficient strength to resist the cyclic shear stresses from these types of waves.

Considering that the LCC section is long (several hundred feet long) compared to its thickness (6 to 13 feet thick), it will be subjected to compression, tension, and shear and may locally crack when is subjected to surface waves or differential ground deformation, creating blocks of LCC. Because of the relatively rigid nature of the LCC, however, the LCC within each block will retain its original strength and stiffness and still provide support of improvements. The placement of LCC will be performed in lifts and segments; accordingly, cold joints will be created which should provide preferential cracking, and thereby limiting the extent of cracking.

If differential movement occurs at LCC cracks, the overlying pavement or sidewalk may crack and need repair after a major seismic event. The level of cracking expected in the pavement or sidewalks will likely be similar to or less severe than the cracking or distress to pavements or sidewalks at nearby sites where they bear on soil that has not been improved.

At locations where cracking occurs, mechanisms are in place that will reduce the likelihood of damage to the utilities. All underground utilities except district energy system (DES) piping are surrounded by bedding and cover sand or gravel. The bedding and cover materials are not compacted in place, and moderate differential movement along LCC cracks is expected to be accommodated in the bedding and cover material. The DES pipes consist of highly ductile high-density polyethylene (HDPE) piping that will be encased directly in the LCC. Considering the strength and ductility of the HDPE piping, we would not expect appreciable damage at locations where the LCC cracks. In general, we would expect better performance of the utilities within the LCC than at nearby soil sites; however, repairs may be necessary following a major seismic event.

We re-evaluated the adequacy of the LCC to support the pavement section in the case where the LCC has cracked because of a seismic event. As part of this evaluation we degraded the modulus of the LCC by 30 percent compared to intact LCC. This modulus degradation was selected based on the anticipated maximum shear strain of 0.07 percent in the LCC, which is based on our linear and nonlinear dynamic analyses under MCE loads using the program DeepSoil. We used the modulus degradation curves developed by Tiwani (2018) and selected a degraded modulus value close to the lower-bound curve at 0.07 percent shear strain. The resulting calculations show no reduction in the amount of ESALs using this degraded modulus; LCC with a reduced stiffness is still adequate to support the roadways.

We conclude that during a major earthquake, it is likely that the LCC will crack when subjected to the combined forces of surface waves and differential ground deformation. However, the likely consequences of LCC cracking from a major earthquake do not jeopardize the ability of the LCC to perform as intended to support the proposed roadway and underground utilities, and the cracking should be able to be addressed with post-earthquake maintenance. Accordingly, to perform as intended, it is not necessary that the LCC be free of cracking, but rather that the effects of cracking be taken into account in the design of the horizontal improvements at Mission Rock.

In conclusion, the anticipated seismic performance of the LCC is favorable, as summarized below:

- 28-day LCC compressive and shear strengths will be sufficient to resist cracking under earthquake cyclic shear stresses for the design-level earthquake.
- Minor or moderate cracking of the LCC is likely to occur and is allowable.
- Post-earthquake bearing capacity of the LCC is sufficient to support the streets, infrastructure, and other facilities.

- Post-earthquake pavement may require repairs similar those at other sites in San Francisco.

- Where the pavement is not damaged, the pavement performance is not jeopardized by LCC cracking.

- Utilities buried within the LCC should have acceptable performance as defined by the various owners of the utilities.

We appreciate the opportunity to assist you with this project, please call with any questions.

Sincerely,
Langan Engineering and Environmental Services, Inc.

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Project Engineer

Scott A. Walker, PE, GE
Senior Associate

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1.9 LCC Specifications
(Authored By Developer Team)

Geotechnical aspects of the specification were prepared by Langan Engineering and Environmental Services, Inc.

1. GENERAL
   1.1. DESCRIPTION
      1.1.1. Work Included: This work shall consist of batching, mixing, placing and testing LCC of the appropriate density as indicated by the specifications. A trained LCC installer shall furnish labor, material, equipment, and supervision for the installation of the LCC in accordance with the drawings and specifications.

   1.2. QUALITY ASSURANCE
      1.2.1. The LCC installer shall be approved in writing by Owner and demonstrate the following qualifications:
         - At least 5 years of experience installing LCC on projects of relatively comparable size and scope.
         - Key project staff including Project Manager, Superintendent and Forman with at least 3 years of relevant experience with LCC.
         - Key craft labor, including foam generator operator with experience, knowledge and skill needed to control mix and end product.
         - Necessary equipment including foam generator, pump and other equipment capable of consistently producing specified LCC in volume required for project.
         - Acceptable Owner, Engineer and Trade references.

   1.3. SUBMITTALS
      1.3.1. The prime contractor shall list the product and qualified installer of the LCC and shall not employ any product or producer without the prior approval of the geotechnical engineer of record (GEOR).
      1.3.2. Product data: within 30 calendar days after award of the contract, the prime contractor shall submit a mix design for approval by the GEOR and civil engineer of record (CEOR)
      1.3.2.1. Manufacturer's specifications, catalog cut sheet, and other engineering data needed to demonstrate to the issuing authority compliance with the specified requirements.
      1.3.3. Mix Design: Submit a mix design that will produce a cast density that complies with those listed in Section 2.2.1 of this specification at point of placement and a compressive strength within the range listed in Section 2.2.1. Include laboratory data using the mix design verifying un-foamed density, final foamed density, permeability (cm/sec) and compressive strengths. Mix design shall include water/cementitious ratio and foam solution dilution ratio, in accordance with manufacturer's recommendations. The mix design should also include Field Permeability Check Testing, by testing the percolation rate in modified 6" x 12" cylinder molds, filled half-way. The mix design should also include field saturation testing by the special inspector.

1.3.4. Work Plan: Submit a work plan before placement of LCC material. The plan shall include:

- Proposed construction sequence and schedule
- Type of equipment and tools to be used
- Material list of items and manufacturer's specifications
- LCC lift thickness
- LCC cure time and minimum strength prior to placing the next lift
- QA/QC and testing items and protocols frequency.

2. PRODUCTS

2.1. MATERIALS

2.1.1. Foaming Agent: A foaming agent shall be used and shall comply with the standard specifications of ASTM C 869 when tested in accordance with ASTM C 796. Admixtures shall be tested by the foam concentrate manufacturer for compatibility with the foaming agent.

2.1.2. Cement: the Portland cement shall comply with ASTM C 150. Other supplemental cementitious material such as fly ash may be used when approved by the project engineer. Supplementary cementitious materials shall be tested prior to the start of the project for compatibility with the foaming agent.

2.1.3. Admixtures: admixtures for accelerating, water reducing, and other specific properties may be used when specifically approved by the GEOR. Admixtures shall be tested in mix design prior to the start of the project for compatibility with the foaming agent.

2.1.4. Water: use water that is potable and free from deleterious amounts of alkali, acid, and organic materials, which would adversely affect the setting or strength of the LCC.

2.1.5. Filter Fabric: Shall have permeability equal to or greater than that of the LCC. Filter fabric shall also have a maximum apparent opening size (AOS, ASTM D4751) of 0.212 mm (U.S. sieve size 70).

2.2. PROPERTIES

2.2.1. Two types of LCC are to be supplied for the project: (1) general LCC to be applied across the site at multiple depths and (2) high density LCC to be cast only in the upper two feet of the LCC section. LCC shall meet the following properties:

General LCC			
	Target	Maximum	Minimum
General Cast Density, pcf (ASTM C 796)	26	28	24
Compressive Strength at 28 Days, psi (ASTM C 495)	NA	200	50
Coefficient of Permeability, cm/sec (ASTM D 2434 – modified)	0.1 (1E-1)	NA	0.005 (5E-3)
Saturated Density, pcf	55	68	50

High Density LCC – to be cast only within upper two feet of overall LCC section			
	Target	Maximum	Minimum
Cast Density of LCC, pcf (ASTM C 796)	30	32	28
Compressive Strength at 28 Days, psi (ASTM C 495)	NA	200	80
Coefficient of Permeability, cm/sec (ASTM D 2434 – modified)	0.1 (1E-1)	NA	NA
Saturated Density, pcf	55	68	50

3. EXECUTION
   - 3.1. Subgrade: Subgrade to receive LCC material shall be free of all loose and extraneous material. Light compaction equipment may be employed to tamp lose material. Subgrade shall be uniformly moist, and any excess water standing on the surface shall be removed. The subgrade shall be approved by the GEOR before placing filter fabric and LCC material.
   - 3.2. Curing: A minimum 12-hour curing period between lifts is required. Backfill or other usual loadings, including additional lifts of LCC, on the LCC shall not be permitted until the LCC has attained a compressive strength of at least 5 psi.
   - 3.3. Weather Conditions: If ambient temperatures are anticipated to be below 40 degrees F within 24 hours after placement, the mixing water shall be heated when approved by the manufacturer of the foaming agent or placement shall be prohibited. Placement shall not be allowed on frozen ground.
   - 3.4. Batching and Mixing: Cellular concrete shall be job site batched, mixed with the foaming agent and placed with specialized equipment certified by the manufacturer of the cellular concrete lightweight material. Cement and water may be premixed and delivered to the job site and the foaming agent added on site. Dilution ratio shall be adjusted as needed per manufacture's recommendation to achieve required end product.
   - 3.5. Placement:
     - 3.5.1. Place LCC in lifts not to exceed 36 inches in thickness, unless otherwise recommended by the LCC manufacturer and approved by the GEOR.
     - 3.5.2. After curing for minimum of 12 hours, any crumbling area on the surface shall be removed before the next layer is placed. Surface stepping to achieve grade and super elevation shall not be less than 6 inches in thickness. Grades of up to 5 percent may be made by adding a thickening agent to the mix in conformance with the manufacturer's recommendation.
     - 3.5.3. Subgrade and LCC should be protected from water inundation until the LCC is sufficiently cured and has sufficient overlying weight so it does not become buoyant.
     - 3.5.4. Freshly placed LCC should be protected from rain until it has been sufficiently cured to prevent damage.

3.5.5. Freshly placed LCC should be cured at least 3 hours before exposed to vibrations higher than a peak particle velocity 0.05 inches per second – such as those that may be generated during ground improvement activities.

3.6. Handling: Avoid excess handling of LCC according to industry standards.

3.7. Filter Fabric: Use filter fabric between LCC and adjacent soil and between LCC and shoring, where shoring will be removed after LCC placement.

4. QUALITY CONTROL TESTING BY CONTRACTOR AND OWNER

4.1. DENSITY CONTROL

4.1.1. During placement of the initial batches, check the un-foamed and foamed densities for each 100 cubic yards of LCC or as recommended per the GEOR and adjust the mix as required to obtain the proper water to cement ratio per the approved mix design and specified cast density at the point of placement per ASTM.

4.1.2. Field saturated density test procedures developed and prepared by the special inspector shall be performed on one sample for each 100 cubic yards of LCC or as recommended per the GEOR. GEOR to review and approve test procedures prior to commencement of work.

4.2. COMPRESSIVE STRENGTH: The compressive strength shall be tested under ASTM C 495 except as follows:

4.2.1. Four (4) specimens (one 7-day and three 28-days) shall be taken for each 100 cubic yards of LCC or as recommended per the GEOR. Unless otherwise approved, the specimens shall be 3 x 6 inch cylinders. During molding, place the LCC in 2 equal layers and raise and drop the cylinders 1 inch, 3 times on a hard surface or lightly tap the side or bottom of the cylinder to close any accidental entrained air. No rodding is allowed.

4.2.2. Specimens must be covered and protected immediately after casting to prevent damage and loss of moisture. Specimens shall be moist cured in the molds for 6 and 25 days and air dry a minimum of 24 hours and minimum of 72 hours before the 7-day and 28- day compressive strength testing, respectively. Specimens shall not be oven dried.

4.2.3. Contractor should maintain process control "run" charts of un-foamed and foamed density, field percolation result, and compressive strength data, updated daily for review by Owner's representative, and distributed weekly to applicable project team members.

4.3. PERMEABILITY:

4.3.1. Proof of permeability (per ASTM D 2434 – Modified) of the proposed LCC mix design shall be provided in the mix design submittal. If there is any change to the mix design during production, additional permeability testing will be required. Two samples per week should be cast per ASTM D 2434 and shipped to a certified and approved laboratory for supplemental testing.

4.3.2. Field falling head permeability per procedures prepared by the special inspector performed on every 500 cubic yards placed, with a minimum of two samples per day. Falling Head permeability test procedures to be reviewed and approved by GEOR prior to commencement of work.

4.4. MOCK UP TEST SECTION: One mock up test section shall be installed prior to construction to prove out the contractor's construction methods.

4.5. Side-by-side sampling and testing by QC and QA staff should occur once daily during the LCC placement on the Mock Up Test Section to identify any issues. At least one set of permeability samples should also be taken for saturation and drain down density and a permeability verification on the Mock Up Test Section.

4.6. UNFOAMED SLURRY TESTING: Test unfoamed slurry density periodically during foaming to verify actual density (PCF) is +/- 1.5% of target. Target to be established in mix submittal.

4.7. QUALITY ASSURANCE INSPECTIONS & ACCEPTANCE TESTING BY OWNER'S AGENCY

4.7.1. Owner shall employ a Special Inspector with an ACI Concrete Field Testing Technician Grade 1 Certification with at least 6 months experience inspecting LCC to observe LCC placement and test LCC as described below.

4.7.2. Laboratory tests for compressive strength, saturated density, and permeability shall be performed by testing lab accredited by AASHTO/CCRL, ACI, NIST or other comparable accreditation organization. Laboratory shall managed by a California Professional Engineer.

4.7.2.1. Lab and lab technicians shall demonstrate ability to perform special non-ASTM standard tests for Saturated Density and Falling Head Permeability by consistently replicating results for control samples tested and provided by an independent testing lab with proven capability to perform these tests.

4.7.3. Daily Inspections should include review of previous day's density testing of un-foamed and foamed test data, field permeability test results, any 7-day & 28-day compressive strength data, and location of samples taken. Initially use mix design for 7-day to 28-day strength correlation, switching to project data when three sets are available to predict 28-day strengths.

4.7.4. Perform one side-by-side comparison test with Contractor every 1000 cubic yards, and verify saturation density, drain-down density, compressive strength, and field permeability every 1,000 cubic yards placed, or whenever the field percolation rates are more than 20% lower than the mix design values.

4.7.5. Perform one laboratory permeability (ASTM D2434) every 5,000 cubic yards, with samples obtained from the 1,000 cubic yard side-by-side comparison from Section 4.7.3.

4.7.6. City personnel may provide additional special inspection at the City's discretion.

5. ACCEPTANCE CRITERIA

5.1.1. Installed LCC shall be considered acceptable provided 95 percent of all test results detailed in Section 4 of this submittal meet or exceed the minimum specified values specified in Section 2 of this specification.

5.1.1.1. Contractor may have the option to reassess batches that have failed the approval criteria if suitable and repeatable passing test values are generated and approved by the GEOR.

5.1.2. LCC work found out of tolerance to be removed and replaced. All material and labor required to perform remedial work or replace rejected LCC shall be provided at no cost to the Owner.

1.10 Field Test Procedure Submittals
(Authored By Developer Team)

Engineering & Construction Management

Montez Group Inc.

Field Saturated Density of PLCC Test Procedure

Prepared: February 28, 2020

Rev 01 5 May, 2020 revised soak time

SUBMITTAL No.:
Field Saturated Density of PLCC Test Procedure

This submittal has been reviewed for the Geotechnical aspects of the design only. Contractor is responsible for all corrections indicated hereon, for dimensions quantities, fabrications, construction techniques, and coordination with other contractors, subcontractors and suppliers. This review does not authorize changes to the contract requirements unless stated in a separate letter or change order.

☒ NO EXCEPTIONS TAKEN ☐ AMEND & RESUBMIT
☐ EXCEPTIONS NOTED ☐ REJECTED-SEE COMMENTS

Checked By: **P. Brady** **Date: 28 February 2020**

LANGAN
135 Main Street
Suite 1500, S.F. CA 94105

Montez Group Inc.
249 Onondaga Avenue, San Francisco, CA 94112

Table of Contents

1.0 Equipment List

1. 4x8” Cylinder Mold
2. Bucket/Wheel Barrel for Taking Samples for 6x12” Molds
3. Unit Weight Air Pot (Used in ASTM C138)
4. 5-gallon Bucket of Water
5. Scale
6. Caliper
7. File or Scraper
8. Cylinder Stripping Tool or Box Cutter

2.0 Significance and Use

The Field Estimation of Saturated Density Test Procedure provides the Saturated Density of Permeable Lightweight Cellular Concrete (see appendix 2 for example of calculation).

3.0 Sampling Procedure

Sampling procedure for PLCC is like taking samples of compressive strengths of LCC (ASTM C39 except mold sizes used are 4x8”.

1. Take and label 4x8” mold
2. Gather material in bucket or other container to transport material from placement location to sampling location
3. Use measuring cup, trowel or container to transfer material into 4x8” mold
4. Fill mold in 2 to 3 lifts up to top. Each lift should be consolidated by tapping the side of mold to release bubbles.
5. Place lid on sample
6. After samples are taken, handle carefully to location to allow to cure undisturbed for at least 24 hours

4.0 Testing Procedure

1. Sample will be cured for 3 days prior to testing
2. Carefully strip the PLCC sample from the cylinder mold using a cylinder stripping tool or box cutter without disturbing sample.
3. Use a file or scraper to remove about ¼” of material from thetop and bottom ends of the cylinder to roughen the surface and expose the cellular structure while ensuring sample’s corners are still squared. If larger amounts of material must be removed, a hand saw can be used, but be sure to square the ends as best

as possible with the file.

4. Measure the height of the PLCC cylinder. Measure to the nearest 1/8". Take the average of 3 to 4 heights around the circumference of the cylinder. Record this value (A).
5. Fully submerge the PLCC cylinder in a full 5-gallon bucket of water, upright and weighting the cylinder down to prevent floatation. Keep the cylinder fully submerged for at least 12 hours. Multiple cylinders can be submerged simultaneously, provided they remain identified.
6. Weight a standard concrete air pot assembly, pot and cap, and record the tare weight (B).
7. Fill the air pot completely with water, with the cap on, fill and remove excess air through the petcocks as though for a concrete air test, close the petcocks when full.
8. Dry the air pot assembly off with a rag or cloth, weight the water filled assembly and record this value (C).
9. Remove the cap from the air pot and place it beside the bucket containing the submerged PLCC cylinder. The air pot should be full of water.
10. Quickly transfer the submerged PLCC cylinder from the water bucket to the air pot, submerging the cylinder completely.
11. Holding the PLCC cylinder under water with one hand, place the air pot cap on with the other and clamp it down.
12. Fill the air pot assembly completely with water through the petcocks, closing the petcocks when full.
13. Again dry the entire assembly off with a rag or cloth, weigh and record this value (D).
14. Calculate the Saturated Density
    a. See Appendix Sample – Test Results & Table of Calculations

5.0 Appendix

1. Sample – Test Results

FIELD ESTIMATION OF SATURATED DENSITY OF PLCC
Test Method Provided by
CASTLE ROCK CONSULTING
TEST DATA SHEET

Project Name: Misson Rock -Lightweight Cellular Concrete Mock-up CEL # 10-37339PW

Sample Date: 12/17/2019 Sampled By: David Chin Lab # N/A

Sample Location/Source: Set 1

Material Description/Condition : Lightweight Cellular Concrete

Test Data

	Measure 1	Measure 2	Measure 3	Measure 4
Cylinder Heights, in	7.82	7.87	7.83	7.83

A. Average Cylinder Height (in) 7.84

B. Air pot assembly tare weight (pot + Cap), lb 17.70

C. Air pot assembly tare weight filled with water, lb 33.50

D. Air pot assembly with water + cylinder, lb 33.15

E. Cylinder Volume, (12.57 x A)/1728, cf 0.0570

F. Displacement water weight, 62.4 x E, lb 3.56

G. Full pot water weight, C-B, lb 15.80

H. Balance Water weight, G-F, lb 12.24

I. Approximate Saturated Unit Weight, (D-H-B)/E **56.26** **pcf**

Tested By: Y.Han
Date Tested: 12/31/2019

2. Sample – Table of Calculations

Field Saturated Density | Updated | Input Data

Item #	Location	Description	Cast Date	Date Tested	A1 Cylinder Height (in)	A2 Cylinder Height (in)2	A3 Cylinder Height (in)3	Cylinder Height (in)4	A Avg Cylinder Height Average (in)	B Airpot assembly tare weight (pot +	C Air pot assembly tare weight filled with water	D Air pot assembly with water + cylinder	E Cylinder Volume	F Displacement water weight	G Full pot water weight	H Balance water weight	Approximate Saturated Unit Weight	Comments
11	Mission Rock Pilot Lift #3	Set1	12/17/2019	12/31/2019	7.82	7.87	7.83	7.83	7.84	17.70	33.5	33.15	0.0570	3.56	15.80	12.24	56.26	

SAMPLE

MONTEZ GROUP
Engineering & Construction Management

Montez Group Inc.

Falling Head Field Permeability Test Procedure

Prepared: February 28, 2020

SUBMITTAL No.:
Falling Head Field Permeability Test Procedure

This submittal has been reviewed for the Geotechnical aspects of the design only. Contractor is responsible for all corrections indicated hereon, for dimensions quantities, fabrications, construction techniques, and coordination with other contractors, subcontractors and suppliers. This review does not authorize changes to the contract requirements unless stated in a separate letter or change order.

☒ NO EXCEPTIONS TAKEN ☐ AMEND & RESUBMIT
☐ EXCEPTIONS NOTED ☐ REJECTED-SEE COMMENTS

Checked By: P. Brady **Date:** 28 February 2020

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Rev0

Table of Contents

1.0 Equipment List

1. Modified 6x12” Cylinder Mold
    a. 6x12 Molds w/ Lids (Molds used for ASTM C31)
    b. Scribing Tool
    c. Tape
    d. 100 grit sandpaper
2. Bucket/Wheel Barrel for Taking Samples for 6x12” Molds
3. 5 Gallon Bucket
4. Heavy Wire Screen or 12” Brass Sieve
5. Steel Ruler
6. Stopwatch
7. Water

2.0 Significance and Use

The Falling Head Field Permeability Test Procedure provides another method of calculating the Permeability Constant (K) while being able to perform in the field.

$$K = \frac{L}{T} \ln \frac{h_1}{h_2}$$

Where:

K = Coefficient of Permeability in cm/sec

L = Sample Length in cm

h_1 = Initial elevation of the water surface

h_2 = Final elevation of the water surface

T = Average time in seconds from h_1 to h_2.

3.0 Preparing 6x12” Modified Cylinder Molds

1. Take a 6x12” cylinder mold and place open end upside down
2. Cut off the bottom of mold
3. Measure 6” from cut end of mold and mark a line on the inside with scribing tool
4. Use the 100 grit sandpaper and roughen the inside of the mold from 6” measurement to the cut end

5. Sand inside face of lid of cut end
6. Place lid on bottom (cut end) of mold
7. Tape lid on cylinder mold

4.0 Sampling Procedure

1. Take and label prepared modified 6x12” mold
2. Gather material in bucket or other container to transport material from placement location to sampling location
3. Use measuring cup, trowel or container to transfer material into modified 6x12” mold
4. Fill mold in 2 to 3 lifts up to pour line (approximately 6” mark). Each lift should be consolidated by tapping the side of mold to release bubbles.
5. After samples are taken, handle carefully to location to allow to cure undisturbed for at least 24 hours
6. Cover open tops of molds with another 6x12” lid or other suitable material to prevent moisture loss while curing

5.0 Testing Procedure

1. Sample will be cured for 3 days prior to testing
2. Place mold open side upside down and carefully remove tape and lid from bottom of mold. Ensure sides of mold will not break contact with samples.
3. Use scraper to scarify surface of bottom of sample and expose cellular structure
4. Turn mold upright and use scraper to scarify top surface and expose cellular structure and remove as little material as possible
5. With the cylinder mold with the open end up, press a ruler into the surface of the material to a depth of 1 inch, at the edge of the surface with the ruler oriented vertically. This is the depth scale for the falling head test. With one inch inserted, the next increment should be the 2” mark, corresponding to 1” of water above the surface, 3” will correspond to 2” of water, and so on
6. Fill a 5-gallon bucket completely with clean water
7. Place a heavy wire screen or 12” bass sieve on top of another, empty 5-gallon bucket. When the sample is removed from the water bucket, it will be transferred to the screen to allow it to drain freely
8. Submerge the mold, bottom surface first into the bucket of water, holding the top edges of the cylinder and pushing the sample down vertically, allowing water to infiltrate from the bottom and move upward through the cellular material
9. Keep mold submerged until water has infiltrated and covered the top surface of the material

10. Fully submerge the entire mold in the bucket, allowing the entire top half of the mold to fill with water
11. Holding the top edges of the mold, lift the entire mold vertically from the water and quickly transfer it to the screen over the empty bucket
12. The first run was to wash the water through to prime the sample. One the sample is prime, it is not necessary to re-prime the sample in between tests.
13. Get a stopwatch ready to record time
14. Repeat steps 8 to 11
15. With the stopwatch ready, start timing when the water level reaches the 5” mark (4” above the material surface).
16. Continue timing until the water level reaches the 2” mark (1” above the surface), stop timing.
17. Record the time (T in seconds) where Trial 1 is T_1, Trial 2 is T_2 etc…
18. Repeat steps 15 to 17 two more times, recording the time for the water level to drop from the 5” mark to the 2” mark, for a total of three trials.
19. Calculate all T per trial and average for T to input into coefficient of permeability, K
20. The approximate permeability coefficient can now be calculated from the average of the three recorded times by the falling head formula as shown in section 2.0:

$$\mathrm{K} = \frac{\mathrm{L}}{\mathrm{T}} \ln \frac{h_1}{h_2}$$

6.0 Appendix

1. Sample – Test Results

FALLING HEAD FIELD PERMEABILITY TEST
Test Method Provided by
CASTLE ROCK CONSULTING
TEST DATA SHEET

Project Name: Misson Rock -Lightweight Cellular Concrete Mock-up CEL # 10-37339PW

Sample Date: 12/23/2019 Sampled By: David Chin Lab # N/A

Sample Location/Source:

Material Description/Condition : Lightweight Cellular Concrete

Test Data

Tested By: Y.Han Date Tested: 12/31/2019

Trial #		Initial	1	2	
L, Length of Sample, cm		15.24	15.24	15.24	
h1, Initial elevation of the water surface, in		4	4	4	
h2, Initial elevation of the water surface, in		1	1	1	
Average time from h1 to h2	Min.	54	34	37	
	Sec.	46.15	32.25	44.36	AVG
Average Time in Seconds, sec		3286.15	2072.25	2264.36	2540.92
K, Coefficient of Permeability, cm/sec K=L/T*ln(h1/h2)					**0.008315**

SAMPLE

2. Sample – Table of Calculations

Falling Head Field Perm Updated Input Data

Item #	Location	Description	Cast Date	Date Tested	L (in Inches)	L (in cm)	h1 (Inches)	h2 (Inches)	Trial 1 T_A (in sec)	Trial 2 T_B (in sec)	Trial 3 T_C (in sec)	T_{avg} (in Sec)	K (in cm/sec)	Comments
10	Mission Rock Pilot Lift #4	Set1	12/31/2019	12/31/2019	6	15.24	4	1	3286.15	2072.25	2264.36	2540.92	8.31E-03	

K Coefficient of Permeability in (cm/Sec)
L Sample length in cm
h1 Initial elevation of water surface
h2 Final elevation of water surface
T Average time in seconds from h1 to h2

$K=(L/T)\ln(h1/h2)$

SAMPLE

1.11 Excavation and Backfill Procedure for LCC (Authored By Developer Team)

EXHIBIT F
Proposed Excavation and Backfill Procedures for Lightweight Cellular Concrete in Mission Rock Streets

(Exhibit by Mission Rock Partners)

PROPOSED EXCAVATION AND BACKFILL PROCEDURE FOR LIGHTWEIGHT CELLULAR CONCRETE

Revision 03 16 April 2020

1. **Purpose:** The purpose of this proposed procedure is describe utility excavation and backfill procedures in streets with Lightweight Cellular Concrete (LCC).

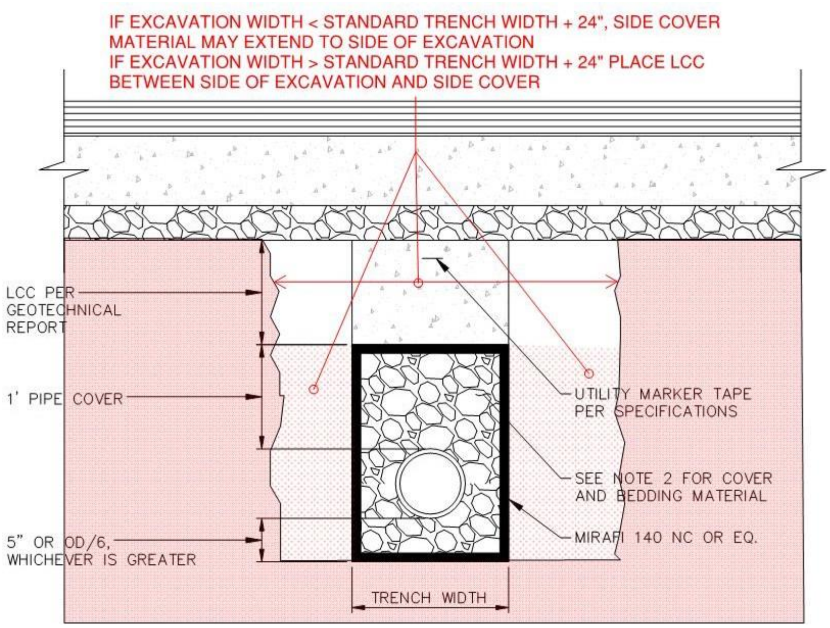
2. **Codes, Regulations:** Unless otherwise noted, DPW Order 187005 Section 10 Trench Backfill Requirements and all codes, regulations and standards referenced therein shall apply to excavation, trenching and backfill in LCC.

3. **Safety:** All trenching and excavation safety requirements required under Cal/OSHA CCR 1540 Article 6, Excavation shall be followed including, but not limited to
   - 3.1. Obtain DOSH Excavation Permit for all trenches deeper than 5'
   - 3.2. Trench shoring shall be installed and removed under the supervision of a Competent Person as defined by Cal/OSHA

4. **Control:** In order to ensure that excavation and trenching in Mission Rock streets, the following controls shall be implemented:
   - 4.1. Signs shall be posted prominently on street sign and/or street light poles with the following wording: "SUBGRADE IN MISSION ROCK STREETS IS LIGHTWEIGHT CELLULAR CONCRETE. EXCAVATION, TRENCHING AND BACKFILL ARE SUBJECT TO SPECIAL REQUIREMENTS. FOR MORE INFORMATION CONTACT SFPW AT (415) 554-5810 OR THE MISSION ROCK MASTER ASSOCIATION AT (415) NNN-NNN"
   - 4.2. All excavation and trenching in streets shall be performed under Excavation Permit. The Permit Section of SFPW shall be provided with a map showing the extend of LCC in Mission Rock Streets which shall be kept on file or recorded in the City Geographic Information System (GIS) and any other maps or other databases.
   - 4.3. When issuing Excavation Permits for street in in Mission Rock with LCC, SFPW shall require that this procedure be followed as a condition of the permit.

5. **Excavation:** LCC can be easily excavated using the same techniques and equipment as normal soil.
   - 5.1. Remove pavement per standard practice.
   - 5.2. Trenching can be done with standard back hoes, mini excavators and larger excavators with standard buckets as required for the particular trench width, depth and length. LCC can also be excavated by hand, or with the aid of small electric chipping hammers in tight places.
   - 5.3. LCC can also be excavated using a Vactor truck with a 2500-3000 psi water wand where it is necessary to excavate fill without damaging adjacent pipes.
   - 5.4. Standard Cal/OSHA shoring practices shall be followed. LCC in Mission Rock streets generally meets the criteria for Type A Soil having a compressive strength of > 1.5 tons/SF (typically the minimum compressive strength is >40 psi or 2.8 tons/SF).

6. **Backfill:**
   6.1. In general the bedding, shading and backfill should be restored to its original condition after pipe repair. Trench widths, bedding and shading material and dimensions for new laterals or mains should follow standards for original utilities in Mission Rock—these are generally the same as standards for other City utilities with the following exceptions:
      6.1.1. Filter fabric such as Mirafi 140NC or equal should be placed between bedding/shading and LCC to prevent fines from migrating into the LCC
      6.1.2. Low Pressure Water (LPW) with standard depth of 44” for 12’ mains shall be backfilled with clean, uniformly-graded sand up to the bottom of pavement basecourse.
   6.2. Place bedding and shading around the pipe per applicable standards. In general, side cover should be the same as the original installation. If the excavation is up to 1’ wider than the original width, sand or pea gravel shading may be placed up to 24” wider than the original trench for up to 20’ where the added width is necessary for installing repair sleeves, valves or other appurtenances. However if excavation is > 24’ wider than original standard trench or longer than 20’, then space between side of excavation and side cover or shading shall be filled with LCC. (see figure below) The reason for this is to maintain the weight of the lightweight fill within the 10% safety margin of the design.



   6.3. Backfill to top of subgrade (bottom of pavement basecourse) shall be LCC per the specification in Appendix A of this Procedure. LCC > 2-3’ below top of subgrade shall have cast density of 26 PCF (+/- 2 PCF). LCC < 2-3’ below top of subgrade shall have cast density of 30 PCF (+/- 2 PCF). <u>NOTE: As an alternate, in case that permeable LCC is not available. non-permeable LCC may be used in repairs above Elevation 95 feet or in localized trenches that with a volume less than 10 cubic yards.</u>

6.4. LCC shall be placed in 3' lifts. If multiple lifts are required, trench shall be covered with road plates or protected with barricades between lifts.
6.5. Quality Control of LCC backfill shall be as described in the LCC Specifications
6.6. Restore warning tape in backfill per applicable City standards.
6.7. A list of approved LCC contractors can be found in Appendix B.

7. **Emergency backfill with other material**: In an emergency unplanned utility repair where the street must be restored immediately, it is permissible to temporarily use normal standard soil backfill, Class II AB or similar materials which have a higher density than LCC, as long as the volume of temporary backfill does not exceed 24 cubic ft. per LF of street ROW and the temporary backfill is removed and replaced with LCC within three months or less, it is not expected to not cause differential settlement because a small amount of localized extra weight should not be enough to induce rapid settlement.

8. **Pavement Restoration:** Shall be per SFPW Standards. 4" of aggregate basecourse shall be placed on top of LCC below PCC pavement or concrete sidewalk.

Appendix A: LCC specification see TAP Report Volume 1, Section 1.9) (Note: Final procedure will have same spec attached. It is omitted here to avoid redundancy.

Appendix B: List of approved LCC Contractors

Cell-Crete Corporation
995 Zephyr Ave,
Hayward, CA 94544
(800) 696-0433
https://cell-crete.com/

Throop Lightweight Fill
701 Hazelwood Drive
Walnut Creek, CA 94596
415-419-6876
http://www.cellularconcrete.com

Confoam (A Conco Company)
5141 Commercial Circle
Concord, CA 94520
925-685-6799
https://www.conconow.com/commercial-concrete-contractors/confoam/

1.12 LCC Exacavatability
(Authored By TAP)

Section 1.12 - Excavatability

Introduction

The Developer proposes LCC to be used in the place of native soil materials within the entire public right-of-way. In a letter dated April 3, 2020, Public Works requests the Developer (MRP) to demonstrate that LCC with a compressive strength of 300 psi can be excavated using hand tools (as required under California law) and after 28-days, the compressive strength does not increase by more than 50% for the life of the project. The demonstration shall include:

- excavation solely with hand tools,
- to the full depth of utilities,
- in LCC representative of long-term strength

In addition, Public Works transmitted a comment / issues matrix dated April 3, 2020. Item #42 from that matrix requests the TAP to:

- Review and make recommendations regarding the hand-diggability of the LCC,
- The potential safety issues for those performing hand-digging, and
- The likelihood (given the relative ease or difficulty of the hand digging) that a crew would comply with State requirements to hand dig

Background on State requirements. California Government Code Sections 4215 - 4216, Protection of Underground Infrastructure, regulates the safe excavation of "subsurface installations" or underground pipelines, conduits, and ducts. Furthermore, City construction contracts as well as permits issued by both Public Works and Port (reference Article 2.4 of the Public Works Code) requiring cross-reference compliance with these state code sections for excavators performing construction in the public right-of-way.

Prior to excavation, utility operators must locate, and field mark their facilities with identifiable delineation, usually paint markings on the pavement surface. A "tolerance zone", based on these paint markings, is 24" each side of that paint marking. Excavations within this tolerance zone is limited to the use of hand tools (defined as using human power and is not powered by any motor, engine, hydraulic, or pneumatic device).

For work below Public Works and Port rights-of-way, sawcutting and powered equipment may only be used on the upper pavement section, usually about 12" thick and consisting of an Asphalt Concrete Wearing Surface (ACWS) layer over a concrete pavement base.

If an excavation is required within the tolerance zone of a subsurface installation and below the pavement section, the excavator shall determine the exact location of the subsurface installations in conflict with the excavation using hand tools before using any power-driven excavation or boring equipment within the tolerance zone of the subsurface installations. This code is intended for the safety and welfare of construction workers and protection of utility operator s' facilities.

LCC Excavatibility

Flowable fills are self-compacting low-strength materials, typically consisting of a combination of cement and/or fly ash, sand and/or rock. They are typically called Controlled Low Strength Materials, (CLSM) with various strength limits suggested, depending upon whether the material will require re-excavation or not and, more specifically, based on whether hand-excavation or normal backhoes will be utilized. ACI 229-13 suggests that CLSMs with compressive strengths of less than 100psi are readily excavatable by hand tools. NRMCA indicates that CLSMs with compressive strengths less than 150psi can readily be re-excavated by hand tools AND conventional machinery, such as backhoes.

The TAP recognizes the City's concern to comply with California's state law that materials over utilities must be excavatable with hand tools.

The ACI 229-13 report includes several items that are relevant to determining if the LCC materials within the project's ROWs should be deemed excavatable by hand tools, even at the maximum specified limits of design (200psi) and for "failure" criteria (300psi). First, Table 5.2.2 lists examples of CLSM mixture proportions. Secondly, Equation 5.3.7 for Removability Modulus, RE (under the Section 5.3.7 "Excavatability") shows a relationship that utilizes unit weight and compressive strength at 28days to predict excavatibility of various materials. A material with RE of less than 1.0 is removable with hand tools. Finally, ACI 229-13's Chapter 9 addresses Low Density CLSMs using PreFormed foams, called LD-CLSMs, which are the same materials that are referred to as LCC on the Mission Rock project. This chapter mentions that "Because of its low density, LD-CLSM is preferred when reduction of dead load is a critical requirement." The report also states that "In addition, LD-CLSM is easily excavated, which is a requirement in some applications", such as required by California law.

Using the mixture proportions of various CLSMs in Table 5.2.2, unit weights were calculated. The equation for RE in the ACI report is in metric units of kg/M3 and kPa. Performing a units conversion, into Imperial units of lb/CF and psi shows that this equation with 0.619 yields the same RE values as using 104 with psi & pcf, as previously reported. Thus, the various examples of CLSMs were analyzed in the attached table with their resulting RE values, as well as other CLSM mixtures.

Please note that in these examples, at or less that 100psi, the mixes with rock exceeded an RE of 1.0 for hand excavatability, and the mixes with sand only come reasonably close. Only the mixes with no aggregate fillers are below the 1.0 recommendation.

For those that were not in attendance for the September kick-off meeting, this analogy/explanation for Removability Modulus was given for various CLSM mixtures with 100psi at 28days. A traditional flowfill made at a concrete batch plant with sand and gravel would likely have a density of 145pcf, and would be difficult to dig with a shovel, due to the rock. A sand only flowfill would have a lower density without the rock, and also be easier to dig without the coarse aggregate. ACI 229-13 mentions that "Mixtures with high coarse aggregate quantities can be difficult to remove by hand, even at low strengths"; the RE equation values reflect this trend. A "slurry" CLSM would have a lower density without sand, and would be even easier to hand-excavate. Table 5.2.2 mixtures S-2 through S-43 show a lower density and RE values much less than 1.0; the mixtures are easier to dig with a shovel without the penetration resistance offered by the sand. When we use pre-formed foam to create large amounts of air, instead of sand and/or gravel the trend continues to make the CLSM easier to excavate.

Reference to various Removability Moduli in Colorado; explanation

To further demonstrate this trend with air, several commercial CLSM mixtures (approved in Colorado, subject to an RE of 1.5 or less) are also listed in the attached table. One producer uses custom "powder-only" volumetric on-site mixing trucks with pre-formed foam to produce cellular material that is significantly less than ACI's recommendation of less than 1.0 for excavation with hand tools. The three mixes with sand and gravels are significantly over the 1.0 recommendation. The high-strength flashfill mixture was developed for Denver Water, who wanted a fast-setting mix (100psi minimum in 4 hours) to resist water hammer, yet still subject to an RE of 1.5 or less. Before the normal flashfill was approved for use in Colorado Springs, the city required a "pot-hole" test of material that was in-place for over a year; the backhoe did not "stand off its pads" nor did the operator "feel" any resistance with his hydraulic controls. Chunks of material were readily broken up by hand, as samples of LCC brought to the kick-off meeting in September were.

Comparison of Removability Modulus at Mission Rock Pilot Project with other projects

The last group of mixes and RE calculations in the below table show the strength and unit weights of the 27pcf and 30pcf LCC tested in the Pilot project. The calculated RE values are significantly less than the 1.0 recommended limit, even at the maximum "design" strength limit of 200psi or 300psi "failure" limit. TAP understands that the Developer will be scheduling another excavation demonstration for city officials that were not able to attend the first one during Pilot Project testing; we would encourage those still concerned with excavatability to attend. Additional long-term coring (over 90 days) will also occur to evaluate actual strength gain, compared to cores obtained during the Pilot project.

Removability Modulus Values for Excavatability

CLSM Examples from ACI229-13 Table 5.2.2

Mix Identification	PCF	PSI	RE
CO DOT, includes rock	145	60	1.41
FL DOT, sand only	130	50	1.09
FL DOT, sand only	130	50	1.09
SC DOT, sand only	135	80	1.46
Mix AF, rock only	143	65	1.43

Mix D, Rock only	136	65	1.33
Non-Air CLSM, includes rock	145	100	1.82
Mix S-2, no aggregate	94	40	**0.60**
Mix S-3, no aggregate	86	60	**0.64**
Mix S-4, no aggregate	91	50	**0.64**

Hypothetical at 100psi & 150psi

Sand only CLSM	130	100	1.54
Sand only CLSM	130	150	1.89
Sand & Gravel CLSM	145	100	1.82
Sand & Gravel CLSM	145	150	2.22

CRC's Colorado Client CLSM Mixes

Client 1, Normal Flashfill,fly ash & foam	55	210	**0.61**
Client 1, Hi-Strength, fly ash & foam	72	490	1.41
Client 1, Cement & Foam,	41	270	**0.45**
Client2, CDOT mix sand & #9 rock	134	70	1.35
Client2, CDOT mix, sand & #9 rock	133	80	1.43
Client3, CDOT mix, sand & rock	138	72	1.43

Mission Rock LCC Data & Forecasts

Pilot project, 27pcf - average	27.6	111	**0.16**
Pilot project, 27pcf - high	28.5	130	**0.18**
Pilot project, 30pcf - average	30	147	**0.21**
Pilot project, 30pcf - high	30	160	**0.22**
Projected 26pcf at 200psi	26	200	**0.19**
Projected 26pcf at 300psi	26	300	**0.24**
Projected 30pcf at 200psi	30	200	**0.24**
Projected 30pcf at 300psi	30	300	**0.30**

Note1 : Colorado RE's less than 1.5 as originally specified to be "excavatable"

Note2 : all LCC RE values, including at Max of 200 & 300psi, much less than

ACI229's recommendation of RE< 1.0 for hand excavatibility.

Long-Term Strength Gain Estimates of LCC

While LCC has been around since the 1940s, we were not able to locate any long-term compressive strength gain data in ACI 536.1R-06 Guide for Cast-In-Place Low-Density Cellular Concrete, or with internet research. However, we were able to find some information of long-term strength gain in concrete dams, and assuming the cement hydration mechanism in LCC is similar to aggregate-based concrete, this is one way of estimating long term strength gain.

The first study was an ASCE 2010 article by the USBR, in recognition of the Hoover Dam turning 75 years old, entitled “Long-Term Properties of Hoover Dam Mass Concrete”. A coring program conducted in 1995 indicated an average core strength of 7230psi at 60 years of age. The average of Quality Assurance testing results at 28days was 3500; an increase of 207% in 60 years. Extrapolating the average 28day strength of the 30pcf LCC of the Pilot project would result in an estimate of **304psi at age 60 years; RE = 0.30**.

A second study with more intermediate data points was also a USBR Report from 2005 entitled Materials Properties Model of Aging Concrete (Report DSO-05-05.) Table 4 below, contained relevant strength gains out to 25 years. Please note Footnote 1 states that the 10year cores were tested dry, resulting in 10%-20% higher strengths. Assuming 10% higher when cores were tested dry, adjusted core strengths at 10years would be 6400psi, mid-way between 5year and 25 year data. With the 6400psi adjustment, the average of two data sets (0.5,1,5,10 & 25 years) and six data sets (0.5, 1 & 25years) were plotted in the graph below.

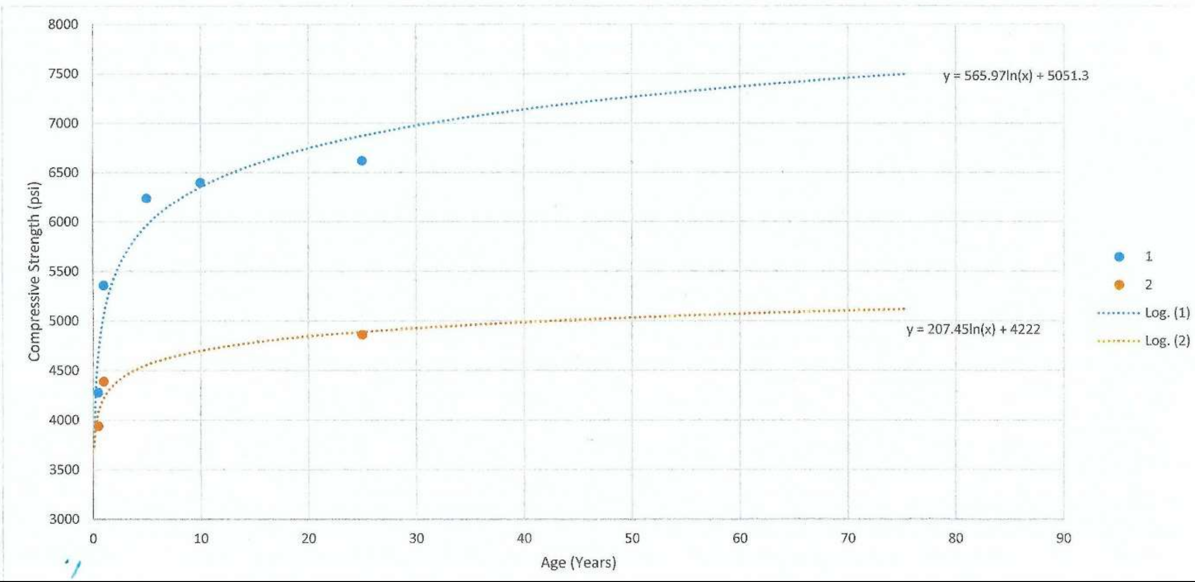
Table 4.—Compressive strength of 25-year cores compared to reference core tests by spatial orientation—Yellowtail Dam issue evaluation—Yellowtail Dam, Montana

			Compressive strength, lb/in²					
Mix	Drill hole	Elevation	6 mo	1 yr	5 yr	10 yr[1]	25 yr	Percent 1 yr
INT9/1963	18-13-V	3179.8	4460	6310	6660	7520	7510	119
INT/9B1963	18-13-V	3176.6	*No comparable data for this lift*				4810	
INT6/1963R	10-9-V	3204.5	4100	4400	5810	6550	5730	130
INT6B/1963R	10-9-V	3198.5	*No comparable data for this lift*				3880	
INT6B/1963R	10-9-V	3194.7	*No comparable data for this lift*				3260	
INT2/1964	5-9-V	3459.6	3300	3250			3390	104
INT2B/1964	5-9-V	3450.1	*No comparable data for this lift*				3450	
INT8/1964	24-10-V	3459.6		3400			3440	101
INT8B/1964	24-10-V	3453.6	*No comparable data for this lift*				4520	
INT8C/1964	24-10-V	3447.9	*No comparable data for this lift*				3290	
EXT3/1964	5-10-V	3459.5	4410	5090			4580	90
EXT3B/1964	5-10-V	3453.7	*No comparable data for this lift*				5730	
EXT3B/1964	5-10-V	3449.7	*No comparable data for this lift*				5750	
EXT5/1964	24-11-V	3459.5	3440	3900			4490	115
EXT5B/1964	24-11-V	3452.5	*No comparable data for this lift*				2450	
Average[2]			4280	5360	6240	7040	6620	
Average (all tests)							4420	
Average[3]			3940	4390			4860[3]	110[3]
Standard deviation (25 years—all tests)							1283	

[1] 10-year cores tested dry (may test about 10-20% higher than saturated test specimens).
[2] Average based on two comparable tests each at 6 mo, 1, 5, 10, and 25 yr.
[3] Average of comparable tests at 25 yr. 25-yr tests as a percent of 1-yr tests only where comparable data exists from the same lift as previous core programs (6 tests). Insufficient comparable data available for 5- and 10-yr tests.

Long Term Concrete Strength Gain, Bureau of Reclamation Study (Report DSO-05-05, December 2005)									
Averaged Data Set	No. Data Sets	Compressive Strength (psi) @ Year					Est. 28 Day Compressive Strength (psi)	Projected 75 Year Compressive Strength (psi)	75 yr/0.5yr
		0.5	1	5	10	25			
1	2	4280	5360	6240	6400	6620	3640	7490	1.75
2	6	3940	4390	-	-	4860	3710	5120	1.30

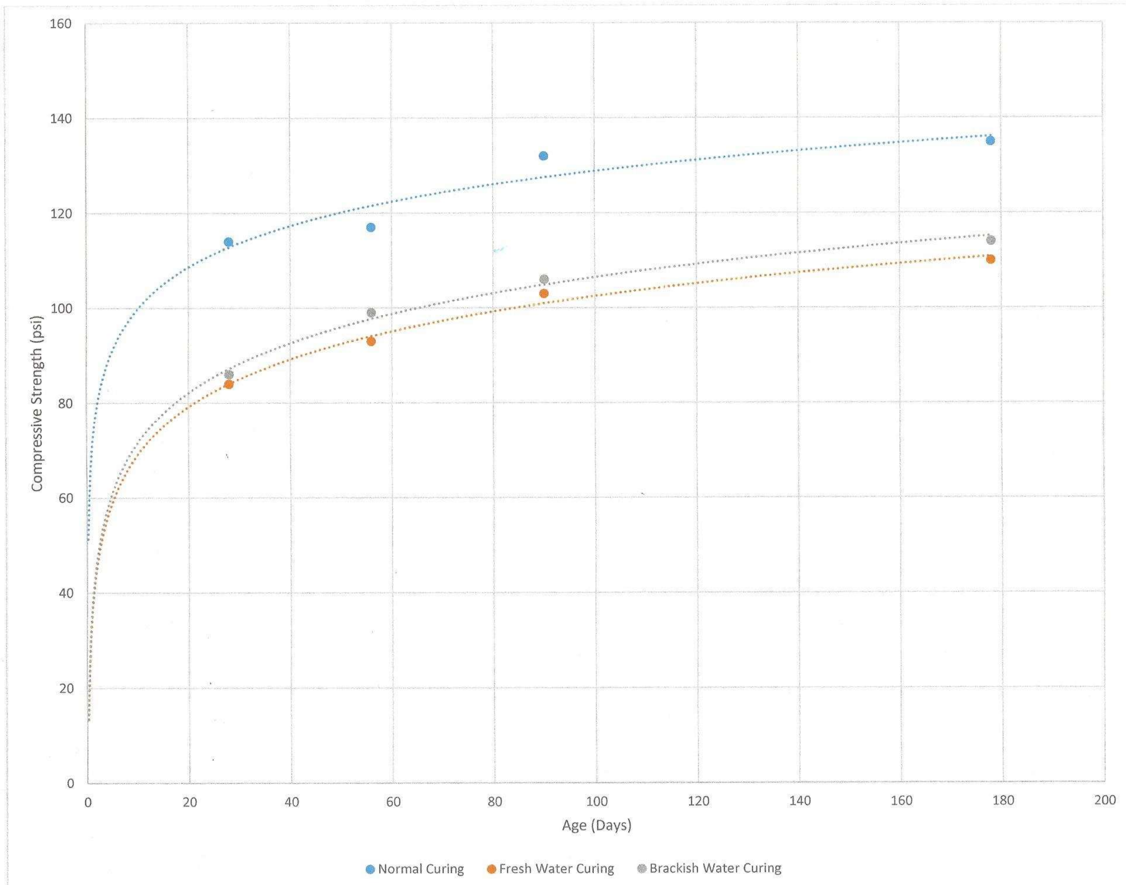
Note: Red value has been reduced by 10% from 7040 psi, per footnote in original publication regarding 10 year sample testing conditions.



Results of the long-term durability in fresh and salt water show a strength increase in normally cured 27pcf specimens of 18% from 28 to 178 days, and current saturation strength losses of 19% and 16% in fresh and salt water respectively. Laboratory values of this long-term durability are shown in the table below.

Long-Term Durability Testing of 27pcf LCC Submerged in Brackish Site Water

Description / Age	28 Days	56 Days	90 Days	178 Days
Normal Curing, psi	114	117	132	135
Fresh Water Curing, psi	84	93	103	110
Brackish Water Curing, psi	86	99	106	114
% Normal Curing / 28day psi	100%	103%	116%	**118%**
FW Submerged Strength Loss %	26%	21%	22%	19%
BW Submerged Strength Loss %	25%	15%	20%	16%



Long Term Durability Testing

Based on the 27pcf strength gain from 28days to 178 days (six months), the 30pcf top LCC mixture (147pcf @ 28) would have an estimated strength of 173psi at 6 months. Using the 10year data above, the 30pcf material might have a strength of 258psi at 10 years. Using the 75% and 30% gains from the table & graph above, **75 year strengths could reach 225psi to 303psi. These strengths would result in RE values of 0.26 to 0.30, both well under the ACI 229-13 criteria of 1.0 for CLSMs excavatable by hand.**

CASTLE ROCK CONSULTING
ENHANCING TRADITIONAL SOLUTIONS WITH INNOVATION

Constant Head Permeability Test of Granular Soils - ASTM-D2434

Test Results:

Client: Castle Rock Consulting
Project: Mission Rock PLDCC - Pervious & Non-Pervious Foam Solutions

Mix ID	Sample ID	Unit Weight (pcf)			K (cm/sec)	Foam
		Un-Saturated	Natural Sat.	Drained		
MR-27-55	MR-27-55-A	27.6	55.6	39.0	5.0E-01	Aquerix Pervious
	MR-27-55-B	27.5	57.2	39.8	4.6E-01	
	Average	**27.5**	**56.4**	**39.4**	**4.8E-01**	
MR-27-68	MR-27-68-A	27.8	53.4	38.9	4.8E-01	Aquerix Pervious
	MR-27-68-B	27.6	54.7	40.3	5.2E-01	
	Average	**27.7**	**54.1**	**39.6**	**5.0E-01**	
NP-27-68	NP-27-68-A	26.5	60.4	37.6	1.0E+00	Aerlite ix non-pervious
	NP-27-68-B	26.6	59.1	38.5	9.4E-01	
	Average	**26.6**	**59.8**	**38.1**	**9.9E-01**	

Testing Notes:

CHAPTER 1—INTRODUCTION

Controlled low-strength material (CLSM) is a self-consolidating cementitious material used primarily as a backfill as an alternative to compacted fill. Terms used to describe this material include flowable fill, controlled density fill, flowable mortar, plastic soil-cement, and soil-cement slurry.

CLSM is a mixture intended to result in a compressive strength of 1200 psi (8.3 MPa) or less. Most CLSM applications require unconfined compressive strengths of 300 psi (2.1 MPa) or less. Long-term strengths (90 to 180 days) should be targeted to be less than 100 psi (0.7 MPa) for excavation with hand tools. Lower-strength requirements are necessary to allow for future excavation of CLSM.

The term "CLSM" is used to describe a family of mixtures for various applications. CLSM mixtures can also be developed as anticorrosion fills, electrically conductive materials, low-permeability fills, thermal fills, and durable pavement bases. For example, the upper limit of 1200 psi (8.3 MPa) allows use of this material for applications where future excavation is unlikely, such as structural fill under buildings. CLSM is a self-consolidated backfill or fill material that is used in place of compacted earth fill and should not be considered as a type of low-strength concrete. Generally, CLSM mixtures are not designed to resist freezing and thawing, abrasive or erosive forces, or aggressive chemicals. Using recycled materials can maximize recycled material content for sustainable construction. Nonstandard materials that have been tested and found to satisfy the intended application can be used to produce CLSM. Chapter 9 describes low-density (LD) CLSM produced using preformed foam as part of the mixture proportioning. Using preformed foam in LD-CLSM mixtures allows these materials to be produced having unit weights lower than those of typical CLSM. The distinctive properties of LD-CLSM and procedures for mixing it are discussed in Chapter 9.

CLSM typically requires no consolidation or special curing procedures to achieve desired strength and should not be confused with compacted soil-cement, as reported in ACI 230.1R. Long-term compressive strengths for compacted soil-cement often exceed the 1200 psi (8.3 MPa) maximum limit established for CLSM.

Long-term compressive strengths of 50 to 300 psi (0.3 to 2.1 MPa) are low when compared with conventional concrete. In terms of allowable bearing pressure, however—which is a common criterion for measuring the capacity of a soil to support a load—50 to 100 psi (0.3 to 0.7 MPa) strength is equivalent to a well-compacted fill.

Although CLSM generally costs more per cubic yard (cubic meter) than most soil or granular backfill materials, its many advantages often result in lower in-place costs. In fact, for some applications, CLSM is the only reasonable backfill method available (Adaska 1994, 1997; Ramme 1997). Table 1 lists a number of advantages to using CLSM (Smith 1991).

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

E = modulus of elasticity, psi (MPa)
f_c' = 28-day specified compressive strength of concrete, psi (kPa)
k = coefficient of permeability, in./s (mm/s)
RE = removability modulus
W = dry mass density, lb/ft^3 (kg/m^3)

2.2—Definitions

ACI provides a comprehensive list of definitions through an online resource, "ACI Concrete Terminology," http://terminology.concrete.org.

CHAPTER 3—APPLICATIONS

3.1—General

The primary application of CLSM is as a structural fill or backfill in place of compacted soil. Because CLSM needs minimal consolidation and can be designed to be fluid, it is useful in areas where placing and compacting fill is difficult. If future excavation is anticipated, the maximum long-term compressive strength should generally not exceed 100 psi (0.7 MPa). The following applications present a range of uses for CLSM (Sullivan 1997).

3.2—Backfills

CLSM can be readily placed into a trench, hole, or other cavity (Fig. 3.2a and 3.2b). Compaction or consolidation equipment is not required; hence, trench width or excavation size can be reduced. Granular or site-excavated backfill, even if compacted or consolidated in the required layer thickness, cannot achieve the uniformity and density of CLSM (Sullivan 1997).

When backfilling against retaining walls, consideration should be given to lateral pressures exerted on the wall by flowable CLSM. Where lateral fluid pressure is a concern, are set on granular backfill. CLSM is then placed until it is 6 in. (150 mm) from the lower surface of the deck. At least 72 hours is required before the CLSM is brought up to the deck bottom through holes cored in the deck. Later, the railing is removed and the deck is widened. The same procedure is then completed on the opposite side of the bridge. The work is done under traffic conditions. The camber of the roadway over the culvert(s) is the only clue that a bridge had ever been present. Iowa DOT officials estimate that the cost of four reclamations is equivalent to one replacement when this technology can be employed (Larsen 1990; Buss 1989; Golbaum et al. 1997).

CHAPTER 4—MATERIALS

* Considered Different

4.1—General

* Conventional CLSM mixtures usually consist of water; portland cement; fly ash or other similar products; and fine aggregates, coarse aggregates, or both. Some mixtures consist of water, portland cement, and fly ash only. LD-CLSM * mixtures, as described in Chapter 9 of this report, consist of portland cement, fly ash, or other cementitious or pozzolanic materials, water, and preformed foam.

Although materials used in CLSM mixtures normally meet ASTM or other standard requirements, the use of standardized materials is not always necessary. Materials selection should be based on availability; cost; specific application; and necessary mixture characteristics, including flowability, strength, excavatability, and density.

4.2—Portland cement

Cement provides cohesion and strength for CLSM mixtures. For most applications, Type I or Type II portland cement conforming to ASTM C150/C150M is normally used. Other types of cement, including blended cements conforming to ASTM C595/C595M or performance cements conforming to ASTM C1157/C1157M, can be used if prior testing indicates acceptable results.

4.3—Fly ash

Coal-combustion fly ash is sometimes used to improve flowability. Its use can also increase strength and reduce bleeding, shrinkage, and permeability. High-fly-ash-content mixtures result in lower-density CLSM when compared with mixtures having high aggregate contents. Fly ashes used in CLSM mixtures do not need to conform to either Class F or C as described in ASTM C618. For example, fly ashes containing carbon contents higher than traditionally used in concrete may be acceptable. Trial mixtures should be prepared to determine whether the mixture will meet the specified requirements. Refer to ACI 232.2R for further information (Naik et al. 1991; Landwermeyer and Rice 1997).

4.4—Admixtures

Air-entraining admixtures and foaming agents can be valuable constituents for the manufacture of CLSM. The inclusion of air in CLSM can help provide improved workability, reduced shrinkage, little or no bleeding, minimal segregation, lower unit weights, and control of ultimate strength development. Higher air contents can also help enhance thermal insulation and resistance to freezing-and-thawing cycles. Water content can be reduced as much as 50 percent when using air-entraining admixtures. Using these materials may require modifications to typical CLSM mixtures. To prevent segregation when using high air contents, mixtures need to be proportioned with sufficient fines to promote cohesion. Most air-entrained CLSM mixtures are pumpable but can require higher pump pressures when piston pumps are used. To prevent extended setting times, extra cement or an accelerating admixture may be required. In all cases, pretesting should be performed to determine acceptability (Hoopes 1997; Nmai et al. 1997).

4.5—Mineral admixtures and other additives

In specialized applications such as waste stabilization, CLSM mixtures can be formulated to include chemical additives, mineral additives, or both, that serve purposes beyond backfilling. Some examples include using swelling clays such as bentonite to achieve CLSM with low permeability. The inclusion of zeolites, such as analcime or chabazite, can be used to absorb selected ions where water or sludge treatment is required. Magnetite or hematite fines can be added to CLSM to provide radiation shielding in applications at nuclear facilities (Rajendran and Venkata 1997; Langton and Rajendran 1995; Langton et al. 2001). Slag cement conforming to ASTM C989/C989M may be used as a substitute for, or in addition to, portland cement. As is the case with portland cement, higher slag cement contents can produce excessive strengths and should be tested before use. Silica fume may also be used in a CLSM formulation.

4.6—Water

Water that is acceptable for concrete mixtures is acceptable for CLSM mixtures. ASTM C94/C94M provides additional information on water-quality requirements.

4.7—Aggregates

Aggregates are often the major constituent of a CLSM mixture. The type, grading, and shape of aggregates can affect the physical properties, such as flowability and compressive strength. Aggregates complying with ASTM C33/C33M are generally used because concrete producers have these materials in stock.

Granular excavation materials with somewhat lower-quality properties than concrete aggregate are a potential source of CLSM, and should be considered. Variations of the physical properties of the mixture components, however, will have a significant effect on mixture performance. Silty sands with up to 20 percent fines passing through a No. 200 (75 μm) sieve have proven satisfactory. Soils with wide variations in grading have also shown to be effective. Soils with clay fines, however, have exhibited problems with incomplete mixing, mixture stickiness, excess water demand, shrinkage, and variable strength. These soil types are not usually considered for CLSM applications. Aggregates that have been used successfully include (Tansley and Bernard 1981):

a) ASTM C33/C33M specification aggregates within specified gradations

b) Pea gravel or pea stone with sand

c) 3/4 in. (19 mm) minus aggregate with sand

d) Native sandy soils, with more than 10 percent passing a No. 200 (75 μm) sieve

e) Quarry waste products, generally 3/8 in. (10 mm) minus aggregates

4.8—Nonstandard materials

Nonstandard materials, which can be more available and economical, can also be used in CLSM mixtures, depending on project requirements. These materials, however, should be tested before use to determine their acceptability in CLSM mixtures.

There are numerous examples of nonstandard materials that can be substituted as aggregates (Naik et al. 1996; Naik and Singh 1997a,b). Such materials include various coal combustion products, crusher fines, discarded foundry sands (Tikalsky et al. 1998, 2000; Deng and Tikalsky 2008; Siddique and Noumowe 2008), glass cullet (Wang 2009), and reclaimed crushed concrete (Achtemichuk et al. 2009). In addition, nonstandard aggregate derived from stable organic sources, such as scrap tire rubber, can be used in CLSM mixtures (Pierce and Blackwell 2003).

Aggregates or mixtures that can swell in service due to expansive reactions or other mechanisms should be avoided. Wood chips, wood ash, or other organic materials may not be suitable for CLSM. Fly ashes with carbon contents up to 22 percent have been successfully used for CLSM (Ramme et al. 1995). Cement kiln dust, also a nonstandard material, may be used as a substitute for other cementitious materials (Pierce et al. 2003; Lachemi et al. 2010).

In all cases, nonstandard material characteristics should be determined, and the suitability of the material should be tested in a CLSM mixture to determine whether it meets specified requirements. Environmental regulations could require prequalification of the raw material, CLSM mixture, or both, before use.

4.9—Ponded ash or basin ash

Ponded ash—typically a mixture of fly ash and bottom ash slurried into a storage or disposal basin—can be used in CLSM. Proportioning of the ponded ash in the resulting mixtures depends on its particle size distribution. Typically, it can be substituted for all of the fly ash and a portion of the fine aggregate and water. Unless dried before mixing, ponded ash requires special mixing because it is usually wet. Basin ash is similar to ponded ash except it is not slurried and can be disposed of in dry basins or stockpiles (Rajendran and Venkata 1997; Langton and Rajendran 1995).

CHAPTER 5—PROPERTIES

5.1—Introduction

The properties of CLSM cross boundaries between soils and concrete. CLSM is manufactured from materials similar to those used to produce concrete, and is placed similarly to concrete. In-service CLSM, especially lower-strength CLSM, exhibits characteristic properties of soils. Characteristics of CLSM are affected by mixture constituents and proportions of the ingredients in the mixture. Because many factors can affect the characteristics of CLSM, a wide range of values can exist for the various properties discussed in the following sections (Glogowski and Kelly 1988).

5.2—Plastic properties

5.2.1 *Flowability*—Flowability distinguishes CLSM from other fill materials. It enables the materials to be self-leveling, to flow into and readily fill a void, and be self-consolidating. This property represents a major advantage of CLSM compared with conventional fill materials that must be mechanically placed and compacted. Because fresh CLSM is similar to fresh concrete and grout, its flowability is best viewed in terms of concrete and grout technology.

A major consideration in using highly flowable CLSM is the hydrostatic pressure it exerts. Where fluid pressure is a concern, CLSM can be placed in lifts, with each lift being allowed to harden before placement of the next lift. Examples where multiple lifts can be used are limited-strength forms used to contain the material or where buoyant items, such as pipes, are encapsulated in the CLSM.

Flowability can be varied from stiff to fluid, depending on requirements. Methods of expressing flowability include using a 3 x 6 in. (75 x 150 mm) open-ended cylinder modified flow test (ASTM D6103), the standard concrete slump cone (ASTM C143/C143M), and flow cone (ASTM C939).

Good flowability, using the ASTM D6103 method, is achieved where there is no noticeable segregation and the CLSM spread is at least 8 in. (200 mm) in diameter.

Flowability ranges associated with the slump cone can be expressed as follows:

a) Low flowability: slump less than 6 in. (150 mm)

b) Normal flowability: slump 6 to 8 in. (150 to 200 mm)

c) High flowability: slump greater than 8 in. (200 mm)

ASTM C939, for determining grout flow, has been used successfully with fluid mixtures containing aggregates not greater than 1/4 in. (6 mm). Chapter 8 briefly describes this method.

5.2.2 *Segregation*—Separation of materials in the CLSM mixture can occur when flowability is primarily produced by adding water. This situation is similar to segregation experienced with some high-slump concrete mixtures. With proper mixture proportioning and materials, a high degree of flowability can be attained without segregation. For highly flowable CLSM without segregation, adequate fines are required to provide suitable aggregate suspension and stability. Fly ash and other mineral admixtures generally account for these fines (refer to Table 5.2.2), although silty or other noncohesive fines up to 20% of total aggregate have been used. Using plastic fines, such as clay, should be avoided because they can produce deleterious results, such as increased shrinkage. Some CLSM mixtures have been designed without sand or gravel, using only mineral admixtures as filler material.

5.2.3 *Subsidence*—Subsidence deals with the reduction in volume of CLSM as it releases water and entrapped air

Table 5.2.2—Examples of CLSM mixture proportions

Source	Cement content, lb/yd³ (kg/m³)	Fly ash content, lb/yd³ (kg/m³)	Coarse aggregate, lb/yd³ (kg/m³)	Fine aggregate, lb/yd³ (kg/m³)	Approximate water content, lb/yd³ (kg/m³)	28-day compressive strength, psi (MPa)
✱ CO DOT	50 (30)*	—	1700 (1010)	1845 (1096)	325 (193)	60 (0.4)
IA DOT	100 (60)	300 (178)	—	2600 (1543)	585 (347)	—
FL DOT	50 to 100 (30 to 60)	0 to 600 (0 to 356)†	—	2750 (1632)‡	500 (297) maximum	50 to 150 (0.3 to 1.0)
IL DOT	50 (30)	300 (178) Class F 200 (119) Class C	—	2900 (1720)	375 to 540 (222 to 320)	—
IN DOT MIXTURE*	60 (36)	330 (196)	—	2860 (1697)	510 (303)	—
IN DOT MIXTURE 2§	185 (110)	—	—	2675 (1587)	500 (297)	—
OK DOT	50 (30) minimum	250 (148)	—	2910 (1727)	500 (297) maximum	—
MI DOT MIXTURE 1	100 (60)	2000 (1187) Class F	—	—	665 (395)	—
MI DOT MIXTURE 2§	50 (30)	550 (326) Class F	Footnote ‖	Footnote ‖	330 (196)	—
OH DOT MIXTURE 1	100 (60)	250 (148)	—	2850 (1691)	500 (297)	—
OH DOT MIXTURE 2	50 (30)	250 (148)	—	2910 (1727)	500 (297)	—
SC DOT	50 (30)	600 (356)	—	2500 (1483)	460 to 540 (273 to 320)	80 (0.6)
DOE-SR#	50 (30)	600 (356) Class F	—	2515 (1492)	500 to 550 (297 to 326)	30 to 150 (0.2 to 1.0)
Unshrinkable fill**	60 (36)	—	1705 (1012) (3/4 in. [19 mm] maximum)	1977 (1173)	257 (152)††	17 (0.1) at 1 day
Pond ash /basin ash mixture‡‡						
Mixture AF‡‡	165 (98)	810 (481)§§	2190 (1300)	—	700 (415)	65 (0.4)
Mixture D‡‡	100 (60)	550 (326)‖‖	2515 (1492)	—	507 (301)	65 (0.4)
Coarse aggregate CLSM##						
Non-air entrained##,***	50 (30)	250 (148)	1900 (1127) (1 in. [25 mm] maximum)	1454 (863)	270 (160)†††	100 (0.7)
Air entrained##,‡‡‡	50 (30)	250 (148)	1900 (1127) (1 in. [25 mm] maximum)	1340 (795)	255 (151)†††	—
Flowable fly ash slurry						
Mixture S-2§§§	98 (58)	1366 (810) Class F	—	—	1068 (634)	40 (0.3) at 56 days
Mixture S-3‖‖‖	158 (94)	1264 (749) Class F	—	—	1052 (624)	60 (0.4) (75 [0.5] at 56 days)
Mixture S-4###	144 (85)	1155 (685) Class F	—	—	1146 (680)	50 (0.3) (70 [0.5] at 56 days)

Note: Table examples are based on experience and test results using local materials. Yields will vary from 27 ft³ (0.76 m³). This table is given as a guide and should not be used for design purposes without first testing with locally available materials.

*Cement quantity can be increased above these limits only when early strength is required and future removal is unlikely.

†Slag cement can be used in place of fly ash or used in combination with fly ash.

‡Adjust to yield 1 yd³ (0.76 m³) of CLSM.

§5 to 6 fl oz of air-entraining admixture produces 7 to 12 percent air contents.

‖Total granular material of 2850 lb/yd³ (1690 kg/m³) with 3/4 in. (19 mm) maximum aggregate size.

#Department of Energy (DOE) Savannah River Site CLSM mixture.

**Emery and Johnston (1986).

††Produces 6 in. (150 mm) slump.

‡‡DOE Savannah River Site CLSM mixture using pond/basin ash.

§§Basin ash mixture.

‖‖Pond ash mixture.

##Fox (1989).

***Produces approximately 1.5 percent air content.

†††Produces 6 to 8 in. (150 to 200 mm) slump.

‡‡‡Produces 5 percent air content.

§§§Produces modified flow of 8-1/4 in. (210 mm) diameter (Table 8.1a); air content of 0.8 percent; slurry density of 93.7 lb/ft³ (1500 kg/m³).

‖‖‖Produces modified flow of 10-1/2 in. (270 mm) diameter; air content of 1.1 percent; slurry density of 91.5 lb/ft³ (1470 kg/m³).

###Produces modified flow of 16-3/4 in. (430 mm) diameter; air content of 0.6 percent; slurry density of 90.6 lb/ft³ (1450 kg/m³).

✱ $\frac{50+1700+1845+325}{27} = 145.2$ pcf

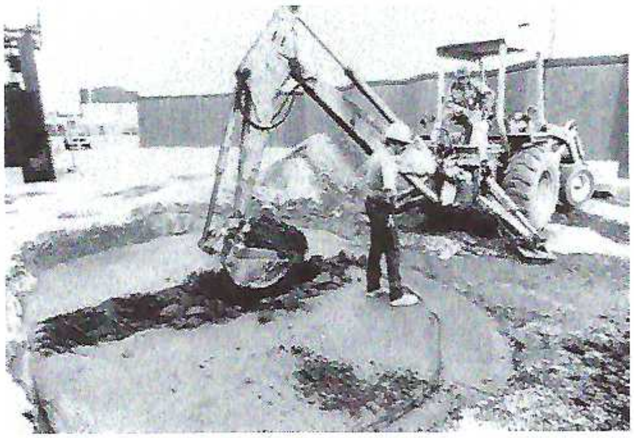


Fig 5.3.7—Excavating CLSM with a backhoe.

aggregate contents are increased (Smith 1991). However, materials normally used for reducing permeability, such as bentonite clay and diatomaceous soil, can affect other properties and should be tested before use.

5.3.6 *Shrinkage (cracking)*—It is believed that shrinkage and shrinkage cracks do not affect the performance of CLSM in the same manner as conventional concrete. Several reports indicate that minimal shrinkage occurs with CLSM. Ultimate linear shrinkage ranges from 0.02 to 0.05 percent (Naik et al. 1990; Tansley and Bernard 1981; McLaren and Balsamo 1986). Recent research indicates that CLSM with high volumes of fly ash (965 lb/yd³ [360 kg/m³]) exhibit higher amounts of linear shrinkage.

5.3.7 *Excavatability*—The ability to excavate CLSM is an important consideration on many projects. In general, CLSM with a compressive strength of 100 psi (0.7 MPa) or less can be excavated manually. A removability modulus (*RE*) can be used to determeе the excavatability of CLSM. The *RE* can be determined as follows

Use 104 with pcf + psi

$$RE = \frac{W^{1.5} \times 0.619 \times C^{0.5}}{10^6} \tag{5.3.7}$$

where W is the dry mass density (kg/m³), and C is the 28-day unconfined compressive strength (kPa). If the *RE* is less than 1.0, the CLSM is removable. CLSM with *RE* values greater than 1.0 are not easily removed.

Mechanical equipment, such as backhoes, can remove materials with compressive strengths of 100 to 300 psi (0.7 to 2.1 MPa) (Fig. 5.3.7). Excavatability limits are arbitrary guidelines and depend on the CLSM mixture constituents. Mixtures using high coarse aggregate quantities can be difficult to remove by hand, even at low strengths. Mixtures using fine sand or only mineral admixtures as aggregate filler have been excavated with a backhoe up to strengths of 300 psi (2.1 MPa) (Krell 1989).

When excavatability of CLSM is a concern, the type and quantity of cementitious materials is important. Acceptable long-term performance has been achieved with cement contents from 40 to 100 lb/yd³ (24 to 59 kg/m³) and Class F fly ash contents up to 350 lb/yd³ (208 kg/m³). Lime (CaO) contents of fly ash that exceed 10 percent by weight can be a concern where long-term strength increases are not desired (Tansley and Bernard 1981).

Because CLSM typically continues to gain strength beyond the conventional 28-day testing period, it is suggested, especially for CLSM with high cementitious content, that long-term strength tests be conducted to estimate the potential for excavatability. In addition to limiting the cementitious content, entrained air can be used to maintain low compressive strength.

* Tests On-Going...

5.3.8 *Shear modulus*—The shear modulus, which is the ratio of unit shearing stress to unit shearing strain, of normal-density CLSM typically ranges from 3400 to 7900 ksf (160 to 380 MPa) (Larsen 1988; Rajendran and Venkata 1997; Langton et al. 2001). Shear modulus is used to evaluate the expected shear strength and deformation of CLSM.

5.3.9 *Potential for corrosion*—The potential for corrosion on metals encased in CLSM has been quantified by a variety of methods specific to the material in contact with CLSM. Electrical resistivity tests can be performed on CLSM in the same manner that natural soils are compared for their corrosion potential on corrugated metal culvert pipes using California Test 643 (California Department of Transportation 1999). Moisture content of the sample is an important parameter for sample resistivity, and the samples should be tested at their expected long-term field moisture content. Unlike soil, high pH values characteristic of CLSM can be beneficial. The high pH of CLSM can provide a protective passive film for iron-based materials, thus reducing potential for corrosion.

The Ductile Iron Pipe Research Association (DIPRA) (Horn 2006; AWWA 2010) has a method for evaluating the corrosion potential of backfill materials. The evaluation procedure is based on information drawn from five tests and observations—soil resistivity, pH, oxidation-reduction (redox) potential, sulfides, and moisture. For a given sample, each parameter is evaluated and assigned points according to its contribution to corrosivity (Straud 1989; AWWA 2010; Hill and Sommers 1997). Although applicable for soils, this procedure in its entirety may not be applicable to CLSM. The DIPRA method indicates that high-pH soils are deleterious for corrosion protection. High pH associated with CLSM, as with concrete, is believed to be beneficial for corrosion protection. These procedures are guides for determining a soil's potential corrosivity to ductile iron pipe and should be used only by qualified engineers and technicians experienced in soil analysis and evaluation.

A continuous metallic material that passes through soils of varying composition may exhibit galvanic corrosion due to differences in its corrosion potential in different soils. The uniformity of CLSM reduces the probability of galvanic corrosion due to dissimilarities in the surrounding environment, which may otherwise occur from the use of dissimilar backfill materials and non-uniform compaction of similar materials. Because of the high pH associated with CLSM and the more neutral pH of conventional soils, however, care must be taken when partially encasing iron-based products

Table 8.1a—Test procedures for determining consistency and unit weight of CLSM mixtures

Consistency	Fluid mixtures	ASTM D6103—Procedure consists of placing 3 in. (75 mm) diameter by 6 in. (150 mm) long open-ended cylinder vertically on level surface and filling cylinder to top with CLSM. Cylinder is then lifted vertically to allow material to flow out onto level surface. Good flowability is achieved where there is no noticeable segregation and material spread is at least 8 in. (200 mm) in diameter.
		ASTM C939—Florida and Indiana DOT specifications require efflux time of 30 seconds ± 5 seconds. Procedure is not recommended for CLSM mixtures containing aggregates greater than 1/4 in. (6 mm).
	Plastic mixtures	ASTM C143/C143M
Unit weight		ASTM D6023
		ASTM D4380
		ASTM D1556
		ASTM D2922

Table 8.1b—Test procedures for determining in-place density and strength of CLSM mixtures

ASTM D6024	This specification covers determination of ability of CLSM to withstand loading by repeatedly dropping metal weight onto in-place material.
ASTM C403/C403M	This test measures degree of hardness of CLSM. California DOT requires penetration number of 650 before allowing pavement surface to be placed.
ASTM D4832	This test is used for molding cylinders and determining compressive strength of hardened CLSM.
ASTM D1196/D1196M	This test is used to determine modulus of subgrade reaction (K values).
ASTM D4429	This test is used to determine relative strength of CLSM in place.

8.3—Consistency and unit weight

Depending on application and placement requirements, flow characteristics can be important. CLSM consistency can vary considerably from plastic to fluid; therefore, several methods of measurement are available. Most CLSM mixtures perform well with various flow and unit weight properties. Table 8.1a describes methods that can be used to measure consistency and unit weight. CLSM generally exhibits large shrinkage. When CLSM is being placed in the ground, this may not be a problem. When used to backfill a pipe or other containment structure, shrinkage may be important to the designer. ASTM C157/C157M can be used for measuring CLSM shrinkage.

8.4—Strength tests

CLSM is used in a range of applications requiring different load-bearing characteristics. Maximum loads to be imposed on the CLSM should be identified to determine minimum strength requirements. In many cases, however, CLSM needs to be limited in its maximum strength. This is true where removal of the material at a later date is anticipated.

The strength of CLSM can be measured by several methods. Table 8.1b describes some of these methods. Unconfined compressive strength tests are the most common; however, other methods, such as penetrometer devices or plate load tests, can also be used. Compressive-strength specimens range in size from 2 x 2 in. (50 x 50 mm) cubes to 6 x 12 in. (150 x 300 mm) cylinders. Special care should be used when removing very-low-strength CLSM mixtures from test molds. Additional care in the handling, transporting, capping, and testing procedures should be taken because specimens are often fragile. Mold stripping techniques have included using a drill or hot probe to place a central hole in the bottom of standard watertight cylinder molds and adding a dry polyester fleece pad on the inside of the cylinder bottom for easy specimen release with or without air compression; splitting the molds with a hot knife; and presplitting molds and reattaching with duct tape for easier specimen removal.

When CLSM is used as subgrade for a pavement or slab, its in-place bearing strength may be important for the designer of the structural element. Bearing ratio tests may be performed in the laboratory or field and subgrade modulus may be determined from field plate load tests. In the field, a properly calibrated pocket penetrometer can be used to determine initial set.

CHAPTER 9—LOW-DENSITY CLSM USING PREFORMED FOAM

9.1—General

CLSM is a self-consolidating cementitious mixture that is intended to result in a compressive strength of 1200 psi (8.3 MPa) or less. Low-density (LD) CLSM not only meets this definition, but its final density is controllable from 20 to 120 lb/ft^3 (320 to 1920 kg/m^3). Because of its low density, LD-CLSM is preferred when reducing dead load is a critical requirement.

Generally, CLSM mixtures contain supplementary cementitious materials (SCM) with some portland cement and other fillers. LD-CLSM usually contains portland cement, possibly some SCM, and preformed foam for most of the volume. Most LD-CLSM applications are alternate solutions to conventional geotechnical solutions such as surcharging soils, bridging poor soils, removal and replacement of poor soils, pile support, and other foundation treatments. In addition, LD-CLSM is easily excavated, a requirement in some applications. The air void or cell structure inherent in LD-CLSM mixtures controls the final density of the mixture, provides thermal insulation, and adds shock mitigation properties to the fill material.

9.2—Applications

LD-CLSM has a very low density, which is a major advantage in most applications. All of the following applications can be constructed with LD-CLSM.

9.2.1 *Backfill*—LD-CLSM is placed against structures such as bridge abutments, retaining walls, and building walls to reduce dead load as much as 75 percent over poor soils. Once LD-CLSM sets, it does not exert active lateral pressure against the wall structure as standard granular backfill does. LD-CLSM is a cementitious material that is consolidated and does not require compaction like soil fills. Settlement is minimal because of its low density. Bridge approach applications are often 10 to 40 ft (3.0 to 12.2 m) or more in height. LD-CLSM is a low-density, self-consolidating fill that is a preferred alternative to standard compacted fill. Usually, LD-CLSM with a maximum in-service density of 30 lb/ft^3 (480 kg/m^3) is cast for most of the fill thickness. The top 2 to 3 ft (0.61 to 0.91 m) may be LD-CLSM with a maximum in-service density of 42 lb/ft^3 (690 kg/m^3), which has excellent resistance to freezing and thawing and provides a solid base for the approach slab or pavement structure.

9.2.2 *Roadway bases*—LD-CLSM is often used as a roadway base over poor soil. Using the material becomes even more important when raising or widening the roadway over poor soil, because added weight and settlement are design concerns (Fig. 9.2.2). These designs often involve load-balancing and buoyancy calculations. Specific site conditions may require special drainage details.

When constructing a roadway over poor soil, a geotextile fabric may be placed after the excavation is complete. The LD-CLSM is cast directly onto the geotextile fabric. This fabric acts as a tension skin and, in conjunction with LD-CLSM, can span most localized settlements.

9.2.3 *Pipeline and culvert fills*—LD-CLSM is often a supporting fill in pipeline applications over poor soils or a containment fill cast around these drainage structures to provide support and stability. Compaction is not necessary as it is with granular fill.

Culvert applications include concrete box culverts, segmented or pipe sections, and metal culvert systems including multi-plate culverts of significant size.

LD-CLSM reduces dead weight on the culvert. The cohesive nature of all CLSM mixtures provides erosion control, which is an advantage over standard granular fills that erode when subjected to moving water. CLSM mixtures may need to be evaluated for freezing-and-thawing resistance.

Placing LD-CLSM on both sides of the culvert simultaneously minimizes eccentric loading. In addition to supporting the culvert from below, LD-CLSM cast around these drainage structures provides lateral support of the culvert or pipeline.

9.2.4 *Void fills*—LD-CLSM is commonly used as a void fill when dead load reduction is critical. It is also applicable to mass structures where access may be limited and flowability is important. Void fill applications include pipeline abandonment, filling around excavations, annular space fills between slip-lined pipes, and structures that are to be abandoned rather than demolished (Fig. 9.2.4).

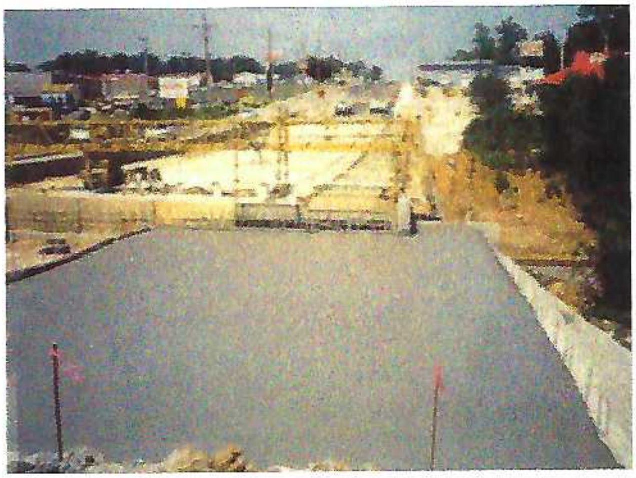


Fig. 9.2.2—Geotechnical roadway base at bridge approach.

Fig. 9.2.4—Filling an abandoned swimming pool with LD-CLSM.

Because every void fill application is unique, each should be examined for special conditions. To contain LD-CLSM, the entire fill area should be sealed, including pipes, drains, and structural discontinuities such as holes in walls or under footings. Lift heights for void fills may be greater than normal if the LD-CLSM can be reasonably contained by earth, forms, or a structure.

9.2.5 *Tank fills*—An acceptable abandonment alternative to the excavation and removal of underground fuel- or oil-storage tanks required by many agencies is a LD-CLSM tank fill (Fig. 9.2.5). The 53 FR 37082-37247 regulations refer to LD-CLSM fills as an "inert substance."

9.2.6 *Insulation and isolation fills*—The discrete air-cell structure within the cementitious matrix of LD-CLSM provides thermal-insulation and physical shock-mitigation properties to this material for applications such as walls (Fig. 9.2.6), roofs, and other similar structures. Giannakou and Jones (2004) describe using LD-CLSM to thermally insulate foundations and slabs.

Table 9.4.2—Physical properties for geotechnical applications (ACI 523.1R-06, 3.11; Elastizell Corporation of America 2013)

Maximum cast density, lb/ft³ (kg/m³)	Minimum compressive strength, psi (MPa)	Bearing capacity, ton/ft² (MPa)
24 (385)	10 (0.07)	0.7 (0.07)
30 (480)	40 (0.28)	2.9 (0.28)
36 (575)	80 (0.55)	5.8 (0.56)
42 (675)	120 (0.83)	8.6 (0.82)
50 (800)	160 (1.10)	11.5 (1.10)

and wall insulation, tunnel and mine fills, energy absorption or shock mitigation, and backfills in sewer and highway construction per ACI SP-29 (ACI Committees 213 and 523 1971).

9.4.3 *Permeability*—Generally, LD-CLSM has a low coefficient of permeability (k) that is constant throughout the lower density ranges (Kearsley and Wainwright 2001a). The coefficient of permeability is inversely related to the effective confining pressure on the sample. Because LD-CLSM is a rigid material rather than a yielding soil, its permeability is measured using a modified triaxial test including a confining pressure to prevent direct water passage (short-circuiting) along the interface between specimen and confining membrane. A constant head should be maintained during the test. Reported k-values at a confining pressure of 2.0 psi (13.8 kPa) range from 5.5×10^{-4} to 4.3×10^{-8} in./s (1.4×10^{-3} to 1.1×10^{-6} cm/s) per ASTM D2434. Recent developments in LD-CLSM mixtures have resulted in greater permeability values due to changes in the preformed foam formulation and, therefore, in its properties. As a result, recently reported values range from 3.9×10^{-1} to 3.4×10^{-6} in./s (1.0 to 1.0×10^{-5} cm/s).

9.4.4 *Freezing-and-thawing resistance*—Freezing-and-thawing resistance of LD-CLSM is evaluated using Procedure B (rapid freezing and thawing) of ASTM C666/C666M, with a modified cycling protocol involving a longer thawing period. This modification is necessary because the insulating properties of LD-CLSM prevent rapid lowering and raising of temperatures at the interior of the specimen and prevent completion of a freezing-and-thawing cycle in the originally prescribed maximum 4-hour period. LD-CLSM intended for exterior exposure should have a relative dynamic modulus of elasticity (E) at least 70 percent of its original value after 120 cycles, when tested according to Procedure B of the modified ASTM C666/C666M. Because the freezing-and-thawing resistance of LD-CLSM increases with increasing density, LD-CLSM within 2 to 3 ft (0.6 to 1 m) of a surface subjected to freezing-and-thawing cycles while exposed to water should have a density of at least 36 lb/ft³ (575 kg/m³). MacDonald et al. (2004) provides an evaluation of the freezing and thawing performance and testing of LD-CLSM.

9.5—Proportioning

Mixture proportioning guidance is generally available from the foam concentrate manufacturers. The mixture proportion specifies the range of proportions of the ingredients needed to attain the desired physical properties (density and compressive strength). The user should test mixture proportions when nonstandard materials or special applications are involved.

9.6—Construction

9.6.1 *Batching*—Materials for LD-CLSM are typically proportioned and batched on site directly into a specialized mixer. The cement, SCM, and other dry materials are weighed on a calibrated scale, and mixing water is metered. Preformed foam is metered into the mixture through a calibrated nozzle. The accuracy of each batching device is critical to the final mixture density and its subsequent reproducibility. Each batching device (scales, water meter, and foam-generating nozzle) should be calibrated before starting a project and during the project if necessary.

9.6.2 *Mixing*—Standard ready-mix equipment is normally not acceptable for LD-CLSM mixtures because the mixer does not combine ingredients with the correct speed and mixing action. A high-speed paddle mixer is preferable because it properly combines the ingredients and blends the preformed foam rapidly and efficiently to produce a uniformly consistent LD-CLSM mixture. Other mixers and processes that produce uniform mixtures include high-shear mixers and some continuous mixers.

In batch mixing, the mixer should be charged with mixture water and dry ingredients, followed by special admixtures and preformed foam. As-cast density should be monitored at the point of placement every 30 to 60 minutes based on consistency of the density. Allowance should be made for any density changes that result from placing methods or conditions, such as pumping distances and extreme weather. Ingredients should be added in proper proportions and sequence during continuous mixing operations. This ensures reasonable uniformity and achieves the required as-cast density at the point of placement.

9.6.3 *Placing*—LD-CLSM should be placed by a progressive-cavity pump or a peristaltic pump. The pump hose should be large enough in diameter (usually 2 to 2-1/2 in. [51 to 64 mm]) to provide uniform delivery of LD-CLSM at the point of placement without damage to the structure or substrate. LD-CLSM can be pumped over 1500 ft (460 m), which is a major advantage over other materials and placing methods, and is important on large, congested projects with difficult access.

9.6.4 *Forming and finishing*—For geotechnical applications, lift thicknesses ranging from 2 to 4 ft (0.61 to 1.2 m) are typical. Lift thickness is job-specific and related to project layout and casting procedure. A greater lift thickness is acceptable for specific job conditions. The heat of hydration developed within the mass, material density, cement content, and the ambient temperature also influence lift thickness. Thinner castings reduce the heat buildup from cement hydra-

1.13 LCC Durability
(Authored BY TAP)

1.13 Long-Term Durability of LCC

While LCC has been around since the 1940s, we were not able to locate any long-term service records in ACI 536.1R-06 Guide for Cast-In-Place Low-Density Cellular Concrete, or with internet research. However, Aerix Industries has provided their Bulletin 18-1602, attached, with recent projects of normal LCC. Not shown, is Mets Stadium in New York, which was constructed in approximately 2006. Four feet of pervious LCC under the playing field provided both drainage for the grass, and raise the elevation over soft soils, similar to the Mission Rock.

By assuming environmental conditions that cause loss of durability and shortened service life in aggregate-based concrete, the TAP investigated the likely long term durability of LCC.

ACI 523.1R-06 "Guide to Cast-in-Place Low-Density Cellular Concrete

ACI523 cites a few topics that can be considered when addressing the long term durability of LCC. <u>Section - 3.4 Drying Shrinkage</u> states drying shrinkage is expected to be 0.3% to 0.6% at six months. Drying shrinkage is not expected to cause problems "When it is used in geotechnical applications, any shrinkage cracking that it might undergo does not significantly reduce bearing capacity". That should be consistent with the TAP's opinion that any cracking caused from earthquake events should not prevent the LCC from continuing to support the concrete and asphalt street sections.

ACI 523 also mentions properties of thermal expansion, "walkability", mechanical attachment, thermal conductivity and fire resistance, none of which are an issue in service at Mission Rock, nor would be a detriment to long-term durability. <u>Section - 3.10 Permeability</u> discusses the permeability of historic, closed-cell, non-pervious foams, which are not being used on this project. <u>Section 3.11 – Freezing and – Thawing Resistance</u> describes C666 testing for environments with routine freezing & thawing environments. Our limited climate research shows San Francisco typically does not have freezing weather in the winter, and even with occasional over-night freezing temperatures, the LCC would be protected by the thermal mass of the street and not likely freeze. So Freeze-Thaw is not a normal concrete deterioration mechanism that would impair the long term durability of the LCC.

ACI318-19 Building Code Requirements for Structural Concrete

ACI 318 lists durability requirements for varying conditions the structural concrete can be exposed to. By looking at these guidelines, and implementing requirements that should also apply to LCC, the durability of the LCC should be improved, and thereby increasing the service life. The various areas affecting durability in structural concrete, and LCC are discussed in the sections below.

<u>Exposure F</u> deals with freeze-thaw environments, which as discussed before is not applicable to the project in San Francisco, due to normal climate.

<u>Exposure S</u> deals with sulfate attack from soils adjacent to the concrete structure. The corrosion report by HJDH Corrosion Consultants, (Appendix D in the geotechnical report) reported sulfates between "non-detect" to 150 PPM, and were considered "non-corrosive" in their report; this would rate the fill soils at an exposure level of S0. While a sulfate exposure of S0 in ACI 318 would not have a specific C150 cement requirement or w/c limitation, the corrosion report had recommendations for concrete in contact with the fill soils to be made with a Type II Portland cement and a maximum w/c ratio of 0.55.

Since LCC can readily be made with a Type II cement and at a 0.55 w/c ratio, we feel it is reasonable to require this conservative approach in the LCC specifications for this project.

Exposure W deals with concrete exposed to water while in service. However, this exposure is primarily a concern with aggregates that may be reactive with the alkalis in the cement in the presence of water. LCC does not contain any aggregates that could react with alkalis, so this durability concern is not applicable to LCC.

The on-going testing with Aerix Industries for the effects of saturation in brackish site water has shown a similar reduction in strength when cured and tested in fresh versus brackish water. However, the compressive strength of the LCC saturated in brackish water continues to gain strength, with the presence of water allowing continued hydration of the Portland cement. This leads us to believe there should not be a durability concern with the LCC's service life, since it is actually getting stronger with time, and no other exposure conditions exist that would cause it to deteriorate.

Exposure C deals with the corrosion protection of reinforcing steel provided by the concrete, when exposed to different weather and chemical de-icers. Since the LCC has no reinforcing steel and will not be exposed to de-icing chemicals nor freeze-thaw cycles, this 318 exposure does not apply to LCC.

ACI301-16 Specifications for Structural Concrete
ACI301 has similar durability concerns as ACI 318, with more material specifics. For example, ACI301 provides tables for limiting chloride contents of the concrete mixture, for either pre-stressed or normally reinforced concrete, based on the environmental exposure to chlorides. The on-site brackish water we tested in our long-term durability study had a 11ppm chloride content, much less than any of the chloride limits in the ACI301 Table 4.2.2.7 (d). This and results of continued strength gain when saturated in brackish water indicate long-term durability should not be an issue in service.

Comment 35 : Cracking, Distortion, Uplift, Stiffness within or above the LCC
LCC is a rigid solid material after initial hardening, and is not expected to expand or contract to any significant amount. Small drying shrinkage cracks are likely to be observed at the surface and propagate downward, where moisture loss will be minimal and also reduce drying shrinkage. Even with earthquake events that may cause some cracking and short-distance rubblizing of the LCC at those crack zones, the overall volume of the LCC subgrade beneath the streets is expected to be constant and continue to support vertical loads from traffic on the pavement section.

LCC when hardened, is an essentially volume-stable material, not readily expanding with moisture changes like dry clays, or compressing under loads like the Bay mud. Modulus testing for Langan on three leftover 27pcf samples was performed at 92days, resulting in an average Modulus of Elasticity of 4545psi; the original 28days strength of those samples was 84psi. In service, the temperature will be fairly constant, so no measureable volume change is expected with its low coefficient of thermal expansion. The LCC itself is not expected to distort itself, but its elevation may change with localized differential settlement of the underlying fill materials and Bay mud.

The TAP concurs with the process of performing compressive strength samples as the LCC approaches the end of the 10 year warrantee period, to determine if it is still performing as expected. Compressive strengths obtained can be used to calculate the Removability Modulus at that time, to determine if it still complies with ACI 229-13 recommendations of an RE of less than 1.0.

Bulletin 18-1602 Low-Density Cellular Concrete (LDCC)
Typical Geotechnical Fills

Location	Project Name	Owner	Description	Approximate Installation Date
Los Angeles, CA	North Outfall Replacement Sewer (LANORS)	Los Angeles Dept. of Public Works	84,000 yd³ annular fill for 46,867 ft of 12 1/2 ft and 8 ft ID PCCP	Prior to 1999
Sacramento, CA	Consumnes River Boulevard	City of Sacramento	16,500 yd³ PLDCC to mitigate the loads placed on the below-grade pipework	2010+/- several years
Oakland, CA	Brooklyn Basin, Estuary Fill	Signature Development Group	17,000yd³ for soil stabilization	2015 +/- several years
Summit County, CO	I-70 Roadway Settlement	CDOT	Settlement mitigation plan utilizing drilled shafts filled with LDCC	2010 +/- several years
Walden, CO	Michigan Ditch Tunnel	City of Ft Collins	1,000yd³ annular fill at a relatively high elevation	2010+/- several years
Ocoee, FL	SR 50 FL Lane Expansion	Florida DOT	2,800yd³ of LDCC was used to reduce the weight associated with traditional fills	2019
Atlanta, GA	Pipe Abandonment	Atlanta Gas Light (AGL)	6,500yd³ of 40pcf LDCC	2018
Honolulu, HI	Kaneohe Kailua Tunnel	City & County of Honolulu, Dept. of Environmental Services, Dept. of Design Construction	One of the longest one point of placement projects for an annular fill between a 13' foot tunnel lined with a 10' gravity sewer line	2018

Baton Rouge, LA	Culvert Annular Fill	Louisiana DOT	265yd³ annular placement under Hwy 61	2018 +/-
New Orleans, LA	The Chapel	St. Joseph Academy	460yd³ of PLDCC was used to bring the subgrade up to elevation	2018+/-
New Orleans, LA	Louis Armstrong International Airport	New Orleans Louis Armstrong Airport	3,000yd³ PLDCC used to create a net zero loading factor in extremely soft soils	2018+/-
Boston, MA	Fore River Bridge	City of Boston	6,000yd³ for bridge approach	2017+/-
Lexington, MA	Annular Underground Pipe Fill	City of Lexington	70 yd³ of LDCC	2017
Cambridge, MA	Central Square Station	Massachusetts Bay Transit Authority	1000yd³ fill to reduce the load over a subway station extension	2017
Lusby, MD	Dominion Energy CovePoint Station	Dominion Energy	600yd³ of LDCC used to reducing the potential for LNG vaporization.	2017
Augusta, ME	Joint Force Headquarters Phase 1	ME Dept of Defense	Project was to stabilize the soil in areas where it could not support the necessary structures	2017
Freeport, ME	Frost Gully Culvert U.S. Route 1	Veterans MaineDOT	Annular Fill project included shoring up the embankment, repairing and placing the annular fill	2017
Branson, MO	Utility Trench Back Fill	City of Branson, MO	PLDCC was used to reduce soil settlement concerns used in the trench construction	2017
Fargo, ND	Rose Coulee Bridge	State of North Dakota	2,600yd³ PLDCC used to stabilize a long-term maintenance issue	2015
Fayetteville, NC	NC 72 Bridge: Void Fill Under Bridge Approach-	North Carolina DOT	LDCC used to fill the void created by a washout at the bridge abutment	2016
New York, NY	New England Thruway, Interstate I-95	NY State Dept. of Transportation	7800 yd³ for backfill and 100% compaction of utility structures beneath the North and South bound roadways	2015
Brooklyn, NY	Coney Island Main Repair Facility	York City Transit Authority	LDCC was used to fill voids beneath the existing floor thereby saving the slab and avoiding future settlement.	Prior to 1999
Chester County, PA	Route 30 Sinkhole	PennDOT	1,200 yd³ of Permeable Low-Density Cellular Concrete (PLDCC)	2018

Philadelphia, PA	I-95 Lane Expansion	PennDOT	6,800 yd^3 back fill for a retaining structure	2017
Malvern, PA	Route 202 Bridge & Culvert	PennDOT	Soft Soil Remediation with 18,000 yd^3 at 1,000 yd^3/day	2016
Arlington, VA	Washington Boulevard Bridge	City of Arlington, VA	1,100 yd^3 for a culvert abandonment	2016
Bellingham, WA	SR 542 Anderson Creek	Washington State DOT	PLDCC was used to create two bridge abutments with shotcrete over the facia walls	2017
Woodville, WA	Brightwater Treatment System	King County Waste Water Treatment Division	Annular fill for conveyance tunnel - 14,000 feet long and 18 feet in diameter and approximately 260 feet below underground	2010 +/-
Martinsburg, WV	CSX Rail Station	CSX Rail	33yd^3 Annular fill between the 12" sewer line and the 10" water line	2010+/-
Brisbane, Australia	Brisbane Airport	Brisbane Airport Commission	Abandonment of 280 meter culvert	2012+/-
Ontario, Canada	Hanlan Water Project	Regional Municipality of Peel	19,300 yd^3 of LDCC annular fill project	2010+/-
Hamilton, New Zealand		Bechtel Petroleum & New Zealand Synthetic Fuels Corp.	11,500 yd^3 as positive fill for underground steel support structures	2010+/-
Japan	Takaido Trunk Line	Japanese National Railway	Backfill for 28 separate railroad tunnels through mountains-total length, 40 km	Prior to 1999
Tokyo, Japan	Void Fill		Low-Density Fill for abandoned bomb shelters to eliminate subsidence problems	Prior to 1999
Bogota, Colombia	Load Reduction Fill	Bogota International Airport	LDCC for load reduction backfill around foundation for new expansion building (<550m3)	Prior to 1999
Puerto Rico	Bridge Approach	???	??? (contact NewCon)	2014
Guatemala	Pipeline Abandonment Backfill	Undisclosed	150yd3 of a maximum 40pcf mix.	2008+/-

1.14 Natural Saturation Density Testing (Authored BY TAP)

1.14 Natural Saturation Density Testing

Public Works' review of the Pilot Project Report noted that several field saturated density tests were below the new 50pcf specification minimum for LCC saturated density. This situation raised questions on the buoyancy calculations. Langan analyzed all the field and laboratory test results from CellCrete's production before and during the Pilot project, as well as two other potential LCC contractor's submitted samples as tested as "pre-qualification" tests. Stan Peters of the TAP reviewed the results as well and provides the following discussion on the results with recommended refinements to the testing procedures.

The data analysis is discussed below, as well as descriptions of permeability and saturation testing methods used, both in the "field" and in the laboratory. Also discussed are ways to improve the consistency and accuracy of the "field" saturation density test (that was developed specifically for this project), prior to its use during project production of LCC.

However it should be noted that the TAP concurs with Langan's opinion, after analyzing all the data presented and graphed in the attached Xcel worksheet, that the LCC produced for the pilot project readily met the new proposed 50pcf minimum specification (when tested properly), and that the 50pcf is a valid minimum design value for subsequent buoyancy calculations.

Description of Testing Methods

Laboratory permeability was performed utilizing an adaptation of a standardized test for measuring permeability of granular soils with permeability coefficients similar to what is generally observed in PLCC. This test method is ASTM D2434 "Standard Test Method for Permeability of Granular Soils (Constant Head)." This test has been adapted by modifying the sample preparation method due to the unique material characteristics of LCC. Granular soils such as sand would normally be compacted within the permeability cell, and contained with porous stones over each end, and fine screens over the monometer ports. LCC for permeability testing is wet-cast into custom-fabricated PVC permeability cells (with drilled and tapped manometer ports, plugged during casting), and shipped back for subsequent D2434 testing. The porous stones on the top and bottom of the sample are omitted, as well as manometer port screens, as they are not necessary to contain the material. Casting the PLCC directly into the cylinder ensures that there is no sidewall leakage between the sample and the cylinder walls. The actual testing procedures are the same as those outlined in ASTM D2434.

Castle Rock Consulting (CRC) fabricates these permeameter cell cylinders from 3" diameter schedule 40 PVC pipe at cylinder lengths of 6", with manometer ports at the requisite 3" distance. These cylinders can be sent to jobsites, where the PLCC can be cast in the cylinders and returned after setting, to be tested in the D2434 apparatus.

A field-performed "falling head" permeability test was developed at the request of the Port, as a means of on-site validation of the permeability of the LCC produced on site. This test was an advancement of a simple "percolation rate" test CRC had developed for on-site QC testing using a modified 6” x 12” concrete cylinder mold, filled halfway with LCC and timing the flow rate of water through the LCC. In the “falling head” method, the time in seconds is recorded for the water level to drop from 4” above the LCC surface to 1”, and a permeability is calculated using the falling head equation.

During the Pilot project, it was recognized that the permeability of the LCC was readily sufficient for water level changes in service, and that the saturation density was a more useful physical property to measure and maintain, for both de-watering durations and for final buoyancy in service. Natural saturation density was first measured in the laboratory with the permeability test cells in a saturation versus hydraulic pressure study, then became part of the D2434 test procedure in measuring and reporting the saturation density, prior to conducting the laboratory permeability test procedure.

A field test method was developed to test 4”x8” test samples for saturated density. Initially the samples were cured for three days, before saturating them submerged in a 5gallon bucket of water for 30 minutes; the soak time was first validated with shorter soak times. The sample was then tested in an ordinary concrete air meter, and the saturated density calculated.

Analysis of Data

As shown on the first page of the attached tables, the saturated densities of field testing averaged about 4pcf lower than when these same samples were measured in the laboratory, prior to D2434 permeability testing. The field data also had a much larger variability than the lab results. The laboratory density measurements were all at 50pcf or higher.

In association with the Pilot project, CRC also tested “submittal samples” of LCC from two other contractors, potentially producing final LCC during construction. Samples included both field saturation density and falling head sample specimens, as well as D2434 permeability molds.   As shown in the table and graph, the standard deviation of the density tests was much smaller than on the project (albeit it with one experienced staff engineer, under controlled conditions), and the difference between the field and lab tests was closer. In all cases, 50pcf was achieved.

Improvements in Field Testing

The field saturation test will be modified to begin overnight soaking after 48hours, with a minimum of 12 hours submerged in the bucket, prior to testing. CRC is fabricating proficiency samples to send to three technicians in San Francisco, three at a certified laboratory in Denver, and one for CRC’s testing. Samples will be two field saturation samples and two field permeability samples per set. The field saturation is being modified to include an overnight soak, to hopefully decrease the difference between field and laboratory test results. It should be noted that the previous hydraulic head for the laboratory saturation density was approximately 2’; it will be modified to approximate the water level in the bucket.

Field & Lab Testing Data Summary, All Samples For Mission Rock						
Sample ID	Field Testing			Lab Testing		
	Cast UW (pcf)	Sat. UW (pcf)	Permeability (cm/sec)	Un-sat UW (pcf)	Sat. UW (pcf)	Permeability (cm/sec)
Stanford 1	27.5	40.5	8.77E-03	27.5	51.1	5.50E-02
Stanford 2	27.5	53.4	1.34E-01	29.5	50	1.80E-03
Stanford 2				29.5	52	5.70E-04
Stanford 3			6.18E-04	26.5	54	1.30E-01
Stanford 3				26.5	55	8.80E-02
Pilot Lift 1	25.8	51	8.65E-03	26	53.5	1.10E-01
Pilot Lift 1		57.5	4.09E-01	26	53.1	5.70E-02
Pilot Lift 1				25.5	58.1	5.20E-01
Pilot Lift 1				25.5	58.7	4.30E-01
Pilot Lift 1 Pour Back		60.7	1.70E+00			
Pilot Lift 2	27.8	47.7	9.34E-01	28.7	53.1	4.20E-02
Pilot Lift 2				28.9	52	3.70E-02
Pilot Lift 2				29	53.6	4.30E-02
Pilot Lift 2		40.8	4.25E-01	28.9	54.5	5.90E-02
Pilot Lift 3	28.3	56.3	1.01E-02	29	55.7	5.40E-02
Pilot Lift 3		53.2		29	56.8	1.20E-01
Pilot Lift 3				27.5	57.1	1.00E-01
Pilot Lift 3				27.5	55.8	7.30E-02
Pilot Lift 4	28.5	47	8.31E-03	29	52.8	2.90E-02
Pilot Lift 4		46.9		29	52.5	6.20E-03
Pilot Lift 4		44.3				
Pilot Lift 4		45.2				
Pilot Lift 5		44.3	3.73E-02	**30.8**	**51.6**	6.30E-03
Pilot Lift 5				**31.1**	**52.8**	1.10E-02
Utility Pour Back 1	24.9	54.5	2.48E-03			
Utility Pour Back 2		54.8	4.95E-04			

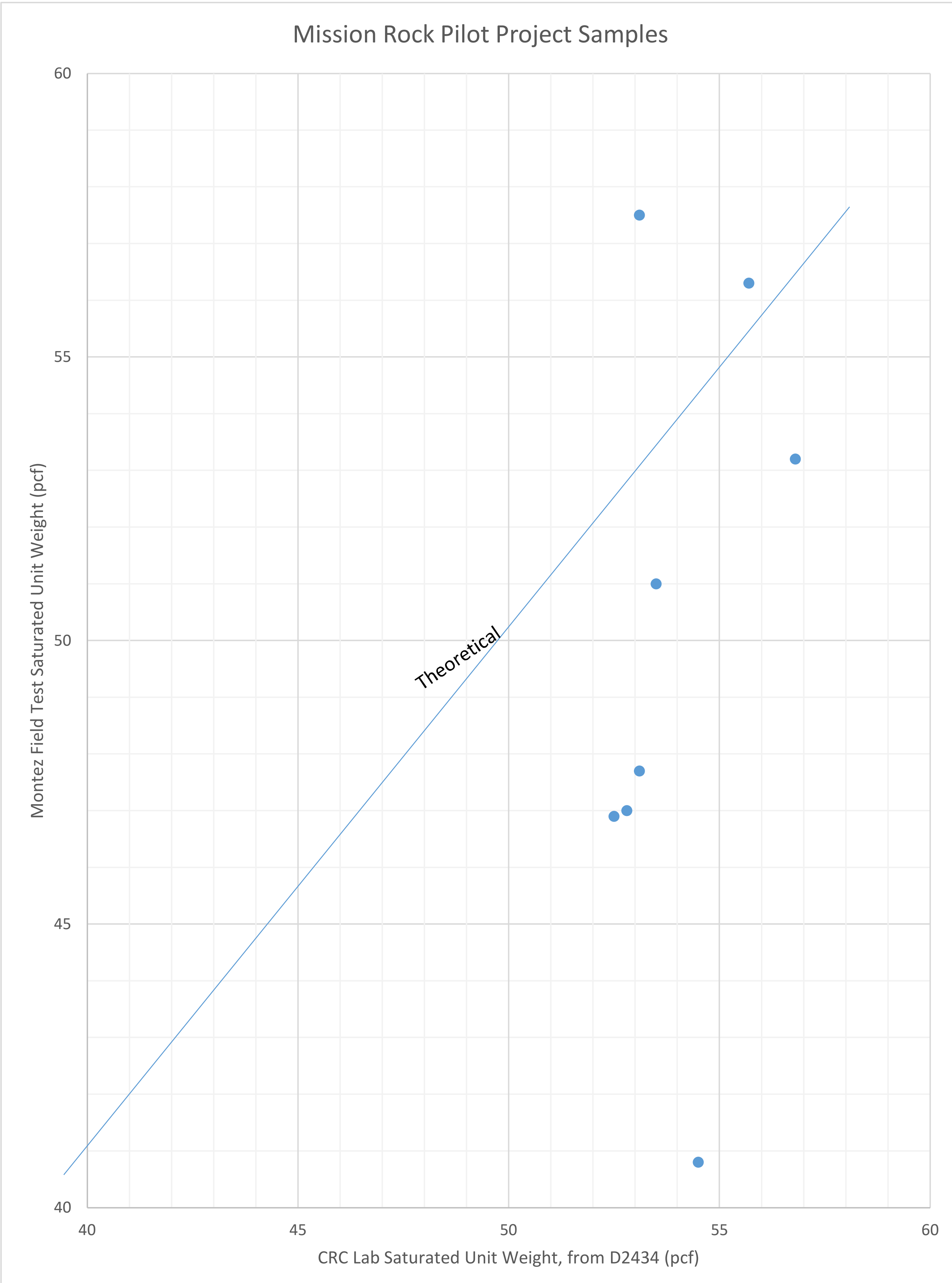
Data from All Cast Densities			
Field Testing		Lab Testing	
Average =	49.9	Average =	54.0
Standard Deviation =	5.9	Standard Deviation =	2.3
Minimum=	40.5	Minimum=	50.0

Data from Lower Cast Densities (26 to 29 pcf) - Excludes Lift 5 from Pilot			
Field Testing		Lab Testing	
Average =	50.3	Average =	54.2
Standard Deviation =	5.9	Standard Deviation =	2.3
Minimum=	40.5	Minimum=	50.0

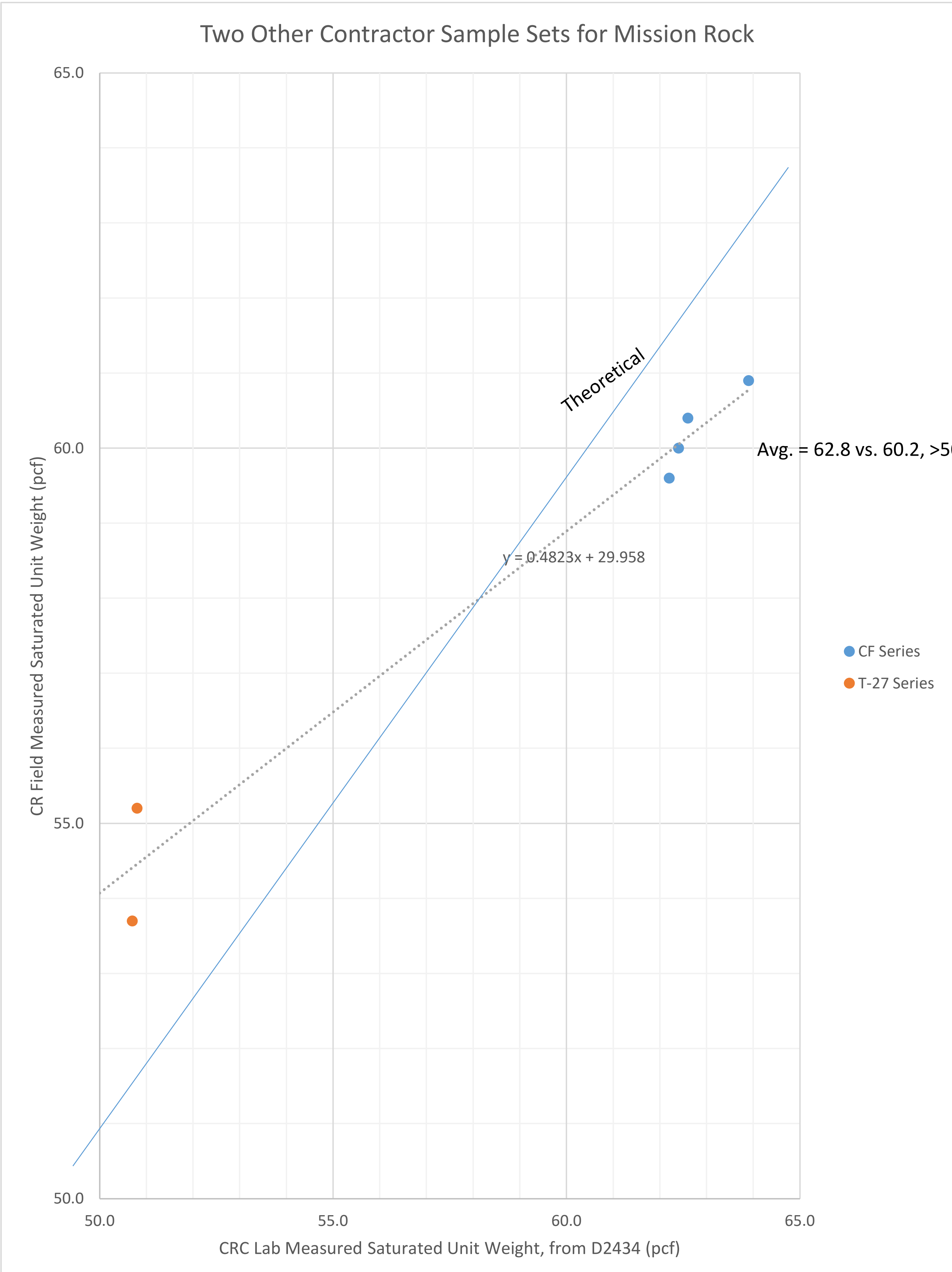
Bolded Values were reported incorrectly in Montez Report Table

Relevant Mission Rock Pilot Samples with Field & Lab Saturation Testing

Sample ID	Field Testing			Lab Testing			Field - Lab Sat. Difference (pcf)
	Cast UW (pcf)	Sat. UW (pcf)	Permeability (cm/sec)	Un-sat UW (pcf)	Sat. UW (pcf)	Permeability (cm/sec)	
Pilot Lift 1	25.8	51	8.65E-03	26	53.5	1.10E-01	-2.5
Pilot Lift 1		57.5	4.09E-01	26	53.1	5.70E-02	4.4
Pilot Lift 2	27.8	47.7	9.34E-01	28.7	53.1	4.20E-02	-5.4
Pilot Lift 2		40.8	4.25E-01	28.9	54.5	5.90E-02	-13.7
Pilot Lift 3	28.3	56.3	1.01E-02	29	55.7	5.40E-02	0.6
Pilot Lift 3		53.2		29	56.8	1.20E-01	-3.6
Pilot Lift 4	28.5	47	8.31E-03	29	52.8	2.90E-02	-5.8
Pilot Lift 4		46.9		29	52.5	6.20E-03	-5.6
	Average:	**50.1**	-		**54.0**		
	Standard Deviation:	**5.6**	-		**1.5**		


Mission Rock Pilot Project Samples
Montez Field Test Saturated Unit Weight (pcf)
CRC Lab Saturated Unit Weight, from D2434 (pcf)
Theoretical
60
55
50
45
40
40
45
50
55
60

Other Contractor Samples for Mission Rock Project					
Sample ID	**Field Testing**		**Lab Testing**		
	Sat. UW (pcf)	**Permeability (cm/sec)**	**Un-sat UW (pcf)**	**Sat. UW (pcf)**	**Permeability (cm/sec)**
T-27-60-A	53.7	1.40E+00	28.0	50.7	4.40E-01
T-27-60-B	55.2	1.50E+00	30.4	50.8	3.10E-01
CF-20	60.0	3.10E+00	23.3	62.4	1.60E+00
CF-25	60.4	1.80E+00	25.9	62.6	8.20E-01
CF-27	59.6	5.20E-01	26.5	62.2	7.30E-01
CF-30	60.9	2.60E-01	30.8	63.9	1.80E-01
CF-Average	60.2			62.8	
CF-StdDev	0.5			0.7	
Overall Average	**58.3**			**58.8**	


Two Other Contractor Sample Sets for Mission Rock
65.0
60.0
55.0
50.0
50.0
55.0
60.0
65.0
CR Field Measured Saturated Unit Weight (pcf)
CRC Lab Measured Saturated Unit Weight, from D2434 (pcf)
Theoretical
Avg. = 62.8 vs. 60.2, >5(
y = 0.4823x + 29.958
CF Series
T-27 Series

1.15 Factor of Safety Against Buoyancy (Authored BY TAP)

1.15 Discussion of Factor of Safety Against Buoyancy (Flotation) of Light Weight Cellular Concrete

Definitions and Background

Hydrostatic – "It encompasses the study of the conditions under which fluids are at rest in stable equilibrium as opposed to fluid dynamics, the study of fluids in motion. Hydrostatics are categorized as a part of the fluid statics, which is the study of all fluids, incompressible or not, at rest (https://en.wikipedia.org/wiki/Hydrostatics)."

Hydrostatic Pressure – "In a fluid at rest, all frictional and inertial stresses vanish and the state of stress of the system is called *hydrostatic*. When this condition of $V = 0$ is applied to the Navier–Stokes equations, the gradient of pressure becomes a function of body forces only. For a barotropic fluid in a conservative force field like a gravitational force field, pressure exerted by a fluid at equilibrium becomes a function of force exerted by gravity.

The hydrostatic pressure can be determined from a control volume analysis of an infinitesimally small cube of fluid. Since pressure is defined as the force exerted on a test area ($p = F/\boxed{A}$, with p: pressure, F: force normal to area A, A: area), and the only force acting on any such small cube of fluid is the weight of the fluid column above it, hydrostatic pressure can be calculated according to the following formula:

$$p(z) - p(z_0) = \frac{1}{A}\int_{z_0}^{z} dz' \iint_A dx'\, dy'\, \rho(z')g(z') = \int_{z_0}^{z} dz'\, \rho(z')g(z'), \tag{1}$$

where:

- p is the hydrostatic pressure (Pa),
- ρ is the fluid density (kg/m^3),
- g is gravitational acceleration (m/s^2),
- A is the test area (m^2),
- z is the height (parallel to the direction of gravity) of the test area (m),
- z_0 is the height of the zero reference point of the pressure (m).

For water and other liquids, this integral can be simplified significantly for many practical applications, based on the following two assumptions: Since many liquids can be considered incompressible, a reasonable good estimation can be made from assuming a constant density throughout the liquid. (The same assumption cannot be made within a gaseous environment.) Also, since the height h of the fluid column between z and z_0 is often reasonably small compared to the radius of the Earth, one can neglect the variation of g. Under these circumstances, the integral is simplified into the formula:

$$p - p_0 = \rho g h, \tag{2}$$

where:

- h is the height $z - z_0$ of the liquid column between the test volume and the zero-reference point of the pressure.

This formula is often called Stevin's law[3][4] (https://en.wikipedia.org/wiki/Hydrostatics)."

For geotechnical applications, the zero-reference pressure is set to zero to represent zero gauge pressure; in other words, zero gauge pressure equals 1 atmosphere of absolute pressure. The hydrostatic pressure at depth h is simply ρgh where g is the gravitational constant.

"**Buoyancy** or upthrust, is an upward force exerted by a fluid that opposes the weight of a partially or fully immersed object. In a column of fluid, pressure increases with depth as a result of the weight of the overlying fluid. Thus the pressure at the bottom of a column of fluid is greater than at the top of the column. Similarly, the pressure at the bottom of an object submerged in a fluid is greater than at the top of the object. The pressure difference results in a net upward force on the object (https://en.wikipedia.org/wiki/Buoyancy)."

For 1-D geotechnical buoyancy calculations, the buoyant force per unit volume of soil or material is:

$$F_b = \gamma_{total} - \gamma_{water} \quad (3)$$

where: γ_{total} is the total unit weight of the soil or material per unit volume (i.e., weight of material solids + weight water), and γ_{water} is the unit weight of water (i.e., 62.4 lb/ft^3).

For completely saturated soils, there are two phases present: (1) solids and (2) water filled voids. In this case, the γ_{total} is equal to (i.e., weight of material solids + weight of water) because there would be no volume associated with air voids in a completely saturated material.

For partially saturated soils or materials, γ_{ps}, the unit weight is calculated from:

$$\gamma_{ps} = \gamma_{dry} (1+\omega/100) \quad (4)$$

where: γ_{dry} is the dry unit weight (i.e., weight of the oven-dried soil or material) and ω is the moisture content of the soil (%).

For design purposes, values of γ_{ps} are usually determined from long-duration laboratory saturation tests in controlled conditions. For materials such a permeable lightweight cellular concrete (P-LCC), the material does not obtain complete saturation due to isolated void/air pockets remaining with the P-LCC fabric that resists saturation because such are not hydraulically connected to the fluid in the fabric of the specimen.

A material specimen with a γ_{ps} less than 62.4 lb/ft^3 will have a net upward buoyant force per unit volume, F_b, that can be estimated for 1D conditions using:

$$F_b = 62.4 - \gamma_{ps} \quad (5)$$

This is fundamentally a hydrostatic calculation. Hydrodynamic forces (i.e., the upward flow of water) are not considered in the evaluation of against buoyancy uplift. If such forces are present and significant, then a heave calculation is also performed that combines the buoyant and upward seepage forces.

Also, because this is a 1D calculation, any resisting forces due to upward shearing between the P-LCC block and surrounding soil are conservatively ignored. Furthermore, any basal bonding of the P-LCC with the underlying soil is also ignored. Thus, in reality the hydrostatic forces required to uplift a block of P-LCC are greater than those calculated from the 1D analysis.

Allowable Stress Design

Allowable stress design (ASD) addresses uncertainty in the capacity and demand equation by applying factor an overall factor of safety (FS).

$$FS = F_C / F_D \quad (6)$$

where: F_C is the forcing resisting potential failure of the system, and F_D are the driving forces that cause failure.

In geotechnical calculations, the actual FS of a system is calculated using the best-estimate or most-likely values of F_C and F_D. The best-estimate values are represented as mean values for symmetrical distributions, median values for slightly skewed distributions, and geometrical mean for log-normal distributions (i.e., highly-skewed distributions). Generally, values of γ_{ps} for P-LCC are assumed to be normally distributed with the mean representing the best-estimate for inputs for Equation (6).

Load and Resistance Factor Design (LRFD)

LRFD is widely used as an alternative to working stress design (WSD) and is popular in structural and geotechnical engineering codes. LRFD has been adopted by the following institutions and societies.

- ACI (American Concrete Institute)
- AASHTO (American Association of Highway and Transportation Officials)
- ASCE (American Society of Civil Engineers)

LRFD considers uncertainty in both the loads and the resistance of the material(s). The overall equation is:

$$\Sigma\ (LF)_i Q_{ni} \leq \Sigma\ (RF)_i R_{ni} \quad (7)$$

where LF = load factors, Q_n = nominal loads, RF = resistance factors, R_n = nominal resistances.

The nominal loads and resistances are either code-specified or determined using best-estimate values similar to that of WSD. The load factors are generally greater than one and increase the design load to account for uncertainty, whereas the resistance factors are generally less than one and decrease the design resistance to account for uncertainty. The design is found to be acceptable as long as the sum of the factored resistance equals or exceeds the sum of the factored loads.

Discussion of MRP Design Inputs

The applicant for the Mission Rock Project (MRP) proposes to use ASD to calculate factors of safety against buoyancy uplift of the P-LCC based on guidance from U.S. Army Corp of Engineers (EM 1110-2-2100) and National Cooperative Highway Research Program (NCHRP) 529.

NCHRP 529 requires a FS of 1.2 for the design basis groundwater elevation or flood event. Consistent with ASD methodology, best-values of γ_{ps} are recommended in calculating the factor of safety.

The guidance in EM 1110-2-2100 pertains to calculating the factor of safety against flotation of concrete structures:

$$FS_f = \frac{W_S + W_C + S}{U - W_G} \quad (8)$$

where

W_S = weight of the structure, including weights of the fixed equipment and soil above the top surface of the structure. The moist or saturated unit weight should be used for soil above the groundwater table and the submerged unit weight should be used for soil below the groundwater table.
W_C = weight of the water contained within the structure
S = surcharge loads
U = uplift forces acting on the base of the structure
W_G = weight of water above top surface of the structure.

For the MRP evaluations, Equation (8) can be simplified to a 1D calculation:

$$FS_f = S / U$$

where S = the sum of the total unit weights of all materials above the water table multiplied by their respective thickness and U = γ_{ps} multiplied by the thickness of the submerged P-LCC. Note that FS_f has units of F/L^2 or pressure.

Site Information Category	Load Condition Categories Usual	Unusual	Extreme
All Categories	1.3	1.2	1.1

The value of FS varies according to the load conditions, as given in the table above. The return period of the loading condition category is given in the table below.

Load Condition Categories	Annual Probability (p)	Return Period (t_r)
Usual	Greater than or equal to 0.10	Less than or equal to 10 years
Unusual	Less than 0.10 but greater than or equal to 0.0033	Greater than 10 years but less than or equal to 300 years
Extreme	Less than 0.0033	Greater than 300 years

The factor of safety recommended in EM1110-2-2100 assumes that for critical and normal structures, the soil and material properties have been "conservatively" established through the explorations and testing. The MRP project proposes to use a design value for γ_{ps} of 50 lb/ft^3. Based on the laboratory testing performed for the Pilot project, most of the laboratory determined values of γ_{ps} were in mid-range between 50 and 60 lb/ft^3 with a minimum value of 52 lb/ft$^{3.}$ The laboratory saturated density testing of 14 samples of the 27 lb/ft^3. The P-LCC of Lifts 1 through 4 of the Pilot Project averaged 54.8 lb/ft^3 with a standard deviation of 2.1 lb/ft^3.

Hence, the TAP agrees with the MRP design team that 50 lb/ft^3 is an acceptable conservative design value for calculating buoyancy or flotation as required by EM1110-2-2100.

In its review, he San Francisco Department of Public Works (DPW) raised the issue that NCHRP 529 is not an appropriate design standard for P-LCC. The primary concern is that NCHRP 529 pertains to geofoam design, and geofoam due to is in-plant manufactured nature might be less variable in its unit weight properties when compared with P-LCC; hence the factor of safety of recommend 1.2 may not be adequately conservative for P-LCC design. Following this issue, there was considerable discussion about applying an "extra factor" of safety to the design value of γ_{ps} of the P-LCC to account for additional variability suggested by the laboratory and field test values of γ_{ps}. A lower bound estimate of mean minus two standard deviations was suggested for γ_{ps} by City Engineers or Reviewers.

The TAP notes the following: (1) Both NCHRP 529 and EM1110-2-2100 have relatively consistent recommended factors of safety (1.2 vs. 1.1 to 1.3, respectively). (2) Using a mean minus two standard deviation estimate of γ_{ps} is unprecedented in engineering design and is not required by current codes and documents. (3) The coefficient of variation (standard deviation divided by the mean) for both geofoam and P-LCC has not been determined. Therefore, it is premature to draw any conclusions regarding additional variability that might be present in γ_{ps} when compared with the coefficient of variation for the density of geofoam. (4) EM1110-2-2100 requires that the designer consider the potential variation in soil properties, which are natural materials with considerable variability in γ_{ps} values. Regarding this, as stated above, the TAP believes that the MRP designers have selected a conservative value for γ_{ps}. (It should be noted that γ_{ps} of 40 lb/ft^3 discussed during the meeting was obtained from a "field" saturation and not a laboratory test; hence it is probably not representative of the true range of γ_{ps}.) The MRP team has proposed a revised procedure for the "field" saturation test to ensure better consistency with laboratory-determined values. Stan Peters of the TAP has reviewed the revised procedure and concurs with the changes. Nonetheless, it is recommended that estimates of γ_{ps} that support the basis of design should be obtained from laboratory-determined values to avoid confounding variability from the two test methods.

Discussion of LRFD Design Guidance

The load and resistant factors in LRFD allow for separate treatment of the uncertainty associated with loading conditions and soil properties. It is another design method that could be considered by the MRP design team, or the rationale and load factors of LRFD can be used to justify the ASD parameters proposed by the MRP design team.

The American Association of State Highway and Transportation Officials (AASHTO) in its LRFD specifications for bridge design (2017, 8th Edition) uses a load factor (WA) for extreme events of 1.00 (p. 3-15, Table 3.4.1-1). The extreme events include loading combinations relating to ice load, collision by vessels and vehicles, check floods, and certain hydrostatic events with a reduced live load other than that which is part of a vehicle collision (p. 3-10). Therefore, for LRFD, the TAP recommends a load factor of 1.00 for the extreme flood event, consistent with that of the WA load factor from AASHTO.

It should be noted that resistant factors, when unknown, are often selected so as to produce a factor of safety consistent with that obtained from allowable stress design. For example, this is the approach taken by BS 6349-3 for maritime structures (British Standard 6349-3 (1998) Maritime structures – Part 1: Code of Practice for general criteria). BS 6349-3 requires a factor of safety not less than 1.2 against hydrostatic uplift. For a favorable stability weight, a reduction factor applied to the mean value of γ_{ps} of 1/1.2 = 0.83 is recommended by BS 6349-3. In addition, Simpson, Vogt and van Seters (2011) conclude “In uplift problems, it is necessary to vary either water pressure or the magnitudes of favorable, stabilizing weight, in order to ensure safety in view of possible secondary actions. In order to avoid factoring water pressure, the possibility of a reduced factor on favorable weight, perhaps between 0.8 and 0.9 should be considered (p. 517).”

Lastly, current AASTHO specifications do not suggest a resistance factor for buoyancy. Hence, until available, the TAP recommends an AASHTO resistance factor of 1/1.2 = 0.83333. . . to be consistent with the safety factors recommended by EM1110-2-2100 and NCHRP 529 if AASHTO LRFD is used.

1.16 Technical Memorandum on Saturated Density, Groundwater, and Supporting Calculations
(Authored By Developer Team)

LANGAN

Technical Memorandum

501 14th Street, 3rd Floor Oakland, CA 94612 T: 510.874.7000 F: 510.874.7001

To: Mr. Steve Minden

From: Peter Brady, PE, and Scott A. Walker, GE

Date: 28 April 2020

Re: Response to City Comments on Lightweight Cellular Concrete
Mission Rock Phase 1 Horizontal Improvements
San Francisco, California
Langan Project No.: 750604203

This memorandum presents our responses to several review comments provided by the City and County of San Francisco regarding several of the design parameters used for raising the proposed streets with permeable lightweight cellular concrete (LCC) for Phase 1 Horizontal Improvements of Mission Rock Project in San Francisco, California.

Saturated Densities used in LCC Calculations

Two different saturated densities are used in the evaluation of the LCC in order to capture the potential for variation in the saturated density of the LCC. One saturated density is 68 pounds per cubic foot (pcf), which is used in determining the overexcavation depth to result in load offset when replaced with LCC and the other improvements. The other saturated density is 50 pcf, which is used in the calculations to check for buoyancy due to sea level rise (SLR).

We used two different saturated densities (the higher density for load offset and lower density for buoyancy) because each value is appropriately conservative for the different evaluations. We understand that DPW is requesting additional backup to justify the use of each density.

The 68 pcf density is the "heavy" density of LCC under long-term saturated conditions for the load offset calculations. This value is the average of the highest representative density from laboratory density tests, where samples of LCC were vacuum saturated, and the theoretical maximum saturated density at 100 percent saturation of all of the air voids with brackish water. See attached Exhibit A for the calculated maximum saturated density and the laboratory data. A density of 62.5 pcf is the highest vacuum-saturated density test recorded in a representative LCC sample (with a similar cast density to the specified density at Mission Rock), and the theoretical maximum saturated density is 74 pcf. We conclude that taking the average of these two values is appropriate in the load-offset calculations.

The 50 pcf saturated density used for the buoyancy resistance is based on the lightest saturated density measured for the project in the laboratory. We recognize that several of the "field saturated densities" fall below this value. The field saturated density tests performed during the pilot test and on the samples taken from the Stanford project were based on a new field test procedure being developed for this project. However, samples of the same materials tested in the more controlled environment of the laboratory all had saturated densities above 50 pcf. Please see Appendix A for a summary of all of the field and laboratory density data available for the project. We conclude that the high degree of variability shown in the field test data is not representative of the actual behavior of the LCC, and the test procedure should be updated to include longer initial saturation times. However, even the field saturated density test data also had an average density greater than 50 pcf. Based on the available laboratory data and understanding of the project, we conclude that the density of 50 pcf for the buoyancy resistance calculations is appropriately conservatively.

Groundwater

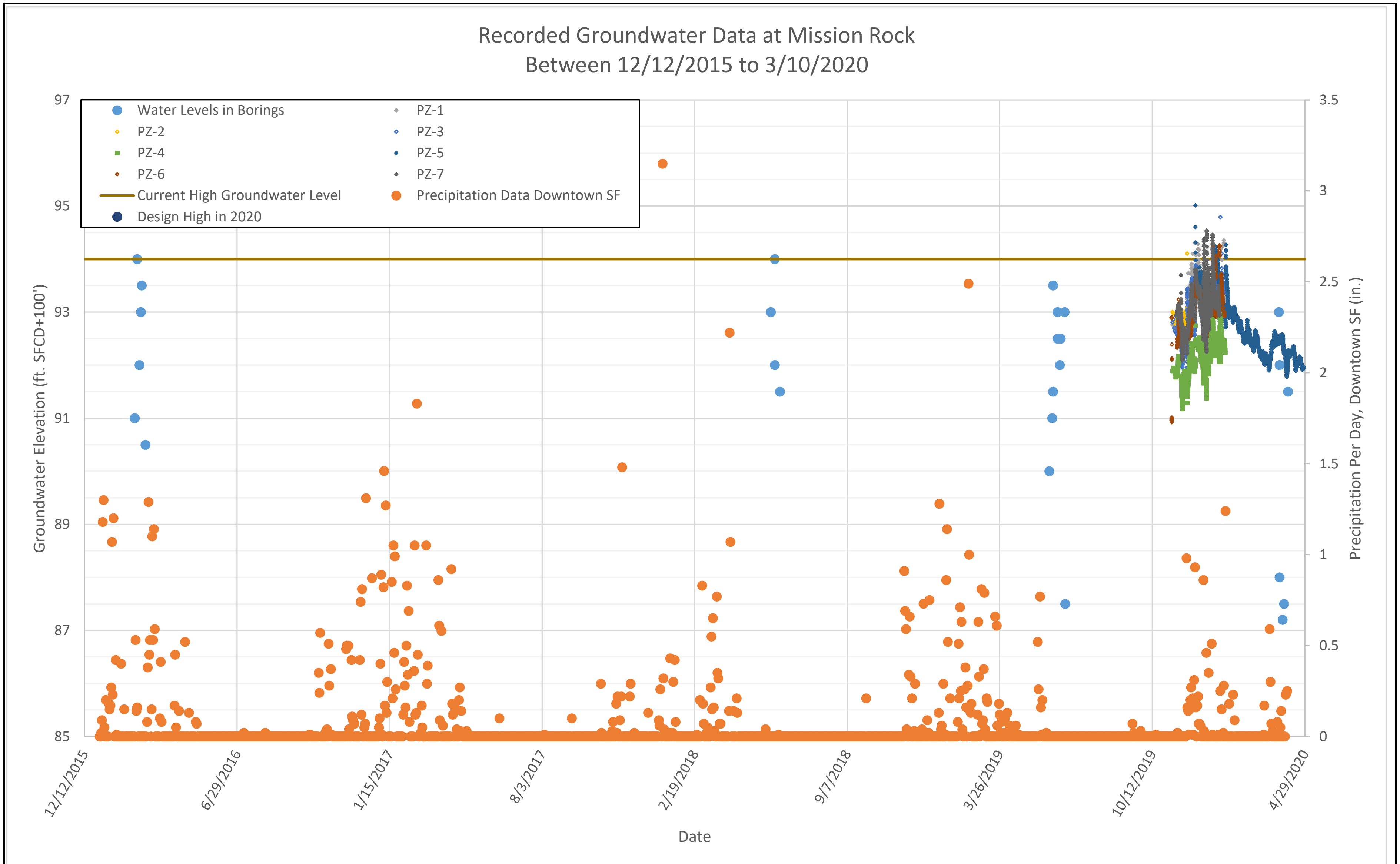
The design future high groundwater level accounting for (SLR) is 99.5 feet (Mission Bay Datum). This is based on the current high groundwater level observed at the site of Elevation 94 feet and adding the estimated high SLR of 66 inches to get to Elevation 99.5 feet. We understand DPW is concerned that the current high groundwater level of 94 feet does not represent existing conditions.

To help illustrate our conclusions that Elevation 94 is the appropriate existing high groundwater level we have plotted all of the water levels recorded at the site and included measured daily precipitation data from downtown San Francisco (see Figures 1 and 2). Elevation 94 matches the highest elevation of groundwater observed prior to the installation of the pilot test program at Mission Rock. During the test programs, piezometers were installed. Data from the piezometers do show short-term spikes in the data that extend above Elevation 94. However, Piezometers PZ-1 through PZ-4 are open standpipe piezometers placed in christie boxes that are flush with the surrounding asphalt--in these piezometers we see short-duration (usually less than 4 hours) spikes due to surface water that was allowed to enter the piezometers. Piezometers PZ-5 through PZ-7 are grouted-in-place load-cell piezometers that were installed in areas where the pavement had been removed, and rainwater was allowed to infiltrate into the fill—these piezometers show short-term (ranging from 1 to 12 hours) spikes in localized water levels up to Elevation 94.5 or 95 feet during storm events. However, the final conditions at Mission Rock will be paved, and little to no rainwater will be able to infiltrate into the ground. Immediately following each of these events, we see a quick dissipation of the water pressure, indicating the water is effectively dispersing into the static groundwater level.

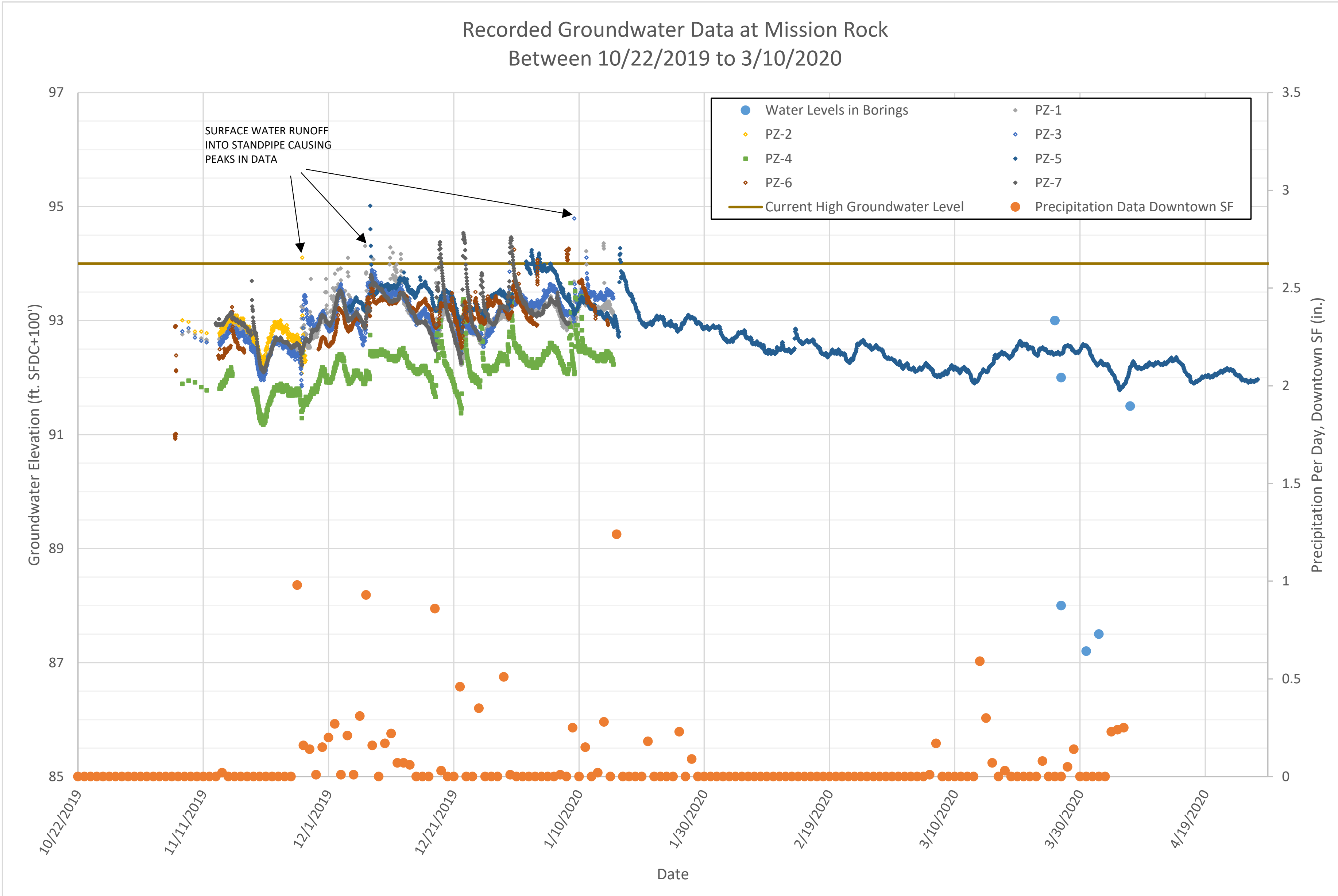
Based on all of the available groundwater data and considering that the future use across Mission Rock Phase 1 will generally be impermeable (with the sole exception of tree wells in sidewalks) we conclude the use of Elevation 94 feet for a high groundwater elevation is appropriate for design. The corresponding increase in groundwater level to Elevation 99.5, corresponding to the high estimate of SLR of 66 inches, is considered appropriate.

Attachments: Figures 1 and 2 – Groundwater Data
Exhibit A – Saturated Density of LCC Calculations

750604203.29 PBD_Mission Rock LCC Review Response_2020-04-28



MISSION ROCK **PHASE 1 DEVELOPMENT** San Francisco, California	Precipitation Data from NOAA for statation GHCND:USW00023272	**GROUNDWATER MEASUREMENTS AND EQUIVALENT WATER PRESSURE ELEVATIONS**	
LANGAN		Project: 750604203	4/28/2020	Figure 1



MISSION ROCK **PHASE 1 DEVELOPMENT** San Francisco, California **LANGAN**	Precipitation Data from NOAA for statation GHCND:USW00023272	**GROUNDWATER MEASUREMENTS AND EQUIVALENT WATER PRESSURE ELEVATIONS** Project: 750604203 · 4/28/2020 · Figure 2

Exhibit A

Saturated Density of LCC Calculations

Goal:

Evaluate the likely long-term saturated density of submerged LCC for use in calculations. at Mission Rock, where the target LCC cast density = 26 pcf

Given:

LCC entrains air bubbles. For permeable LCC, some of the air bubbles become interconnected to allow water to flow through the sample. However, some air bubbles will remain in place, making the long-term saturated density challenging to estimate.

Available Information:

1) Conventionally Saturated LCC from the project data has a density between 50 pcf and 61 pcf (see Pages 4 + 5 with density information from laboratory testing).

2) Vacuum Saturated LCC testing was performed to try to simulate long-term conditions on samples from the Stanford Project.
   For LCC with a cast density < 30 pcf, the maximum Saturated density = 62.5 pcf - see page 4

3) Based on a target Cast density of 26 pcf and a w/c ratio of 0.55 for the LCC paste, we estimate the maximum theoretical LCC density could be 74 pcf. This assumes every air bubble will become completely filled with brackish water.

Conclusion:

For geotechnical calculations for load-offset, we need to estimate the "likely" fully submerged density. We conclude this density may be higher than the laboratory tests, but some air will likely remain entrained. We therefore conclude taking the average of

Mission Rock Horizontal Saturated Density of LCC	BY SAW DATE 4/24/2020 CKD. DATE	PROJ. NO. 750604203 SHEET 1 OF 5

LANGAN

Conclusion:

For the theoretical load offset calculations (that include a reduction in stress of 10 % @ the Bay Mud surface) we need to estimate the likely fully submerged density for long term conditions. We conclude this density could be higher than the laboratory test data, but would be lower than the theoretical maximum - as some air bubbles will remain entrained. We therefore conclude that taking the average of the heaviest representative lab data density and the theoretical maximum density is appropriate for load-offset calculations.

$$\therefore \text{ Saturated density } = \frac{62.5\text{ pcf} + 74\text{ pcf}}{2}$$

$$= 68.25 \longrightarrow \boxed{\text{Use } 68 \text{ pcf}}$$

These density values are only to be used for load-offset calculations, where a heavier density is conservative.

For uplift calculations, a lighter saturated density is conservative. For these, use the lightest representative laboratory data. - See pages 4 + 5 of this calc. - Recommend using 50 pcf for uplift Calcs.

Mission Rock Horiz.	BY SAW DATE 4/28/2020	PROJ. NO. 750604203
	CKD. DATE	SHEET 2 OF 5

LANGAN

Goal: Determine maximum density of LCC if all voids are filled with water.

Given / Assumed:

Cast Density = 26 pcf – Target cast density per Spec.
W/cement Ratio = 0.55 – Example from Pilot Test Mix design
γ_w = 63 pcf – Salt water (for the final condition) γ_w = 62.4 pcf for fresh water
Paste Density = 111.5 pcf – From Pilot Test Mix design.
Specific Gravity of Cement = 3.15 $\therefore \gamma_{cement}$ = 196.6 pcf (62.4 pcf × 3.15)
Expansion Factor = 111.5 pcf / 26 pcf = 4.29
% of Paste that is cement = $1/_{1+w/c}$ = $1/_{1.55}$ = 0.645 by weight

Solve for: Volume of Water + Cement.

Volume of Cement in Paste = $\dfrac{\text{Wt. of Cement}}{196.6 \text{ pcf}}$

Wt of Cement = % of Pasete that is cement · Paste density
= 0.645 · 111.5 pcf = 71.9 # of cement per c.f. of Paste

Volume of Cement in paste = 71.9 #/cf / 196.6 pcf = 0.3657 ft³/ft³ of paste

Volume of Cement in LCC = $\dfrac{\text{Volume in paste}}{\text{Expansion Factor}}$ = $\dfrac{0.3657 \text{ ft}^3/\text{ft}^3 \text{ of paste}}{4.29}$
" " " " " = 0.0852 ft³/cubic foot of LCC

If all voids are filled with water:

Volume of Voids = 1 – Volume of cement = 1 – 0.0852
= 0.9148 ft³

Weight of voids when filled with water = 0.9148 × $\gamma_{w\,salt}$
0.9148 × 63 pcf = 57.6 pcf

Weight of cement = 0.0852 ft³ · 196.6 pcf
= 16.75 pcf

Total Theoretical Maximum Unit weight = 57.6 pcf + 16.75 pcf
= 74 pcf

Mission Rock Horizontal	BY SAW DATE 4/24/2020	PROJ. NO. 750604203
	CKD. DATE	SHEET 3 OF 5

Field & Lab Testing Data Summary, LLC Samples from Stanford

Sample ID	Field Testing (Montez Group)				Lab Testing (Castle Rock)			
	Cast Density (pcf)	Sat. Density (pcf)	Perm. (cm/sec)	Vacuum Sat. Density* (pcf)	Cast Density (pcf)	Sat. Density (pcf)	Perm. (cm/sec)	Vacuum Sat. Density (pcf)
Stanford 1	27.5		8.77E-03		27.5	51.1	5.50E-02	
Stanford 2	29.5	40.5	1.34E-01		29.5	50	1.80E-03	
Stanford 2	26.5	53.4	6.18E-04		29.5	52	5.70E-04	
Stanford 2	26.5				29.5	52		
Stanford 3					26.5	54	1.30E-01	
Stanford 3					26.5	55	8.80E-02	
Stanford 1	24.5	50.5		59.3	24.8	60.4		62.8
Stanford 2	29.8	51.8		62.5	31	56.9		61.8
Stanford 3	32.2	57.9		63.2				
	Average =	50.8	pcf	61.7	Average =	53.9	pcf	62.3
	StdDev =	5.7	pcf	1.7	StdDev =	2.0	pcf	0.1

LCC Samples obtained at the Stanford projec and tested by special inspectors and TAP as part of the LCC Evaluation; mix similar to Mission Rock.
* Vacuum Saturated Density performed by Montez Group performed in a separate laboratory

Field & Lab Testing Data Summary, Samples From Mission Rock Project

Sample ID	Field Testing (Montez Group)				Lab Testing (Castle Rock)		
	Cast Density (pcf)	Sat. Density (pcf)	Perm. (cm/sec)		Cast Density (pcf)	Sat. Density (pcf)	Perm. (cm/sec)
Pilot Lift 1	26	51	8.65E-03		26	53.5	1.10E-01
Pilot Lift 1	25.5	57.5	4.09E-01		26	53.1	5.70E-02
Pilot Lift 1					25.5	58.1	5.20E-01
Pilot Lift 1					25.5	58.7	4.30E-01
Pilot Lift 1 Pour Back		60.7	1.70E+00				
Pilot Lift 2	28	47.7	9.34E-01		28.7	53.1	4.20E-02
Pilot Lift 2					28.9	52	3.70E-02
Pilot Lift 2					29	53.6	4.30E-02
Pilot Lift 2	27.5	40.8	4.25E-01		28.9	54.5	5.90E-02
Pilot Lift 3	29	56.3	1.01E-02		29	55.7	5.40E-02
Pilot Lift 3	27.5	53.2			29	56.8	1.20E-01
Pilot Lift 3					27.5	57.1	1.00E-01
Pilot Lift 3					27.5	55.8	7.30E-02
Pilot Lift 4	29	47	8.31E-03		29	52.8	2.90E-02
Pilot Lift 4	28	46.9			29	52.5	6.20E-03
Pilot Lift 4		44.3					
Pilot Lift 4		45.2					
Pilot Lift 4		45.2					
Pilot Lift 5	30	44.3	3.73E-02		**30.8**	**51.6**	6.30E-03
Pilot Lift 5					**31.1**	**52.8**	1.10E-02
Utility Pour Back 1	23.5	54.5	2.48E-03				
Utility Pour Back 2	26.25	54.8	4.95E-04				

Bolded Values were reported incorrectly in Montez Report Table

Saturated Density from All Cast Samples applicable to MR Includes all data above (Mission Rock and Stanford)						
Field Testing (Saturated Density)				Lab Testing (Saturated Density)		
Average =	51.7	pcf		Average =	54.9	pcf
StdDev =	6.6	pcf		StdDev =	3.3	pcf
Minimum=	40.5	pcf		Minimum=	50.0	pcf

Saturated Density from Lower Cast Densities (26 to 29 pcf) Excludes Lift 5 from Pilot and high Cast Densities from Stanford						
Field Testing (Saturated Density)				Lab Testing (Saturated Density)		
Average =	51.5	pcf		Average =	55.1	pcf
StdDev =	5.7	pcf		StdDev =	2.6	pcf
Minimum=	40.5	pcf		Minimum=	50.0	pcf

Average MR Data from Lower Cast Densities (26 to 29 pcf) Excludes Lift 5 from Pilot and all Stanford Data						
Field Testing (Saturated Density)				Lab Testing (Saturated Density)		
Average =	50.4	pcf		Average =	54.8	pcf
StdDev =	5.7	pcf		StdDev =	2.1	pcf
Minimum=	40.8	pcf		Minimum=	52.0	pcf

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Additional Field Testing on Cored Samples from Pilot			
Sample ID	**As-Cored Density (pcf)**	**Sat. Density (pcf)**	
Core- Pilot Lift 1		58.6	
Core- Pilot Lift 1		60.1	
Core- Pilot Lift 1		58.6	
Core- Pilot Lift 3	24.1	60.7	
Core- Pilot Lift 4	25.1	61.3	
Core- Pilot Lift 4	23.5	60.2	
Core- Pilot Lift 5	23.8	59.8	
Average =	24.1	59.9	pcf
StdDev =	0.6	0.9	pcf
Minimum=	23.5	58.6	pcf

Samples prepared by Contractors bidding on the Mission Rock Project							
	Field Testing			**Lab Testing**			
Sample ID	**Un-sat density (pcf)**	**Sat. Density (pcf)**	**Perm. (cm/sec)**	**Un-sat density (pcf)**	**Sat. Density (pcf)**	**Perm. (cm/sec)**	**Contractor**
T-27-60-A	28.0	53.7	1.40E+00	28.0	50.7	4.40E-01	Throop Cellular Concrete
T-27-60-B	30.4	55.2	1.50E+00	30.4	50.8	3.10E-01	
CF-20	23.3	60.0	3.10E+00	23.3	62.4	1.60E+00	CellFill, LLC
CF-25	25.9	60.4	1.80E+00	25.9	62.6	8.20E-01	
CF-27	26.5	59.6	5.20E-01	26.5	62.2	7.30E-01	
CF-30	30.8	60.9	2.60E-01	30.8	63.9	1.80E-01	
	Average =	58.3	pcf	Average =	58.8	pcf	
	StdDev =	0.5	pcf	StdDev =	0.7	pcf	

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